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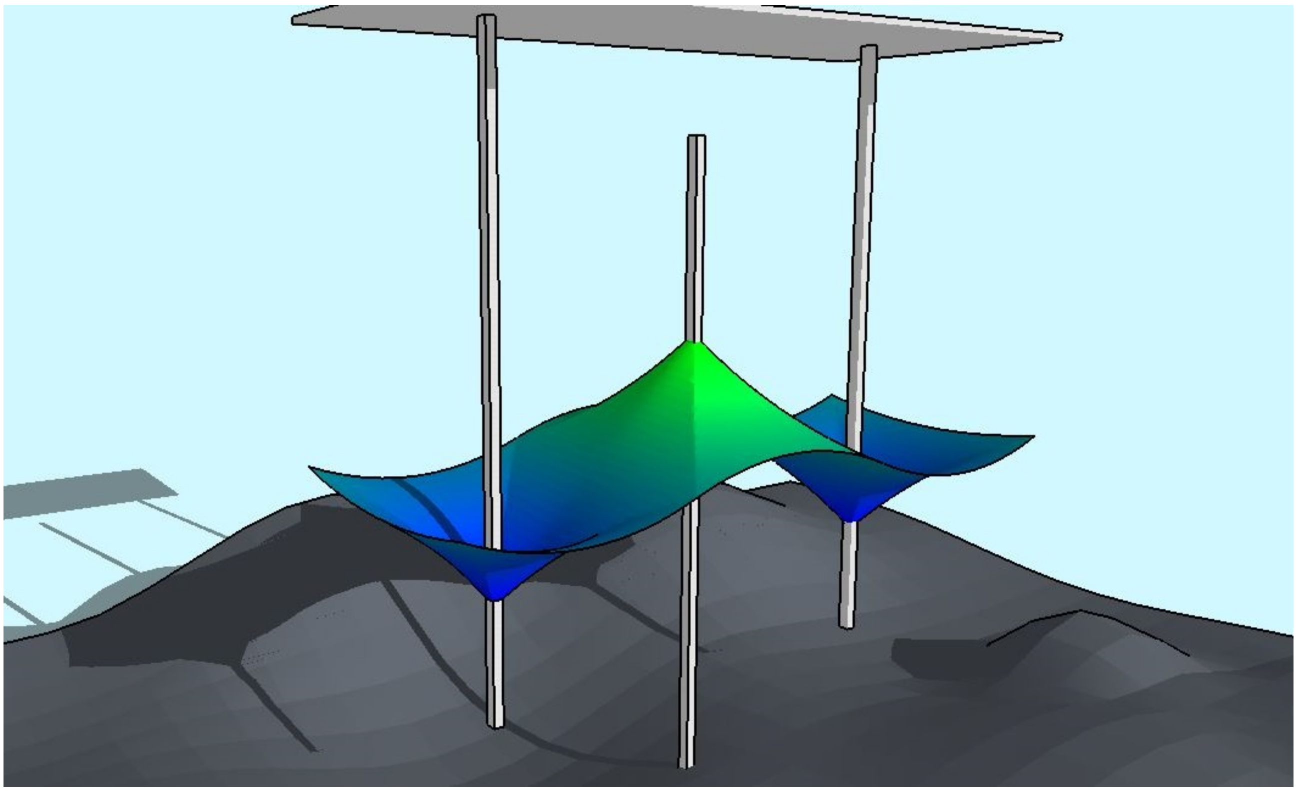

On long term behaviour of overlapping pile foundations

Master's thesis in the Master's Programme Structural Engineering and Building Technology

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Cover:
A 3D illustration of overlapping piles

Chalmers Reproservice
Gothenburg, Sweden 2016

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ABSTRACT

Piles are used world wide as foundation or as a soil strengthening method. The purpose of this thesis is to analyse how a system of overlapping piles, as a foundation technique, behaves over a design life of 100 years. Commonly piles are installed as either floating or end bearing which both have their advantages and drawbacks. In the early sixties a new technique was introduced which combined the two traditional methods in order to cope with the disadvantages from using one method alone. Overlapping piles have been used widely in Mexico City but in Sweden this piling system has only been used in a handful of projects, all constructed within the last decades. The idea of analysing overlapping pile systems with traditional pile systems was brought alive with the project of Regionens Hus located in the center of Gothenburg. The analysis is carried out with the finite element software Plaxis 2D together with the soil model Creep-SCLAY1S in order to capture long term behaviour of the clay. Soil model parameters are derived from the soil, CRS and triaxial tests conducted at Regionens hus. Several models, of both overlapping piles and more common floating pile systems, are created and compared in order to study the differences in terms of forces, settlements and soil response. As long term behaviour is dependent on soil loading history, the installation effects from pile driving are considered. It is shown that the most suitable way of modelling overlapping piles in Plaxis 2D is with the use of plate elements in a plane strain environment. With these it is possible to capture some installation effects as well as settlements, force distribution and load transmission. However, when using plate elements in 2D plane strain analysis Plaxis will consider piles as rigid walls. This wall effect will create a soil response that in some cases are not realistic. To capture the installation effects, without compromising the result, it is shown that a suitable method is to replace a volume of soil around the pile with remodelled soil. It is also shown that if overlapping piles are used it is possible to either shorten the floating piles about 50 % or increase the spacing between them about seven times compared to a system with floating piles only. This thesis indicates that there might be something to the idea of overlapping piles in terms of capacity and suitability, but more studies have to be conducted in order to confirm it, for example a study of economics as the price in most cases govern what foundation technique is going to be used. Additionally, the results from this thesis should ideally be verified and compared to real measurements and more advanced models.

Keywords: Overlapping Piles, Creep-SCLAY1S, Installation effects, Plaxis 2D, FEM, Soft soil

CONTENTS

PREFACE

First of all we would like to thank our examiner Prof. Minna Karstunen and Dr. Jelke Dijkstra for their encouragement to act and think independently and for the guidance into interesting aspects of the subject.

We also would like to thank Anders Kullingsjö and Cecilia Edmark, our supervisors at Skanska, for the opportunity to write this thesis. We have appreciated all our discussions and the valuable input you have provided us throughout the project.

In addition we acknowledge the invaluable help we have received from Per-Ola Svahn, Torbjörn Edstam and Sven Liedberg from Skanska. We thank you for the interest you have shown in our project and for our conversations during the, sometimes extended, coffee breaks.

Finally we would like to thank Ph.D. Students Jorge Yannie and Amardeep Amavasai for their insights in pile installation and soil modelling respectively. Last but not least we are grateful to research engineer Peter Hedborg for sharing his expertise in laboratory testing and increasing our understanding of soil behaviour.

NOMENCLATURE

Roman letters

A	Cross sectional area of piles
b	Pile width
c_u	Undrained shear strength
E	Young's modulus
$E_{A,B}$	Young's modulus for A- and B-piles
E_u	Undrained modulus
e_0	Initial void ratio
I	Moment of inertia
K_0^{NC}	Lateral earth pressure at rest (Normally consolidated soil)
$L_{A,B}$	Length of A- and B-piles
L_c	Distance between B pile head and foundation
L_o	Length of overlap
L_{tot}	Total length of piles in system
M_c	Critical state line in compression
M_e	Critical state line in extension
p'	Mean effective stress
p'_p	Size of normally consolidated surface
p'_{eq}	Size of current state surface
p'_{mi}	Size of intrinsic yield surface
q	Deviatoric stress
q_n	Negative skin friction
r_p	Radius of plastic zone
S_T	Sensitivity
Sp_A	Spacing between A-piles
Sp_{AB}	Spacing between A- and B-piles
s'	Average value of stress
t	Difference between stress
u_y	Displacements in Plaxis y-direction
$w_{A,B}$	Pile width in overlapping pile models

Greek letters

α	Adhesion factor
α_f	Pore pressure coefficient
$\alpha_{x,y,z}$	Creep-SCLAY1S state variables
α_0	Initial inclination of yield surface
β	Friction angle coefficient
γ'	Buoyant unit weight
$\Delta\epsilon$	Strain increment
Δu_s	Pore pressure at shaft
Δu_t	Pore pressure at tip
ϵ_{xx}	Strain in x-direction
η_{K_0}	Initial stress ratio
κ^*	Swelling index
λ^*	Modified compression index
λ_i^*	Intrinsic compression index
μ^*	Modified creep coefficient
μ_i^*	Intrinsic creep coefficient
ν	Poissons ratio
ξ	Absolute rate of destruction
ξ_d	Relative rate of destruction
σ'_a	Axial stress
σ'_c	Effective pre-consolidation pressure
σ'_r	Radial stress
σ'_v	Effective vertical stress
σ'_{v0}	Initial effective vertical stress
$\sigma'_{1,2,3}$	Principle stresses
τ_d	Reference time
$\tau_{fu,v}$	Uncorrected shear strength
ϕ'_c	Friction angle
ϕ'_{cs}	Critical state friction angle
χ_0	Initial bonding
ω	Absolute effectiveness in rotational hardening
ω_d	Relative effectiveness in rotational hardening

Abbreviations

FE	Finite Element
OCR	Over Consolidation Ratio
POP	Pre Overburden Pressure
NP	Neutral Plane
CEM	Cavity Expansion Method
ULS	Ultimate Limit State
SLS	Serviceability Limit State
CRS	Constant Rate of Strain
WIP	Wished In Place
TSP	Total Stress Path
ESP	Effective Stress Path
CSS	Current state surface
NCS	Normally consolidated surface

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1 Introduction

In the Gothenburg area, the common practice when planing piled foundations is to either design the piles as end bearing or floating. Both methods have their drawbacks, but these can be reduced with a intuitive use of the systems combined.

1.1 Background

Piles have been used worldwide as a foundation method for decades. Two common piling techniques are floating piles and end bearing piles. In the early sixties a new piling technique was introduced which combined the two previous piling techniques. The new technique is called overlapping piles and it has been used widely in Mexico City. In Sweden there is only a handful of constructions that uses this type of foundation. All of these are built in the past 25 years and there is no long term studies of these. Skanska won a tender in 2015 for the construction of Regionens Hus in central Gothenburg where one suggested solution for the foundation was based on the use of this new technique. Since there is limited research done on this foundation technique it suited well for a Master Thesis.

1.2 Problem description

Foundations are designed to fulfil demands of stability and serviceability. The foundations on deep clay layers in Gothenburg are usually governed by the latter. In general, constructions are designed for a lifetime of either 50 or 100 years and the serviceability demands of piles are usually expressed in terms of settlements. These can be either a maximum allowable settlement for the structure or a maximum relative settlement between the structure and the adjacent soil. An engineer can predict these settlements in many different ways, where the use of finite element analysis is one option. The problem is that there exists only limited papers and guidelines on how to treat the new pile system, especially when using finite element analysis. Since the overlapping pile technique is relatively new in Sweden, it can be tricky to convince a contractor to use it. The main reason for this is the limited knowledge of the potential benefits of the technique, as well as the lack of documented history of the long term effects.

1.3 General aim

The main purpose of this Master Thesis is to examine, by finite element analyses, how the overlapping pile foundation technique behaves and affects the soil over the total lifespan. This includes the installation of the piles, as well as the lifetime of the overlying structure. It is of interest to know how the soil is affected with variations in the pile positioning and geometry. This leads to evaluating potential benefits and limitations of using this technique with an ambition of optimizing the geometry of the pile system.

1.4 Definitions

In order to study this subject some simplifications are made.

- The idea is to study the behaviour of the piling technique, and not to evaluate the suggested foundation at Regionens Hus. Hence, only some data, such as soil properties, relative positioning and spacing between piles will be taken from the actual project.
- Long term effects will be studied for 100 years after construction. Certainly there will be some major changes in the area due to other new constructions and projects which will affect the soil. In the model there will be no consideration to these eventual changes.
- All modelled piles are considered to be:
  - Perfectly straight without initial imperfections or deformations from installation.
  - Subjected to vertical loads without any eccentricity.
  - Driven precast concrete piles.

2 The behaviour of piles in soft soils

All construction projects involve foundations to some extent, and based upon the underlying material different foundation solutions are better than others. Piles have been used for decades as foundation on soft soils to ensure enough stability for constructions. In order to analyse and model a problem involving soft soils it is crucial to understand the basic behaviour and physics of soft soils. In this Chapter the most relevant basic concepts of soft soils and piles are presented.

2.1 Behaviour of soft soil

Soil, from an engineering point of view, can be treated as a three-phase material; particles, water and gas. The particles are usually minerals from weathered rock but can also be of an organic substance. Between the particles there are voids which can be filled with either water, gas or both. The amount and composition of either of the three phases will affect how the soil behaves and as opposed to for example steel, soil is highly anisotropic.

When soil is undisturbed in its natural environment there is an equilibrium between the forces acting on the particles in the soil. These forces, which can origin from the self weight of the soil or the level of the water table, are causing stresses within the soil structure. If something should change this equilibrium, may it be from added loads from a construction or a change in the water table, the stress state in the soil is also changed. In the following sections relevant soil behaviours is described that are of importance to the study.

Pure elastic behaviour is easily described by considering a simple rubber band. If tensile forces are applied, the rubber band will stretch out. When these forces are removed the rubber band will go back to its original shape and length, hence a pure reversible elastic behaviour is observed. Plastic behaviour can be described by considering a thin metal plate. This plate can easily be bent by hand and if it is bended just slightly it will return to original shape after releasing it. But if it is bent over a certain amount the deformations will be permanent and hence an irreversible plastic behaviour is observed.

Common construction materials are usually modelled by considering an elastic and a plastic region in a strain-stress plot. It is also usual to define a yield point at which the material will change from elastic behaviour to plastic behaviour, see Figure 2.1. In soft soils it is instead common to talk about a yield surface which can be defined in a s'-t plane (plane strain) or a p'-q plane, see Figure 2.2. The planes are defined in Equation (2.1) and Equation (2.2) which in an triaxial test ($\sigma'_a = \sigma'_1$ and $\sigma'_2 = \sigma'_3 = \sigma'_r$) can be simplified to Equation (2.3).

$$s' = \frac{\sigma'_1 + \sigma'_3}{2} \qquad t = \frac{\sigma'_1 - \sigma'_3}{2} \tag{2.1}$$

and

$$p' = \frac{\sigma'_1 + \sigma'_2 + \sigma'_3}{3} \qquad q = \sigma'_1 - \sigma'_3 \tag{2.2}$$

$$p' = \frac{\sigma'_a + 2\sigma'_r}{3} \qquad q = \sigma'_a - \sigma'_r \tag{2.3}$$

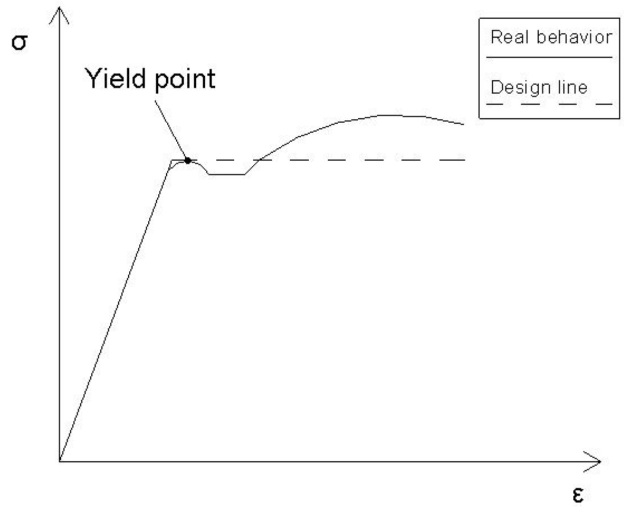

Figure 2.1: A typical stress-strain relationship for a metallic material and a idealization that can be used when modelling the yielding.

σ_1' and σ_3' are principal stresses in the shear plane and σ_a' and σ_r' represents the axial and radial stress (cell pressure), as in a triaxial test (Muir Wood 1990). All of these equations are special cases of more general principal stress equations in an 3D space, but are ideal for the purpose in this thesis.

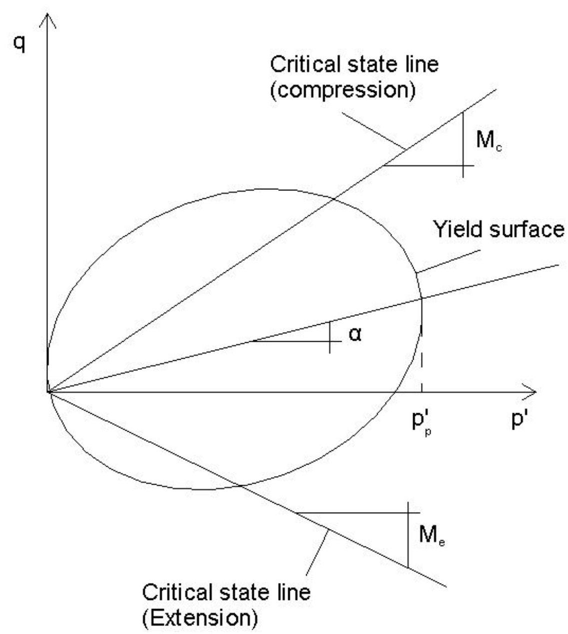

Figure 2.2: Yield surface plotted in the p'-q plane.

Consider a small element of soil in its natural environment. This element will be in an equilibrium corresponding to a stress state somewhere in the p'-q plane. Figure 2.2 illustrates a yield surface in the p'-q plane and a change of stress on the soil element that lies within this yield surface would not cause the soil to yield. It also shows two critical state lines, one compression and one extension. The scalar quantity α describes the orientation of the yield surface in triaxial space. For more details about the behaviour of an elastic-plastic model for clay the reader is adviced to Muir Wood (1990). Determining the yield surface's angle and shape is of great importance in geotechnical problems in order to predict a realistic behaviour of the soil.

As mentioned earlier, soft soils consists of both particles, water and gases. Imagine holding a chunk of wet clay in your hands and then squezing it. This will cause the water to flow out of the clay and the volume of the clay to reduce. In the field of geotechnics this volume reduction is known as consolidation. This is a well known phenomena, and it is common to talk about both primary and secondary consolidation where primary consolidation refers to the volume reduction when water is being squeezed out of the soil. The amount and rate of this primary consolidation depends on numerous factors such as excess pore pressures and the compressibility of the soil (Havel 2004).

Secondary consolidation, or creep, is more complex and origins from a change in the soil structure. When all water (and gases) is squeezed out from the soil only the particles remain and if the soil is still loaded this can cause further deformations of the soil, refered to as creep. This can be described as increased deformations or strains during loading over time. In other words how the stresses and strains relate to each other over time. Both primary and secondary consolidation are time dependent but creep is a slowly occurring phenomena (Havel 2004). This time dependency can be formulated into mathematical terms with some sort of relationship between stress σ, strain ϵ and time t such as:

$$\sigma = f(\epsilon, t)$$

The anisotropy of soft soils is one reason for the complexity of consolidation. First of all the permeability of the soil is not the same in all directions thus effecting the primary consolidation. Over the last century there have been numerous proposals on consolidation theories and one of the first, and still widely used, is the 1-dimensional consolidation theory by Terzaghi (1923). This theory is based on a few assumptions where one is that the permeabillity is constant which is not accurate. Later more advanced and complex way of treating consolidation problems has been proposed, such as 3-dimensional theory by Biot (1941) which includes flow of water in all directions. Today there are also several finite element analysis programs that offers various ways of treating both primary and secondary consolidation. But regardless of what method used, the result rely on capturing the real behaviour of the soil as close to the reality as possible.

Work-hardening or strain hardening is a well known behaviour of metallic materials experiencing stresses above their yield limits. Simply put strain hardening will increase the yield stress of the material at the expense of ductility. In most metals there is a clear distinction between the elastic and plastic region. Usually soil models define a yield criterion as a distinction between an elastic-plastic region and a plastic strain hardering region. Muir Wood (1990) shows an elegant elastic-plastic model with a yield surface that changes due to plastic hardening.

When a soil structure is changed the particles in the soil are rearranged. The particles can be arranged in different ways and some arrangements are more closely packed than others. If some particles in a coarse-grained soil are rearranged due to shear, causing a volume increase, it is known as dilatancy (Craig and Knappett 2013).

Dilatancy is a more studied phenonema in coarse-grained soils, such as sands, but can be relevant in other soils. Dilatancy is not only dependent on the grain sizes of the particles but also the shape of them (Santamarina and Cho 2004). Commonly used soil models usually have some way of dealing with dilatancy and a dilation angle is introduced (variables β or Ψ), describing the relation between plastic volumetric strain and shear strain (Muir Wood 1990).

2.2 Piles

Sometimes the bearing capacity of soft soils is not sufficient to carry the extra load from a planned construction. In these cases some sort of soil strengthening is required. A commonly used method to treat the problem is with the use of piles. Piles are used to transfer loads from an overlying structure to a underlying layer with enough bearing capacity. One way is to transfer the loads all the way down to bedrock or into an underlying, more rigid, soil layer. These type of piles are referred to as end-bearing. Another type of piles are so called floating piles which are installed in the soil with the pile head not reaching a harder surface, see Figure 2.3. The bearing capacity of floating piles are determined by the soil-pile interface.

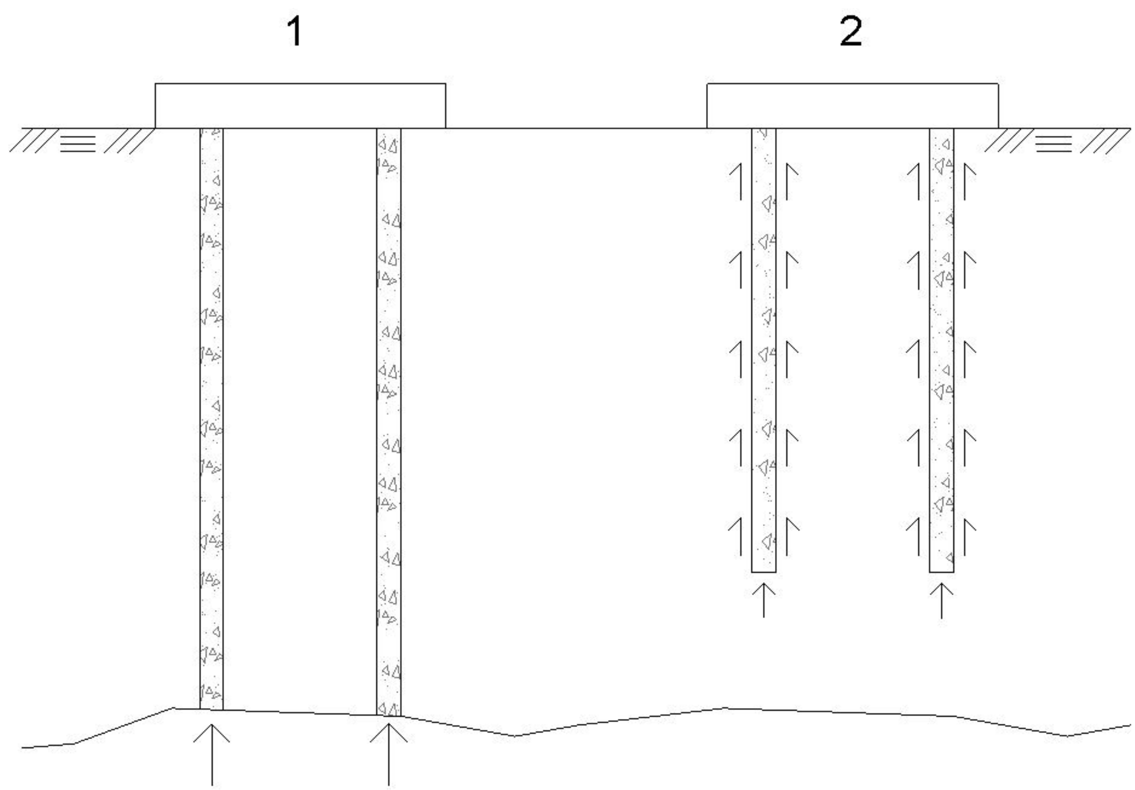

Figure 2.3: Principle of common pile foundations. 1: End-bearing piles, 2: Floating piles.

End-bearing piles transfer the load to an underlying rigid layer. These piles can be seen as columns surrounded with soil connecting the overlying structure and the underlying rigid layer. Most of the load goes through the pile but some parts of the load is carried at the pile-shaft interface by friction. This is a rigid solution but one disadvantage when using end-bearing piles is the settlement differences that occurs between the soil directly under the construction and the soil in the nearby region. The differences occurs because the rate of consolidation will decrease, since the soil close to the piles is partly unloaded by friction between the piles and the soil. The surrounding soil will however undergo the general subsidence which will cause the building to sequentially emerge from the neighbouring constructions and utilities, built on foundations that allow ground settlements (Girault 1964). Another drawback is when the soil directly underneath the foundation settles, small pockets of air will form under the foundation.

The bearing capacity of floating piles is dependent on the soil-pile interface strenght along the shaft and at the toe. For longer piles the toe capacity becomes small in relation to the shaft and is often neglected in calculations. The term bearing capacity of a single pile refers to the amount of load when the pile starts to settle at a constant or increasing rate (Karl et al. 1996). The total bearing capacity can be calculated by summing the bearing capacity from the pile base and the bearing capacity from the shaft.

Equal to the end-bearing piles, the floating piles have some disadvantages that has to be taken into account when designing foundations. For instance, the pile toe will have to leave sufficient space down to firm ground in order not to become end-bearing. This is governed by the predictions of the soil settlements for the building's life span. Another factor is that the building might experience some uneven settlements due to a number of factors, such as uneven load distribution between the piles and varying soil properties, or other factors that might have been overlooked during the design. A third aspect is that the soil is disturbed during the pile driving which in turn means that the full bearing capacity of the pile cannot be utilized at once. The bearing capacity will, however, develop over time as the soil reconsolidates and reach its full intended capacity after some time (Lied 2010).

2.3 Negative skin friction

Negative skin friction occurs when the surrounding soil settles more than the pile. In clay this is mainly caused by consolidation and creep. The consolidation rate can be increased by loading of the soil or lowering of the ground water level. When the soil settles more than the pile this will cause a drag load to be induced to the pile with a magnitude that will increase whith the pile length (Fellenius 1984). A similar phenomena occurs when the soil surrounding a pile is unloaded, for example by an excavation or an increase of the ground water level, which will cause the soil to swell. This will induce positive skin friction to the pile which will try to lift it, which can be referred to as a pull-out load. The underlying mechanism of both negative and positive skin friction is that there exists relative displacements between the pile and the soil causing shear forces along the pile surface. The magnitude of negative skin friction is dependent on both the earth pressure coefficient of the soil K_s, the drained interpretation of the soil friction angle ϕ', and the pile surface (Fellenius 1984). The practice in Sweden is to calculate the negative skin friction using the wellknown α and β methods according to Equation (2.4) and Equation (2.5) (Eriksson et al. 2004) and (Alheid et al. 2014).

$$q_n = \alpha c_u \tag{2.4}$$

and

$$q_n = \beta \sigma'_{v0} \tag{2.5}$$

Equation (2.4) is for the undrained case and Equation (2.5) is for the drained case. q_n is the negative shear force or negative skin friction, α is an adhesion factor and β is a coefficient based on the effective stress friction angle of the soil. If the calculation is based on Eriksson et al. (2004) the value of c_u should be taken as the uncorrected shear strength value $\tau_{fu,v}$ from field vane tests. σ'_{v0} is the initial effective vertical stress.

The adhesion factor α can be empirically determined to 0.7 (Eriksson et al. 2004). There are numerous ways of determining the β coefficient, both analytical and empirical (Parry and Swain 1977). In Sweden it is common to use the empirical value of 0.25 - 0.30 for the β coefficient for clays (Eriksson et al. 2004).

The bottom part of a floating pile is subjected to positive skin friction, sometimes referred to as positive shaft resistance, while the upper part is subjected to negative skin friction. The point where the change between positive and negative skin friction, and hence a change of direction in the shear stresses along the pile, occurs is called the neutral plane and is also the point along the pile where there is no relative movement between the soil and the pile, see Figure 2.4.

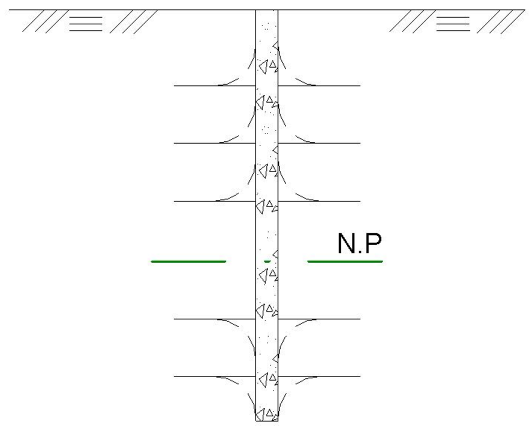

Figure 2.4: Location of the neutral plane.

The exact location of the neutral plane is dependent on various factors. It is dependent on the applied load both in terms of magnitude, point of load application and type of loading, for example push or pull load (Fellenius 1984). If there is a change of these, there will be a change in the position of the neutral plane. The soil properties also determine where the location will be. For example if the pile toe is resting on a dense medium, such as hard packed sand, the neutral plane will end up lower than by the same pile situated purely in clay. Another example is the end-bearing piles standing on bedrock where the neutral plane will be situated close to the pile toe (Fellenius 1984).

The determination of the neutral plane can be done by calculating an equilibrium between the applied load together with the negative shaft resistance, contra the toe resistance together with the positive shaft resistance (Fellenius 1984). This can be illustrated by plotting all forces with depth, see Figure 2.5.

A single pile will be compressed slightly due to loads acting on the pile. If neglecting this compression the settlement of the pile is equal to the amount of settlements in the soil at the level of the neutral plane (Fellenius 1984). Several field studies during the last century have been conducted in order to study the settlements on piles and are summarized in a publication by Fellenius (1998). Some conclusions made was that the load distribution relates to effective stress principles and is hence dependent on pore pressure distribution. This distribution is changed due to installation of piles, which affects the negative skin friction. Also secondary compression, or creep, will affect the negative skin friction. Hence, if it is possible to change the position of the neutral plane, by for example the use of overlapping piles, the magnitude of settlements can change.

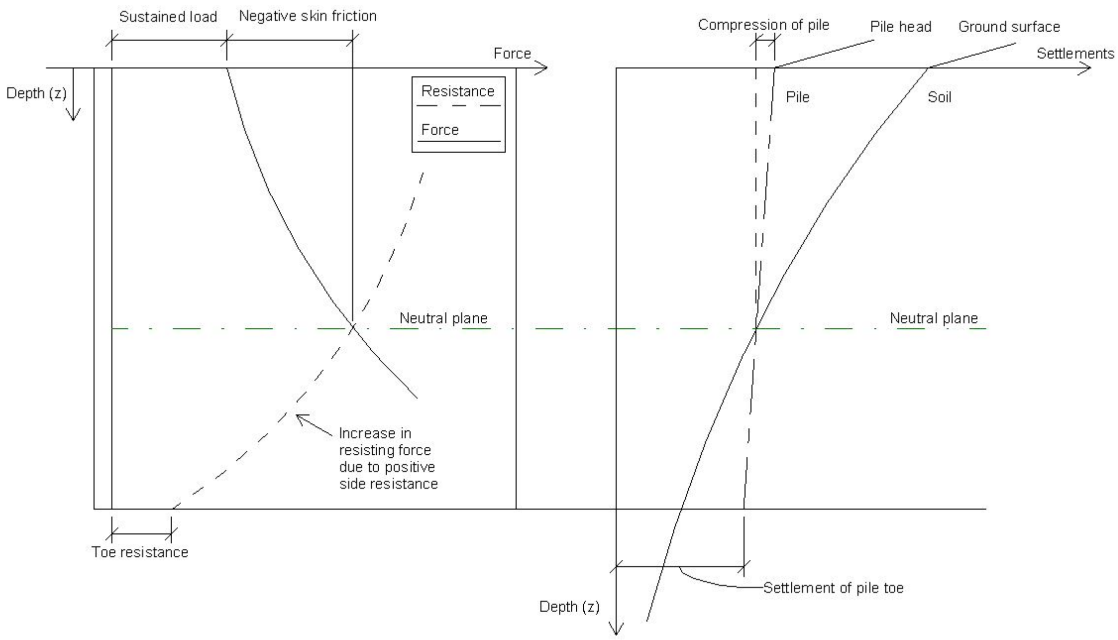

Figure 2.5: Left side: Illustration of the loads at the pile-soil interface to. Right side: Settlements.

2.4 Overlapping piles

In Mexico City an alternative type of pile foundation was introduced by Girault (1964). The reason was to combine the benefits of both end-bearing and cohesion piles, and to exclude some issues that can occur by using one method alone.

The goal was a more economical foundation for heavy structures constructed on soil undergoing subsidence (Girault 1964). The method can be described by considering two type of piles; pile type A which are ordinary floating piles and pile type B which can be considered as a form of end-bearing piles, see Figure 2.6. From now on these two pile types will be refered to as "A pile" and "B pile". The difference between a B pile and an ordinary end-bearing pile is that a B pile is cut at a level below the foundation slab. In other words, there is no structural connection between the B piles and the structure in the same way as there are no connection between the A piles and the bedrock (Girault 1964). Another way to view the method is to look at it as two systems of floating piles where one has the building foundation as its base and the other has been turned around with the bedrock or dense layer as its base.

The principle of this method is that the loads from the structure will be carried through the A piles and, by shearing of the soil, transfer a part of the load to the B piles while the rest is carried by the soil itself. Additionally some of the shear forces normally acting on an A pile in a cohesion pile system will in theory be carried by the B piles. If the B piles solely where to be installed, the settlements in the area would decrease in comparison to the the surrounding soil. This due to the fact that the soil is partially unloaded as the piles would induce what can be seen as upward forces when the soil is subsiding, known as *skin friction* which was introduced in Section 2.3. This counters the increase of vertical pressure, induced by the weight of the building. It is theoretically possible, with carefully planned positioning of A and B piles, to match the general subsidence in the area, and hence in a way compensate the construction to a certain level in terms of stress, and thereby settlements.

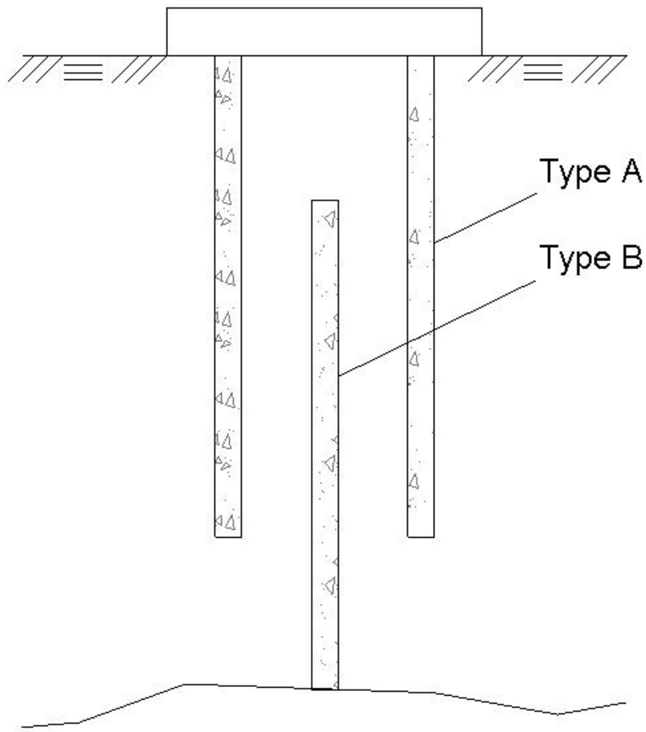

Figure 2.6: Overlapping piles.

Some of the drawbacks of using floating and end-bearing piles mentioned earlier will be avoided with overlapping piles. As long as the A and B piles are not in contact with the bedrock or the building foundation, respectively, the event of the building emerging will be avoided. At the same time the negative skin friction, acting on the A piles will be reduced. Girault (1964) states that the stresses in the soil when using an overlapping pile foundation can be described as illustrated in Figure 2.7. The Zero stress axis is the change of vertical stresses if no piles would be used, for comparison. The A piles would create an increase in vertical stresses if used alone. The B piles helps in unloading the soil hence a reduction in vertical stresses. The resultant, if both pile types is used, is a reduction in the overall increase of vertical stresses.

2.5 Installation of piles

Instead of categorizing piles on their functional behaviour they can be categorized by the method of installation. Two common installation techniques are driven and bored piles. Driven piles are installed by the use of machines, pile drivers, dropping a large hammer repeatedly at the top of the pile. When the pile is driven into the ground an equally large volume of soil will be displaced to make room for the pile. This will cause consolidation of the surrounding soil which will affect the soil properties (Prakash and Sharma 1990). According to De Mello (1969) the properties of the soil are changed due to remoulding and distortion of the soil. It also affects the stress state and the pore pressures in the nearby region.

Another consequence of driving piles is that it causes vibrations. These vibrations together with the

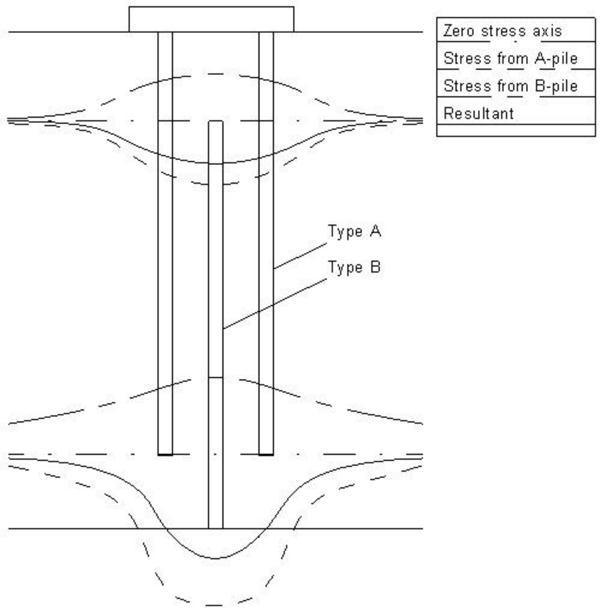

Figure 2.7: Soil stresses if using no piles, A and B piles separately or combined in an overlapping manner (Girault 1964).

displacement of soil could be harmful to nearby structures in already exploited areas.

The method of boring piles is to first remove the soil by drilling a hole in which the pile is later cast in place. This method will, contrary to the driving technique, not cause as large outwards displacements of the surrounding soil, and the installation will cause less vibrations and disturbance to the surroundings. The downside to the method compared to others is that it tends to be more expensive due to the longer time of installation.

When overlapping piles are being used as a foundation the main installation procedure can be described as following: The first step is to install the first B pile which causes a first disturbance in the soil adjacent to the pile. Also, in order for the pile not to be connected with the structure, the pile is either hammered down to the right level with a dolly or cut at the right level in an excavation. If a dolly is used this would leave an open hole between the ground surface and the pile. Either this hole is filled with soil or covered for safety reasons. After this the next B pile is installed in the same manner and this continues until all B piles are installed. The next step is the installation of the first A pile. This cohesion pile is installed in near proximity to the B pile in order to get the intended effects. However the soil surrounding the A pile may already have been disturbed due to the installation of B pile depending on distances between the piles. Depending on soil properties, orientation and geometries of the piles and the amount of disturbance the neutral plane of the A pile will vary. It is of great interest in knowing how the load is transferred from the A piles through the neutral axis into the B piles.

It is impossible to know the exact depth to firm bottom at the location of every pile. Piles are usually made in standard lengths varying between 6-13 m (Skanska 2016), and jointed to full length, which make it difficult to install the final part to an exact depth. When installing ordinary end-bearing piles this is solved by hammering down enough pile sections, until reaching firm bottom, and then cut them at the right foundation level. This is not as easily done when installing the B piles since these have to leave sufficient space between the pile head and the foundation.

2.6 Installation effects

Several studies have been conducted to measure the pile installation effects by for example De Mello (1969), Bozozuk et al. (1978), Randolph et al. (1979), Clark and Meyerhof (1972) and more recently Ottolini et al. (2014). Roughly the pile installation can be described in three steps namely installation, equalization and loading (Ottolini et al. 2014). As mentioned earlier the soil around a pile is disturbed when installing it by driving, step 1. Secondly the pore pressure origin from this pile driving will start to dissipate radially outwards, step 2, and finally the pile is loaded, step 3.

First of all, the driving pushes the soil away from the pile in both lateral and vertical direction. Due to this movement the soil is remoulded and the strength and deformation properties are changed. The state of stress of the soil within a distance of 2 times the pile diameter can be affected when installing the pile. De Mello (1969) and Bozozuk et al. (1978) showed that the in-situ shear strength in marine clay was reduced 30% due to pile driving. It has also been shown that no matter what overconsolidation ratio (OCR) the soil have before pile driving, the soil close to the pile surface will end up as normally consolidated (Randolph et al. 1979). The amount of soil that is disturbed by the installation is very complex to determine, but studies made suggest that the border between elastic and plastic deformations is approximately 2 times the pile radius (Clark and Meyerhof 1972). How the displaced soil affects the adjacent environment in terms of heave, displacements and increased stresses are mainly depending on the soil type, stress state and geometrical conditions (Hintze et al. 1997).

Another important fact during pile driving is the change in excess pore pressures. In areas close to the pile the excess pore pressures can reach values up to two times the in-situ vertical effective stress for some types of clay (Poulos and Davis Hughesdon 1980). There are field observations that show in what way the pore pressure changes due to driving of piles. It is shown that pore pressures generated at the pile tip Δu_T are resulting from nearly spherical cavity expansion (Roy et al. 1981). Pore pressures at the shaft, Δu_S of the piles are of a smaller magnitude and is resulting from a cylindrical cavity expansion. The magnitude of pore pressure and the affected zone can be estimated by the following equations by (Vesic 1972).

$$\frac{\Delta u}{c_u} = 2\ln\left[\frac{r}{\rho}\left(\frac{E_u}{2(1+\upsilon)c_u}\right)^{1/2}\right] + 0.82\alpha_f \tag{2.6}$$

$$\frac{r_p}{r} = \left[\frac{E_u}{2(1+\upsilon)c_u}\right]^{1/2} \tag{2.7}$$

$$\frac{\Delta u}{c_u} = 4\ln\left[\frac{r}{\rho}\left(\frac{E_u}{2(1+\upsilon)c_u}\right)^{1/3}\right] + 0.94\alpha_f \tag{2.8}$$

$$\frac{r_p}{r} = \left[\frac{E_u}{2(1+\upsilon)c_u}\right]^{1/3} \tag{2.9}$$

Equation 2.6 and 2.7 are for the cylindrical case and 2.8 and 2.9 for the spherical case, where r is the radius of the cavity, ρ is the radius where Δu is calculated, E_u and c_u are the undrained Modulus and shear strength respectively. r_p is the radius of the plastic zone and α_f is a pore pressure parameter at failure stated by Henkel (1959). For normally consolidated Weald clay the value of α_f is about 0.67 (Bishop and Henkel 1900), which is typical for many clays (Muir Wood 1990). On the same Weald clay, with an overconsolidation ratio of 2, the α_f value is around 0 (Bishop and Henkel 1900). This could indicate that the value of α_f should be somewhere between 0 and 0.67 for an typical slightly overconsolidated Gothenburg clay. The study by Roy et al. (1981) shows that the distance, from the center of the pile to where the pore pressures are affected, is about 4-5 times the pile diameter. It was also stated that all of the excess pore pressures had dissipated after 600 days. Another study showed that in sensitive marine clay, piling caused en increase in excess pore pressures of about 40% which dissipated after about 8 months (Bozozuk et al. 1978).

Another effect of pile driving in solf soils is heave. Suggested by Hintze et al. (1997) is that an area included by a 45 degree angle from the pile tip might be exposed to heave. This means that in order to get reasonable results and excluding interference with boundaries in numerical modelling, the width of the model will have to be at least the length of the pile from the pile center. Heave will also affect previously driven piles which might call for post driving of end-bearing piles to refusal in order to keep them in contact with the firm layer.

Due to the fact that soil movement occurs in both vertical and lateral direction during pile installation it is clear that this will have an effect on already installed piles. In a publication by Poulos (1994), several documented measurements of movements on adjacent piles are compared with calculated theoretical values. One conclusion from this study was that the installation will tend to push other piles in an outward direction, but these effects, due to lateral soil movements, can be reduced by planning the installation order of the piles (Poulos 1994).

To be able to predict the installation effects has long been of great interest to the geotechnical engineer. Lambe (1967) proposed the Stress Path Method, which today can be seen as a fundamental knowledge in geotechnics. It is a simple way to illustrate how a stress state changes in a soil element, when the equilibrium is disturbed. The stress path method can be used to plot an historical change in this soil element as well as to predict how a future construction will affect the soil. Both pore water pressure, strain and strength of the soil are dependent on the stress path which makes the stress path method a powerful tool for the goetechnical engineer. Different stress paths can be plotted and Lambe (1967) presents three common stress paths namely the Total Stress Path (TSP), the Effective Stress Path (ESP) and one finally total stress minus the static pore pressure. Each of these can be illustrated in a p-q plot as mentioned in Section 2.1.

The stress path method was later considered not to be the best option when analysing deep foundations, but rather suitable for shallow foundations (Baligh 1985). The Strain Path Method was introduced as a way of analysing deep geotechnical problems and rather than estimating incremental stresses the incremental strains are considered (Baligh 1985). The deviatoric strain components can be used to illustrate strain paths instead of stress paths. Regardless of either stress or strain path method is implemented in an analysis, it will provide much information in an organized way.

2.7 Pile design

The design of piles is based on both ultimate limit state (ULS) as well as serviceability limit state (SLS). Designing piles for (ULS) means no failure of the pile material itself and no failure due to lacking of bearing capacity of the surrounding soil. In (SLS) the design means that the deformations are within reasonable limits and eventual cracks in the concrete structure are limited (Eriksson et al. 2004).

When designing a deep foundations, it is not only the piles that are important but the structure overlying the piles have large influence on the design. This structure can be considered as weak, stiff or something in between and should be designed according to this. If the structure is weak it will not be able to handle differential settlements as a stiff structure would do (Eriksson et al. 2004).

A deep pile foundation consists of a number of piles and the behaviour of a single pile is different from a pile group. Pile groups will affect both the bearing capacity and the settlements. The bearing capacity could be considered as the combined capacity of all single piles, or calculated with regards to group effects (Eriksson et al. 2004). The main parameters determining if the pile will act as a single pile or as a pile in a group is the distance between piles, pile length and the ratio between them. One way of considering pile group effect is to treat the whole area of piles as one unit, transferring the load from a cross-sectional plane in an uniform way, for example by the use of Boussinesq´s equation (Karl et al. 1996). This is also praxis in Sweden according to Eriksson et al. (2004), but a good estimation can be done, and is often done so, by the use of the 2:1 method.

3 Regionens Hus

A new building is planned to be constructed in connection to the already existing Regionens Hus siuated in the center of Gothenburg. The construction of the new parts of Regionens Hus is to be built by Skanska for Västfastigheter by a decision of the regional council. The purpose of the new building is to collocate some of the Gothenburg based administration and management activities with the ambition of making the work more efficient in terms of office space, coordination and also for environmental advantages (Götalandsregionen 2015).

3.1 Surrounding area

The new part of Regionens Hus will be built in the central areas of Gothenburg, which demands some consideration to the surrounding constructions and infrastructure. The already existing building is a culturally protected, founded mainly on wooden piles which makes it sensitive to, for example alterations of the water table. Close by is also the city railway station, and in the near future, a lowering of the highway to the north of Regionens Hus is being investigated, as well as the construction of a new underground railway system. These amongst other are considerations that will influence the production and construction of Regionens Hus (Västfastighter 2015). However, the effects will not be considered in this thesis.

3.2 Ground conditions

The clay layers in the Gothenburg region are of both glacial and postglacial origin. The areas closest to the river Göta Älv have very deep clay layers. The exact depth is not known but the clay layer in the region of Regionens Hus is believed to reach a depth of 100 m at some locations (Wood 2014). There is also deposits of frictional material underneath the clay. There are some geotechnical investigations performed at Regionens Hus and the soil data is presented in Figure 3.1. Several CRS and triaxial tests have also been conducted for this project.

Site investigations shows that the soil consist of a man made fill to a depth of about 3-5 m followed by clay to a depth around 90 m. Below this is a stiffer friction layer of about 5-10 m before bedrock. The ground water level is situated within the range of 1.5-2.5 m below the ground level (Norconsult 2010). The fill is assumed to have been deposited in the beginning of the 19th century. The clay below the top layer is a silty clay with small amounts of shells at some depths. The unit weight of the clay varies between 1.53 - 1.83 kN/m^3 and the water content is about 60% and slightly increasing at deeper levels. The clay is considered as sensitive with values around 20. The data is summarized in Figure 3.1, where the top filling is not included. The permeability of the soil varies through the depth but in average a value of around $x \times 10^{-10} m/s$ is reasonable to assume (Wood 2014).

The area where Regionens Hus is to be constructed is experiencing some ongoing subsidence predicted to be around 2 mm/year. This is based on measurements in the nearby region (Wood 2014).

During 2012, eight B piles was installed as preliminary test piles in order to evaluate their behaviour. Piles of three different cross-sections were installed, namely square concrete piles with an area of 350×350, 400×400 and 450×450 mm. The largest pile could not be installed with ordinary piling

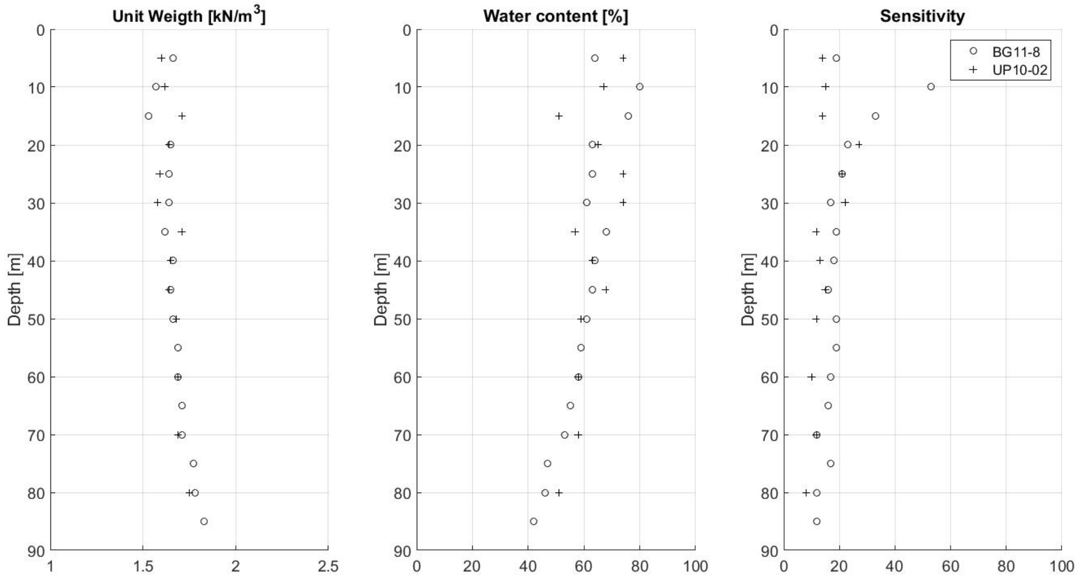

Figure 3.1: Soil data from site investigations in the nearby region from two different boreholes.

machines used in Sweden, so a larger machine was shipped to Gothenburg for this purpose, and another pile was damaged during installation. Since these piles were installed in the year of 2012 the effects from installation, such as excess pore pressures, would have dissipated. The planned solution with overlapping piles also includes A piles with a standard dimension of 270×270 mm. The exact layout of the piles can be seen in Appendix A.

4 Modelling

When modelling complex problems such as the pile foundation at Regionens Hus it is necessary to make simplifications. Although it would be possible to model the situation in full scale and with all contributing factors, such adjacent buildings and so on, the model would end up being too heavy with regard to computational capacity. In addition a full model would not necessarily be more accurate in reflecting the real behaviour. Common simplifications in order to save computational time is for example to study a small part that is considered to be representative for larger parts. Another can be to interpret a 3D problem into several 2D sections. To analyse the behaviour of overlapping piles the first step would be to study simplified 2D sections. The modelling of these sections in this thesis is made using the commercial software Plaxis 2D version 2015.2.

In order to capture the long term effects of the soil, a user defined soil model, Creep-SCLAY1S, is used in Plaxis. This soil model requires a lot of input data which are described in this Chapter. This data is later validated against CRS and triaxial tests as well as the ongoing settlements in the area.

4.1 Parameter derivation

The material model Creep-SCLAY1S is used for all clay layers and is described in detail in a following section. The top layer of fill is modelled using the Mohr-Coulomb soil model. The soil data is based on available information from the project Regionens Hus.

This model suits well for studying installation effects of piles and capturing long term effects such as creep, and has been tested with good results (Sivasithamparam et al. 2015). It does however require more model parameters than other widely used material models such as *Mohr Coloumb* or *Soft soil Creep*. The parameters needed for the Creep-SCLAY1S model are summarized in Table 4.1. The model is based on the *Modified Cam-Clay* but also incorporates parameters describing anisotropy in the soil, as well as effects of destructuration, creep and bonding. In the following sections the parameters will be briefly described.

The pre consolidation pressure σ'_c is the most important parameter, as it determines when the compression modulus of the soil will change drastically. This will in turn determine how large the settlements will be as a consequence of the applied load (Sällfors and Andréasson 1980). The value will be dependent of what the largest effective stress the soil has experienced since the deposition. In order to determine the pre consolidation pressure the CRS-tests performed on different depths are analysed. As proposed by Sällfors (1975) the lines of the straight parts of the CRS-curve are extrapolated to intersection in order to form a triangle. Afterwards at an equal distance x from the intersection point, a tangent to the curved part is placed. The point A where the tangent crosses the extrapolated line from the re-compression part will yield the pre consolidation pressure see Figure 4.1. This way of determining the pre consolidation pressure is the common way of doing it in Sweden. Furthermore the pre-overburden pressure (POP) and the over consolidation ratio (OCR) are achieved by calculating the current effective vertical stress state, σ'_v for each layer, seen in Appendix B.

The coefficient of earth pressure at rest K_0^{NC} and K_0 for normally and over consolidated soil respectively, are determined by Equation (4.1)a and Equation (4.1)b.

$$K_0^{NC} = 1 - \sin\phi'_{cs} \qquad K_0 = (1 - \sin\phi'_{cs})OCR^{\sin\phi'_{cs}} \tag{4.1}$$

Table 4.1: Parameters used in Creep-SCLAY1S

Parameter type	Parameters	Symbol
Isotropic	Modified swelling index	κ^*
	Intrinsic compression index	λ_i^*
	Poission's ratio	ν'
	Friction angle	ϕ'_{cs}
	Stress ratio of critical state in compression	M_c
	Stress ratio of critical state in extension	M_e
Anisotropic	Initial inclination of yield surface	α_0
	Absolute effectiveness in rotational hardening	ω
	Relative effectiveness in rotational hardening	ω_d
Destructuration	Initial bonding	χ_0
	Absolute rate of degradation	ξ
	Relative rate of degradation	ξ_d
Viscous	Intrinsic creep coefficient	μ_i^*
	Reference time	τ_d
Initial stress	Buoyant unit weight	γ'
	Effective pre-consolidation pressure	σ'_c
	Lateral earth pressure at rest (normally consolidated soil)	K_0^{NC}
	Pre-overburden pressure	POP
	Over-consolidation ratio	OCR
	Initial void ratio	e_0

where the friction angle at critical state, ϕ'_{cs}, is determined from triaxial compression tests.

The isotropic parameters used in Creep-SCLAY1S are similar to ones used in the Modified Cam-Clay. For loading above σ'_c the parameter λ_i^* (modified intrinsic compression index) is used in the relation between mean effective stress and void ratio. In situations of loading under σ'_c the parameter κ^* (modified swelling index) is used instead. These two parameters can be derived by studying incremental loading oedometer tests with at least one unload/reload cycle. As no incremental loading tests are available for the project Regionens Hus, the values are instead determined by using the *soil test facility* in Plaxis and optimize the values in order to match the curves for the CRS- and triaxial data available, see Section 4.2.

Creep-SCLAY1S also uses two critical state lines as ultimate limits. The critical state lines M_c and M_e can be determined from triaxial tests in compression and extension, respectively. For the project of Regionens Hus no triaxial extension tests were conducted. Hence, M_e will be derived from Equation (4.2)a and M_c from Equation (4.2)b (Muir Wood 1990). The lines are are illustrated in Figure 4.2 together with the normal-consolidation line.

$$M_e = \frac{6 \sin \phi'_{cs}}{3 + \sin \phi'_{cs}} \qquad M_c = \frac{6 \sin \phi'_{cs}}{3 - \sin \phi'_{cs}} \tag{4.2}$$

Figure 4.2 also shows the different surfaces in the Creep-SCLAY1S model. The innermost surface related to p'_{mi} is the intrinsic yield surface. The surface related to p'_{eq} is the current state surface (CSS) and the last surface is the normally consolidated surface (NCS) related to p'_p.

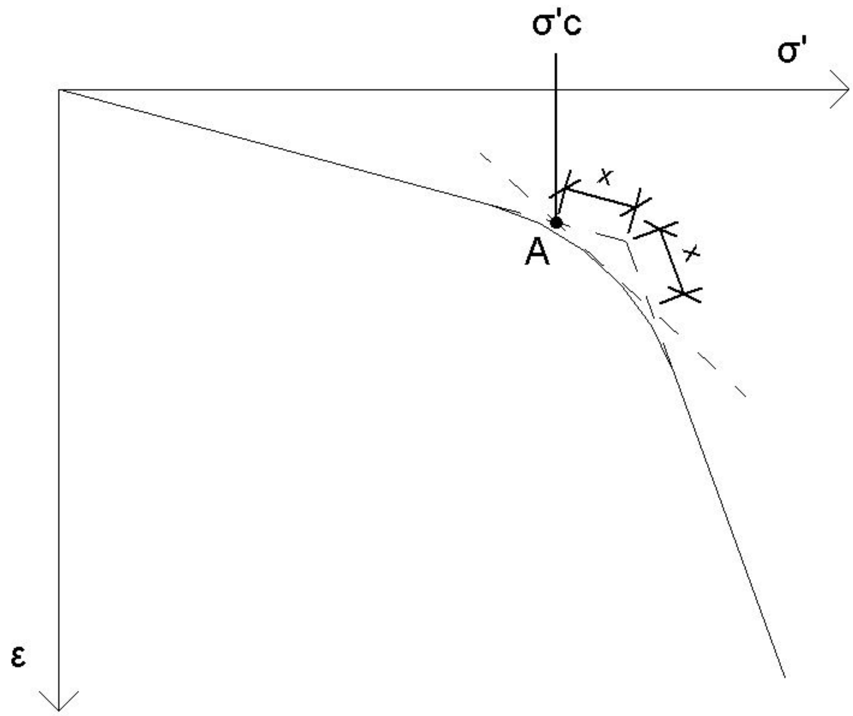

Figure 4.1: Determination of pre-consolidation pressure from CRS-curve.

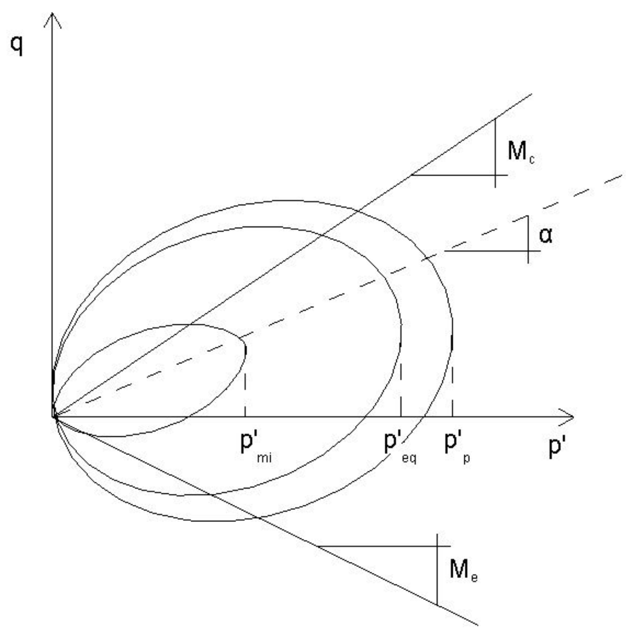

Figure 4.2: Parameters defined in the p'-q plane used in Creep-SCLAY1S.

In order to reflect the anisotropy of the soil, the Creep-SCLAY1S model introduces a couple of parameters. The parameter α_0 is used to describe the initial anisotropy of the soil, while ω and ω_d defines the rate of rotation of failure surfaces and relative rate of surface rotation respectively (Wheeler et al. 2003). The parameters are derived from Equations 4.3 and 4.4.

$$\alpha_0 = \frac{\eta_{K_0}^2 + 3\eta_{K_0} - M_c^2}{3} \qquad \text{where} \qquad \eta_{K_0} = \frac{3(1 - K_0^{NC})}{(1 + 2K_0^{NC})} = \frac{q^{NC}}{p'^{NC}} \tag{4.3}$$

$$\omega_d = \frac{3}{8} * \frac{4M_c^2 - 4\eta_{K_0}^2 - 3\eta_{K_0}}{\eta_{K_0}^2 - M_c^2 + 2\eta_{K_0}} \qquad \omega = \frac{1}{\lambda^*} ln \frac{10M_c^2 - 2\alpha_0\omega_d}{M_c^2 - 2\alpha_0\omega_d} \tag{4.4}$$

As stated by Sivasithamparam et al. (2015), the derivation of ω is based on numerous assumptions which sometimes yields unreasonable results. Instead, the value can estimated in the empirical range:

$$\frac{10}{\lambda^*} < \omega < \frac{20}{\lambda^*}$$

The creep index μ_i^* is determined by first capturing μ^* for different load magnitudes, using Equation (4.5). Afterwards these are plotted against the stress level normalised by the in-situ vertical effective stress and checked for consistency. The method for determining μ^* is shown in Figure 4.3.

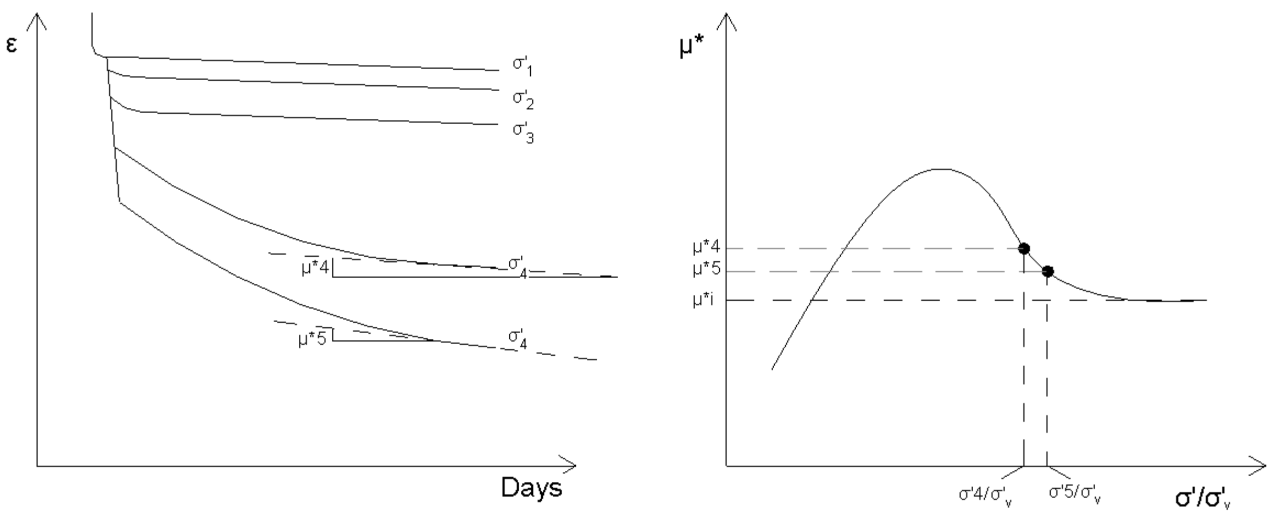

Figure 4.3: Determination of μ_i^*, ideal case.

Comparing Figure 4.3 with Figure 4.4 shows that the desired stress level seems to not have been reached. The creep-tests will give a reasonable range in where the true value of μ_i^* lies, see Figure 4.4. The value of μ_i^* is, however, later validated by matching ongoing settlements.

$$\mu^* = \frac{\Delta\epsilon}{\Delta ln(t)} \tag{4.5}$$

The Creep-SCLAY1S model includes parameters for bonding and destructuration. The amount of bonding between the particles is described by a value χ which can be estimated using the sensitivity of the soil by Equation (4.6). The variable χ_0 is linking the size and position of the intrinsic yield surface and the NCS.

$$\chi_0 = S_T - 1 \tag{4.6}$$

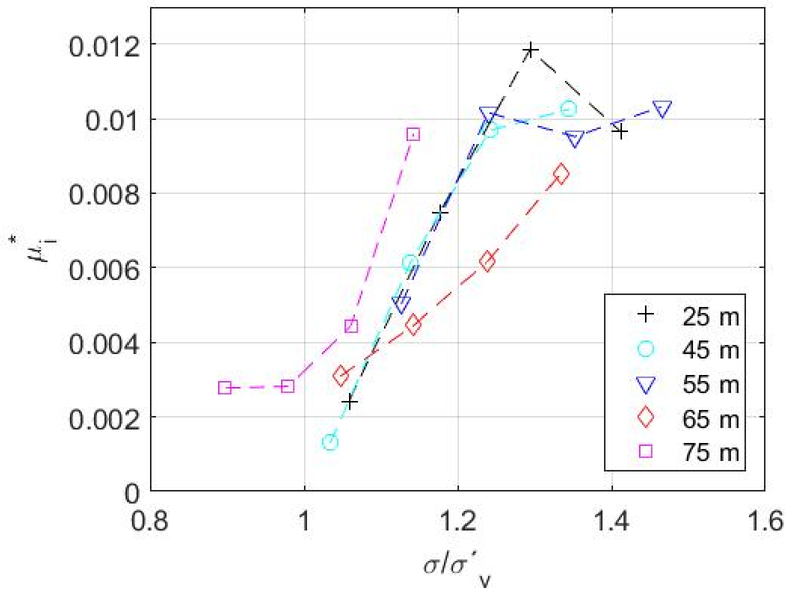

Figure 4.4: The variation of μ_i^* at different depths.

The destructuration, or the degradation of the bonds, between the particles due to volumetric strain and deviatoric strain are described by (ξ_v) and (ξ_d) respectively. These parameters can be determined by dedicated tests but as such have not been conducted for the project Regionens Hus the parameters will be taken from default values (Yin and Karstunen 2011), as follows: $\xi_d = 0,2 - 0,3$ and $\xi_v = 8 - 12$

State variables are describing the current state of the soil model and after each new calculation step in Plaxis these are updated. The values of these variables describes different changes in the model. Some state variables of interest in this thesis are the variables describing the anisotropic vector α_x, α_y and α_z, the shear component $\alpha_x y$, $\alpha_x z$ and $\alpha_y z$ as well as χ_0 which describes the amount of bonding. Either OCR or POP are used for describing the load history of the soil through the determination of σ'_c. The Creep-SCLAY1S model is a 3D model and when implemented to Plaxis 2D, these variables are helpful when describing stress/strain paths for the soil.

4.2 Parameter evaluation

The soil profile is divided into three layers, two layers of clay and one layer of fill material. The clay is divided with respect to the friction angle, water content and sensitivity, which are shown to differ at depths below 50 m. The soil model will have to be calibrated to real measurements and tests in order to validate the predicted model parameters. The measurements include triaxial compression tests, CRS-curves and the ongoing settlements in the area. The two layers of clay will be referred to as the upper and the lower layer. All soils parameters for the two clay layers after the parameters have been validated are shown in Appendix C. There might still be some uncertainties concerning the lower clay layer since triaxial tests are only available to a depth of 50 m.

Initially, and all the way up to the pre-consolidation pressure, the modified swelling index κ^* is governing the behaviour of the Plaxis Soil Test curve. As can be seen in Figure 4.5 and Figure 4.6

the curves from Plaxis is initially behaving a bit different from the test curves but other than that it is capturing the curve. This is partially due to the fact that κ^* is dependent on stresses and in order to not get any numerical difficulties in Plaxis a small amount of stress is prescribed to the Soil Test.

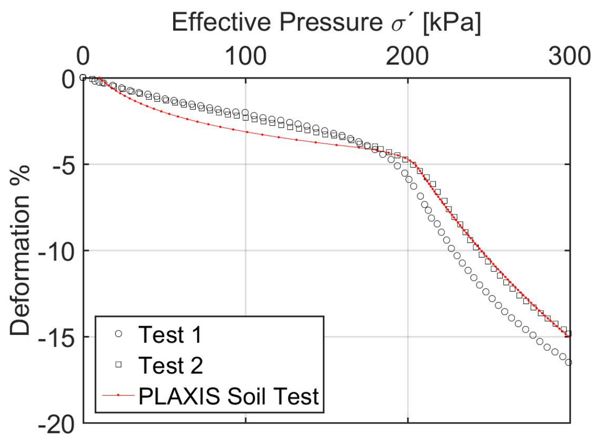

Figure 4.5: CRS matching at 20 m depth.

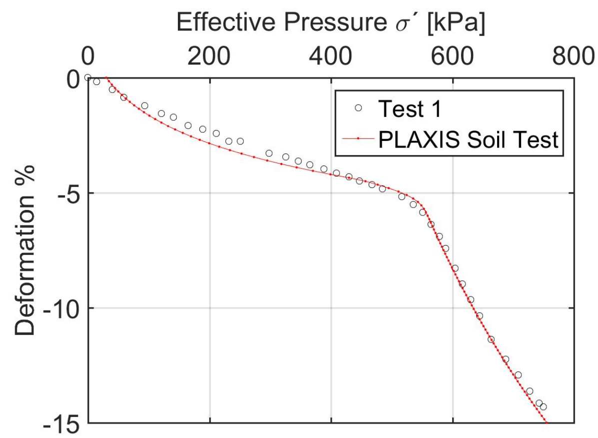

Figure 4.6: CRS matching at 55 m depth.

In Figure 4.7 to Figure 4.10 triaxial test values are compared to values from Plaxis soil test. The tests captures the behaviour in the p'-q plane quite well. However in the stress-strain plots the behaviour is only matching the behaviour up to 2% strain.

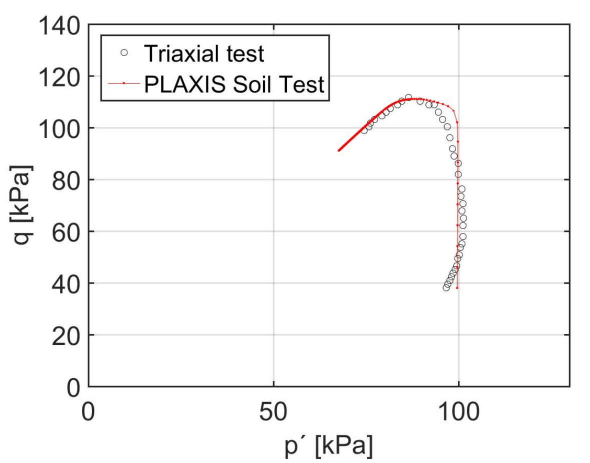

Figure 4.7: Triaxial matching at 20 m depth.

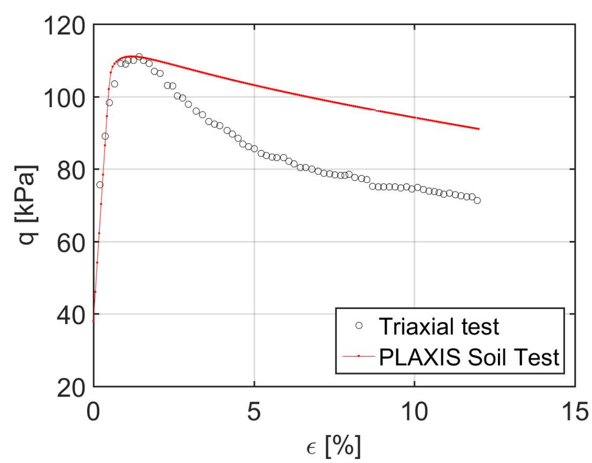

Figure 4.8: Stress-strain match at 20 m depth.

The ongoing settlements in the area are to be around 2 mm (Wood 2014). There are other investigations in the nearby region where the settlement rates have been estimated to be as high as 4-6 [mm/year] (Sweco 2014). In order to validate the soil model a simple Plaxis model is created to match the ongoing settlements. The model consists of the fill layer and the two clay layers mentioned earlier together with a total depth of 90 m. The total width is 240 m with top and bottom boundaries open in terms of water flow. The model is left to consolidate for 200 years and the result is a settlement rate of around 5 [mm/year] at the top of the clay. Based on this, the assumed creep rate appear to be reasonable.

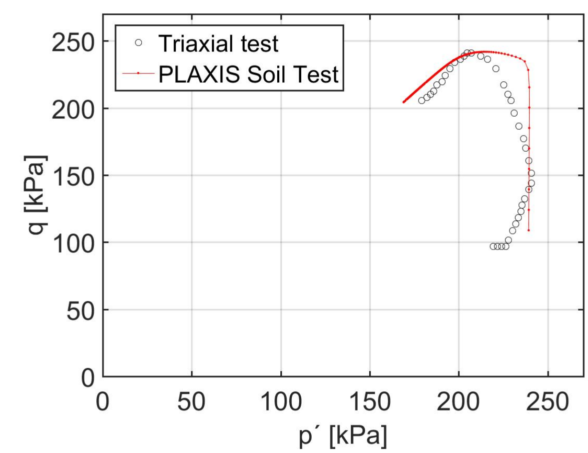

Figure 4.9: Triaxial matching at 50 m depth.

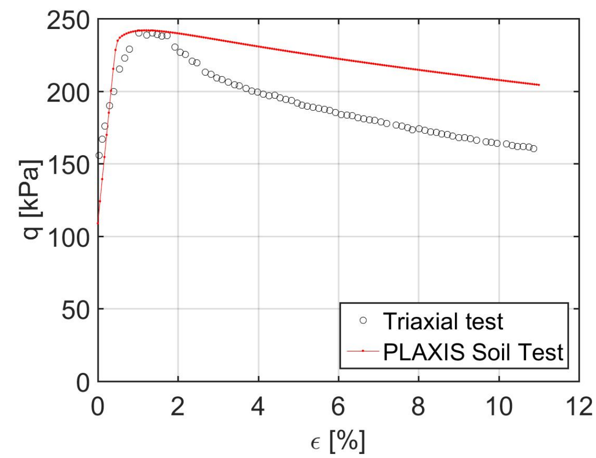

Figure 4.10: Stress-strain match at 50 m depth.

4.3 Modelling piles in Plaxis

Plaxis offers a couple of techniques for modelling piles, which individually have their own advantages and drawbacks. The following ways of modelling are available in the used version of Plaxis:

- Embedded pile row
- Plates
- Node to node anchors
- Volume elements

In order to choose a suitable method for overlapping piles the behaviour of each solution first has to be examined. Also the interpretation of a 3D situation in a 2D model will have an effect on the decision.

Plaxis offers the user a choice of either using 6- or 15-noded elements. Depending on the problem and how it is going to be modelled, the two different ways will be more or less suitable, examples of which can be seen in Table 4.2. In the case of serviceability limit state and deformation problems the

6-noded	**15-noded**
Plane strain models	Axi-symmetric models
SLS	ULS
Faster computation	Slower computation

Table 4.2: Suitabillity of 6-and 15-noded elements

simpler 6-noded element will give quite accurate results, which in addition to the shorter computation time can exceed the performance of the more precise 15-noded elements in some cases. In more complex problems or in situations where failure in the soil might occur, the 15-noded option however exceds the 6-noded as the later might over-predict for example safety factors in some cases which is not desirable (Plaxis 2011).

In Plaxis a problem can be modeled as either axisymmetric or plane strain. In the axisymmetric case the geometry modeled is symmetric around the y-axis. In plane strain, the geometry is modelled in just one cross-section which leads the program to interpretate the problem as can be seen in Figure 4.11. As only one cross-section can be captured in a plane strain model by Plaxis 2D, the center to center distance between the piles in the out of plane direction will, to a great part, govern which method that is the most suitable in capturing the real behaviour. Depending on the distance between the piles, the pile behaviour can be divided into three different groups: Wall behaviour, Pile row behaviour and Single pile behaviour. In the case of having the piles next to each other the piles will behave as a wall. Increasing the distance between the piles will, to a certain point, keep the wall like behaviour as the soil may arch between them. Increasing it further will let the soil "flow" between the piles but still be affected by them and hence make the behaviour more like a pile row. Increasing the spacing further will make the piles less dependent from each other and causing them to behave as a single piles (Plaxis 2016b). Further on Plaxis (2016b) suggests uses for the different techniques depending on what behaviour that is at hand according to Table 4.3.

Table 4.3: Suitability of modelling piles in 2D

Approach	Wall behaviour	Pile row behaviour	Single pile behaviour
Volume elements	A	C	-
Plate	A	C	-
Node to node anchor	-	C	-
Embedded pile row	-	B	C

A: Good approximation B: Reasonable approximation
C: First order (crude) approximation -: Not feasable/recommended

Modelling a pile as a volume element is done by simply creating the dimensions of the pile section and assigning it with material parameters. Another way of modelling a pile is by the use of plate elements. If a plate is considered, Plaxis requires input values of the stiffness. The needed values are the area, Young's modulus and the moment of inertia entered in the forms EA and EI. Young's modulus is known from pile material and the area and moment of inertia is based on the cross section of the pile. This means that the plate, even though it is modelled in Plaxis as a thin line, will have a width b.

One of the drawbacks of these two options is that the model will consider the pile as a continuous wall as shown in Figure 4.11. For piles this can yield result that are not reflecting the natural behaviour for all load cases. The direct opposite of this would be node to node anchors which have no interaction with the soil and lets the soil flow through. These do not however have any properties in the lateral direction which also is not the actual case with piles.

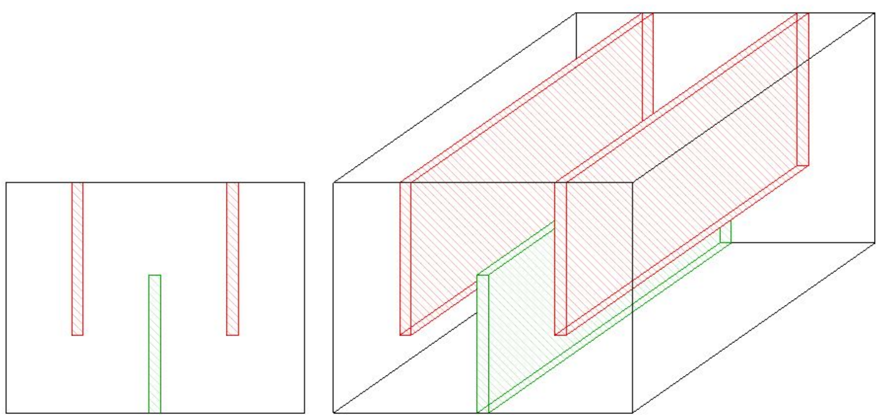

Figure 4.11: 2D Plane strain interpretation of a 3D problem by the use of volume or plate element piles.

As previously stated, if piles are modelled by the use of volume elements or plates in plane strain a wall behaviour is achieved which in total will have an unreasonable stiffness compared to the real situation. In order to achieve the true pile behaviour in a plane strain model, the pile properties can be altered. As a reference to how a single pile would behave, an axisymmtrical model is created. The axisymmetrical model includes one volume element pile with the true dimensions and properties of the piles to be installed. The same behaviour in plane strain can be approximated by calculating a more suitable cross-sectional area and when the width of the pile is set to 2 cm, a more realistic stiffness behaviour is observed. The comparison between the axisymetrical model and the plane strain model is shown in Figure 4.12. The same behaviour is observed when the piles are modelled as plates and in order to get more accurate results the behaviour should be controlled for different widths.

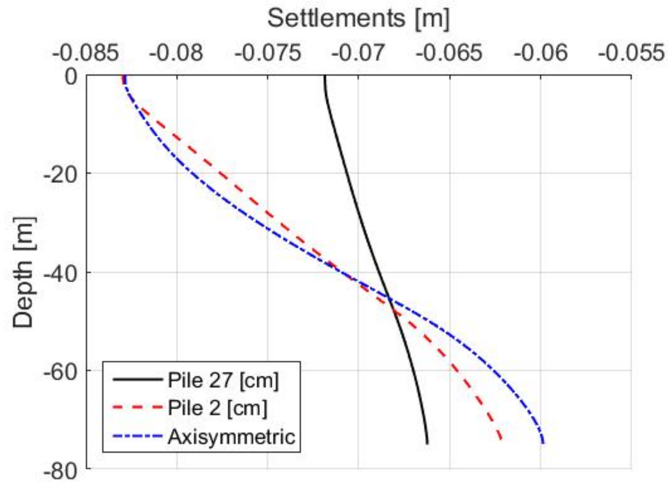

Figure 4.12: Total settlements dependent on the way the pile is modelled. The figure shows the behaviour of two piles with different widths in plane strain and one pile in axisymmetry.

As can be seen in Figure 4.12 the pile modelled with the real width in plane strain is too stiff. However with a width of only 2 cm, the pile in a model with a total width of 240 m would create problems with the mesh. An alternative would be to model the pile as a plate element with a fictitious width not affecting the mesh. The drawback of modelling piles in this way would be that installation effects, such as soil consolidation in lateral direction will not be captured. Additionally the wall behaviour would still be apparent to some extent.

Another option is to model piles by the use of Embedded pile rows. This option allows for control of the shaft and tip bearing capacity as well as a center to center spacing parameter in the out of plane direction. This also allows for the soil to flow through the row, contrary to the plate and volume elements. The embedded pile row does however have some limitations, for example the consolidation of the soil can not be captured. One advantage of embedded pile rows is that it is possible to arrange A and B piles with different spacing in the out of plane direction. However, although this method seems to be the most suitable for modelling piles, it can not be used for a pile behaviour analysis, given the embedded pile rows require input values of the soil-pile interface to determine their bearing capacity. This study aims to examine how different pile configurations behave and how the B piles affect the system. If values of the soil-pile interface is entered by the user the behaviour and results would to some extent be effected.

Interfaces are used in Plaxis in order to disconnect the soil from the modelled structure, to some extent and can be applied to either volume elements or plates. In this way the simulation is able to capture some relative displacements between the two materials which otherwise would have been strictly bound together until failure. The interface elements does this by assigning the soil and the structure one node each of a coupled pair instead of having them share one. By doing so the model can capture the gap between the soil and structure interface as well as slip displacements (Plaxis 2016a). The use of interface does, similar to embedded pile rows, require the user to manually define soil-pile interface properties. This means that interfaces can not be used in this study.

4.4 Modelling installation effects

Installation effects do have a large influence during and some times after the installation but how much do they affect the performance of a foundation over a one hundred year span? It is of interest to examine how different effects (stresses, pore pressures, displacements) affect the final result. Some ways of capturing installation effects in Plaxis are examined, and a comparison between models is carried out. Both models with different installation effects, as well as simple wished in place pile models with no consideration to installation effects, are compared.

As a pile is installed in soil some both horizontal and vertical consolidation and displacements occur. This can be modelled by either cavity or volume expansion. Cavity expansion is done by creating a small void in the soil model, expanding it and inserting a pile element. Some finite element programs offers the possibility to insert new elements after a calculation step, but Plaxis is not one of those. The other option, volume expansion, is done in a similar manner but instead of a void, a small pile element is used. Volume expansion can be used to generate an increase in pore pressures, stress increase and displacements in the adjacent soil, much like when installing a pile. In Plaxis this is done with a wished in place pile, created with volume elements, with a smaller width than planned and then increasing the volume. This is done simply by prescribing an increase of ϵ_{xx} so that the pile will end up at its true width. However, this becomes problematic when the model (as in this case) is large, and the piles are modelled to just a fraction of the model width. The pile mesh, when using volume expansion, is just extended and no extra elements are created. Since the pile mesh is just two elements wide, when using the finest mesh settings, some instability issues occurs in addition to the low accuracy. If only the installation effects were to be studied the pile could be modelled with a greater number of elements in a smaller model. Another issue that occurs when using volume expansion in plane strain is referred back to the fact that Plaxis interpretates the volume elements an infinite wall in the out of plane direction. When expanding this wall a much larger amount of soil is being displaced hence causing an unrealistic deformation and stress state of the soil in the model. This behaviour is presented for two volume element thickness's in Figure 4.13.

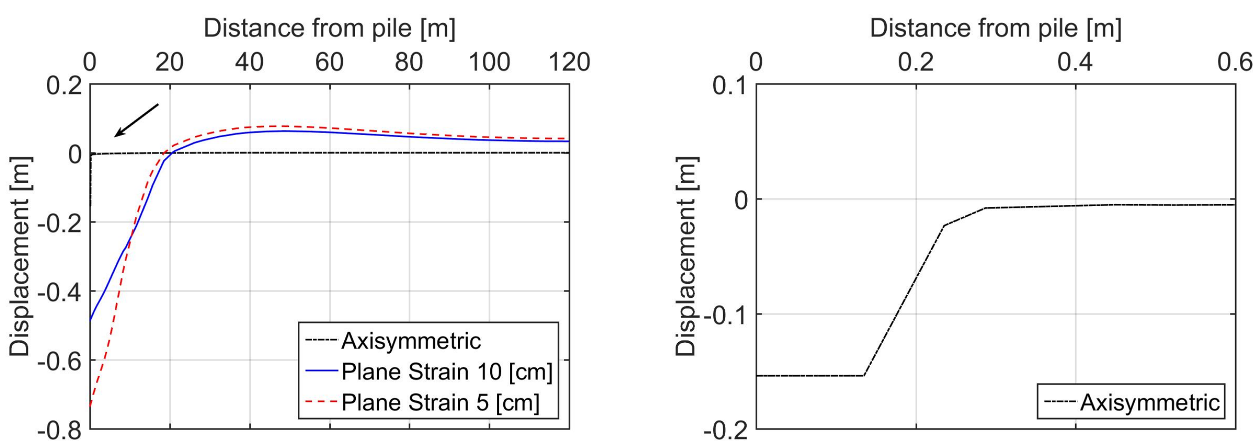

Figure 4.13: Lateral displacements from volume expansion at 40 m depth when ϵ_{xx} is increased 10%. The left side shows both an axisymmetric model and two plane strain models and the right side shows the initial part (see arrow) from the axisymmetric model.

The axisymmetric model matches well with predictions using equation Equation (2.6) and real case studies based on measurements on single piles. But no matter how small the volume elements are, the wall behaviour is always present.

The second way of capturing some of the installation effects is by shearing the pile to failure in the soil. This is done by applying a prescribed displacement along the pile. In the same manner as the volume expansion, the plane strain analyses of shearing behaves quite differently compared to the axisymmetrical response. In plane strain the deformations spread to a larger distance away from the pile. This is illustrated in Figure 4.14, where it is obvious that the displacements of the soil are unreasonably large at a far distance from the pile.

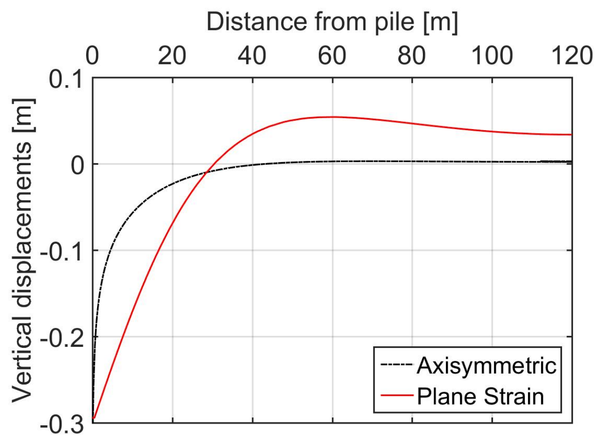

Figure 4.14: Vertical displacements from pile shearing at 40 m depth. Left side shows both an axisymmetric model and a plane strain model and the right side shows the initial part (see arrow) from the axisymmetric model.

One final, more simple, way of considering the installation effects is to think about what is happening to the soil during pile driving. As mentioned in Section 2.6, a zone of disturbance can be distinguished. A simple way of capturing parts of this disturbance is to replace the soil close to the piles with the same soil but with an OCR value of 1. However, parts of the installation effects will not be captured by the use of this method, such as lateral and vertical displacements of the soil and an increase of pore pressures close to the pile.

4.5 Conclusion

Volume expansion will not be considered further due to the problems described earlier. The other ways of modelling the installation will affect the system differently which is shown in Figure 4.15. It is clear that both soil replacement and the shearing have an effect on the pile settlements.

As Figure 4.15 shows, the settlements of the A pile head will reduce when shear is used. When replacing the soil with OCR = 1 in an area around the pile this will yield larger settlements, compared to a pile with no installations effects considered. The amount of soil that is replaced in the OCR models is based on previously conducted studies. The difference between the OCR model and the sheared model is that in the shear model the adjacent soil is not only disturbed but excess pore pressure is also generated in

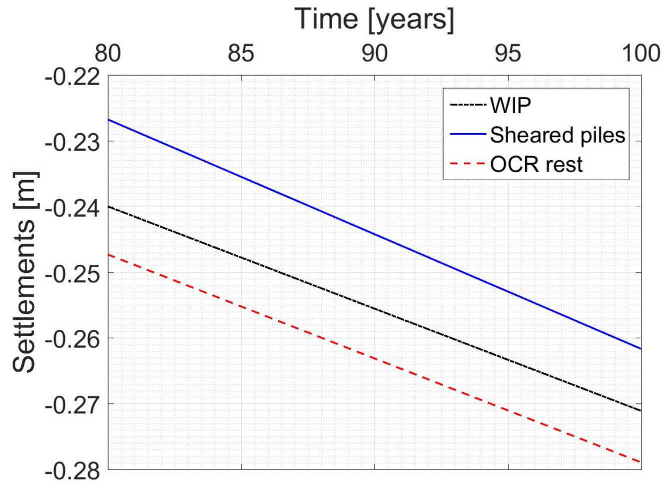

Figure 4.15: Settlements at the pile head of a single A pile.

the adjacent soil, along the pile shaft and the pile tip. With this varying behaviour, between the sheared models and the OCR models, they are both of interest to study and compare.

An axisymmetric model will be impossible to use when modelling more then a single pile which leaves plane strain as the only option. The following pile modelling options in Plaxis are excluded:

• Node-to-node anchors	Not suitable because the pile behaviour along its length can not be studied.
• Embedded pile rows	Requires bearing capacity as an input rather than being provided as an output.
• Volume elements	Mesh instability in the piles due to model dimensions.

This leaves the plate elements as the most suitable option, even though the wall behaviour is still present

4.6 Overlapping piles analysis

In order to examine the behaviour of overlapping piles, their limitations and effectiveness a few models will be created in Plaxis. To achieve this, the models are varied in terms of changing for example the overlapping length or the distance between the piles. Since there are no long term measurements of overlapping piles, a first step in the analysis is to create very simple models with no installation effects included, to get some values for comparison.

All models created have the same basic properties; the total width of the models are 240 m and the soil depth is 90 m divided into a layer of fill and the two clay layers mentioned earlier. The reason for having such a wide model is to make sure that all effects can be captured without boundary interference when full pile length of 90 m is used. The way of checking if the model is wide enough is done by looking at the directions of the principal stresses at the outermost nodes. All models are plane strain models using 6-node elements. The deformation boundary conditions for all models are the same which can be seen together with the dimensions of the model in Figure 4.16. The groundwater flow is closed at the edge borders but open in the top and the bottom of the model, i.e. double drained conditions. Initially

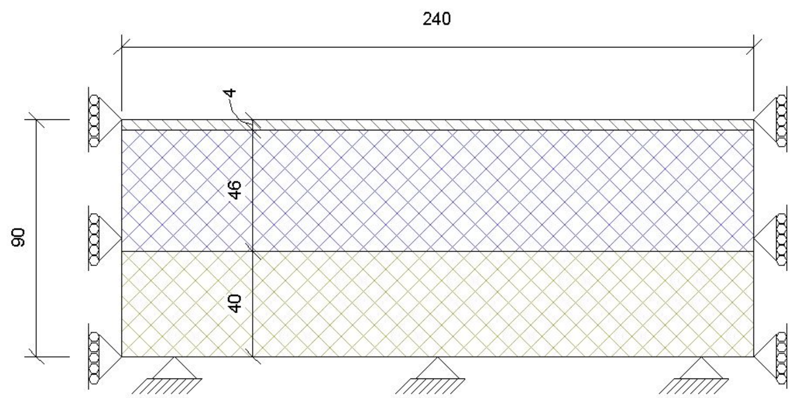

Figure 4.16: The basic concept for the different pile models, measured in m, with boundary conditions.

the fill is placed and left to settle for 200 years to simulate actual placement of the masses during 19th century. After this the fill layer is replaced with an uniformly distributed load matching the load coming from the weight of the fill. The reason for removing the fill is that eventual interference between the upper soil layer and the fill will be avoided.

4.7 Pre-evaluation of the overlapping system

In order to validate that the model, and the principle of overlapping piles is working as expected, a preliminary study is conducted by creating three simplified models. In Figure 4.17 these three models are illustrated. In the first model (A) two A piles with a cc distance of 7 m between them are modelled. Both piles are wished in place and later left to settle for 100 years. The second model (B) consists of a single B pile, wished in place in the same manner as in the first model, also left to settle for 100 years. The final model (C), using wished in place elements, is made with two A piles and one B pile each placed with a cc distance of 3,5 m. This model is also left to settle for 100 years. The concrete plate is modelled as a infinite stiff plate without any density in order to structurally connect the A piles to get a realistic behaviour.

From all these models, the settlements behaviour is analysed for various depth over the total width of the model in order to see how the B piles affect the system. This behaviour is illustrated for the three systems mentioned above as well as pure soil, at a depth of 80 m, in Figure 4.18. As expected the B pile seems to unload the A piles in a satisfactory way. Also notable is the magnitude of displacements and progression in horizontal direction which exceeds what would be expected in a real situation.

The next step is to take model C in Figure 4.18 and gradually increasing the complexity by adding installation effects to the model. The results, in forms of stress paths, from this analysis is presented in Section 5.1.

In the previous sections, various ways of modelling piles, pile installation effects as well as different elements has been presented. The final analysis with different pile set-ups will all be modelled in plane strain, 6-noded elements using plates to simulate the piles and shearing technique or soil replacement to

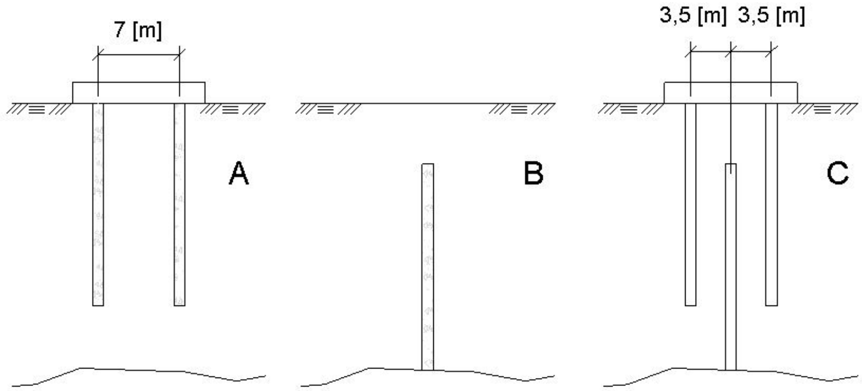

Figure 4.17: Three basic pile models using wished in place elements.

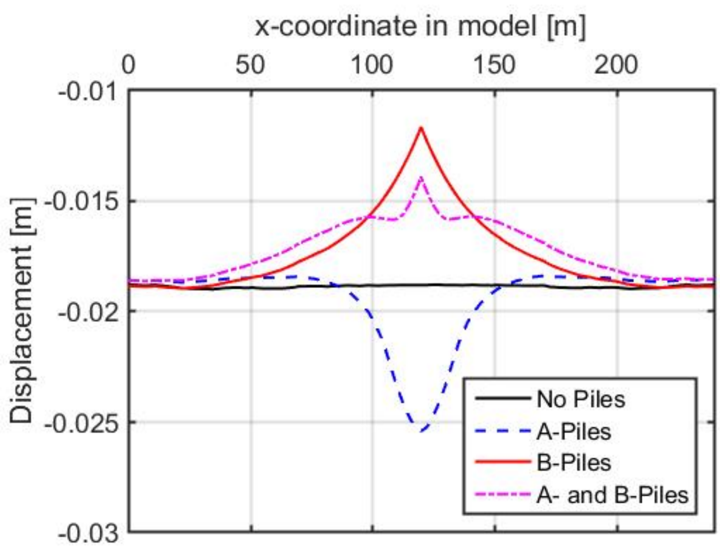

Figure 4.18: Soil settlement curves for the three systems A,B,C at 80 m depth.

capture some of the installation effects.

As mentioned earlier a plate element requires the user to enter the Area stiffness EA and the bending stiffness EI. The area A in the expression is the same as the width b since Plaxis treats the element per meter in the in-plane direction. It was discovered, when applying shear as a way of capturing the installation effects, that the stiffness of a pile with a fictitious width of $b = 0,035$ m is too small. If this small stiffness is used the pile is compressed elastically and after the shear stage the pile will elongate and cause unrealistic amount of heave. Instead the piles are modelled with their actual width to reduce this effect, keeping in mind that the pile behaviour is too stiff compared to an axisymmetric pile. The following stiffness values are used for the plate elements; $E_A = 30 * 10^6$ kPa, $E_B = 30 * 10^6$ kPa for

A- and B-piles, respectively, based upon a width of $b = 0,270$ m. It is of interest to examine how the foundation, as a whole system, behaves when different set-ups of the piles are used. In Figure 4.19 the set-up variables are illustrated. In addition to the values presented in Figure 4.19 the number of piles and the total length of the piles are of interest. The pile set up variables are explained in Table 4.4.

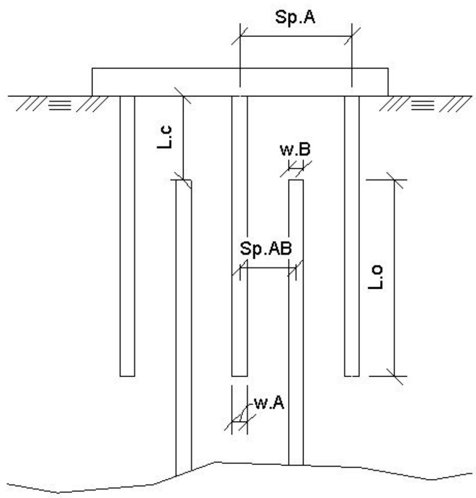

Figure 4.19: Variables describing the possible set-ups.

Table 4.4: Variables used to model different set ups

Pile set up variables	
n_A and n_B	Number of A and B piles respectively
Sp_A	Spacing between the A piles
Sp_{AB}	Spacing between A and B piles
L_O	Length of overlap between A and B piles
L_C	Distance between B pile head and foundation
L_A and L_B	Length of A and B piles respectively
L_{tot}	Total length of piles in the system
w_A and w_B	Width of A and B piles respectively

4.8 Modelling phases in Plaxis

The way of modelling can be divided into four different main stages. First of all comes the placement of the fill material followed by a consolidation stage of 200 years in order to capture the current soil state. Following is the installation of the piles. When this is done the actual construction is built and finally the model is left to settle for 100 years. In Plaxis these 4 stages are divided and built up and can be seen in Appendix D where both the structure of the shear and OCR models are shown. The difference between the two modelling types is that they uses two separate ways of capturing installation effects. In the first the pile is sheared and in the second the soil around the piles are replaced with disturbed soil. In the latter the soil in the model is replaced in an area equivalent to 3 times the pile diameter around the piles. These models are needed to study forces in the actual piles since the shearing of the piles will cause tensile forces to arise in the pile as the adjacent soil wants to return to the initial state due to elasticity. Theses tensile forces are unrealistic and would produce false results when looking at the piles.

Another difference between the sheared pile model and the OCR reset model is that the soil is allowed to heal for 90 days in the shear model. This is done as the piles otherwise will go back to their pre-shear position due to soil elasticity, which is also the reason for the tensile forces in the pile. By leaving 90 days for pore pressure to dissipate this effect is lowered by about 70%.

The piles are installed in separate phases in order to simulate the actual installation order. For example the first A pile is installed and sheared before installation of the next pile starts. The total load that is applied on the plate on top of the piles is always equivalent to 1500 kN.

In the models where the OCR is reset, the area of influence is based on previous studies mentioned in Section 2.6. These studies showed that the disturbed area was approximately 2 times the pile diameter and similar results was obtained when modelling a single pile in axisymmetry where the disturbed zone was about 3 times the pile diameter.

5 Results

The results in this chapter are based on several models that differs in spacing Sp_A, overlapping length L_o and how the pile installation is modelled. The outcome are presented as axial forces, settlements and soil response.

5.1 Stress path interpretation

Six points have been selected for a stress path study, as seen in Figure 5.1. Point A is selected far away from the pile, point B is located above the NP close to the pile, C-E at 66 m depth, below the NP, and point F is located underneath the left A pile. The aim is to capture some typical response from the soil affected by the system.

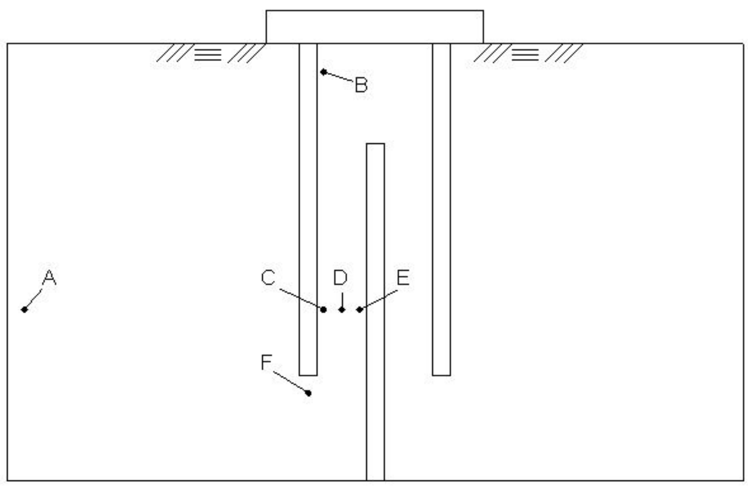

Figure 5.1: 6 points selected for stress path interpretation.

Stress plots in the p'-q plane for point C in both a sheared and OCR reset model are illustrated in Figure 5.2 - Figure 5.3. The same plots for the remaining points can be found in Appendix E. As can be seen in these figures some phases do have a big influence on the stresses. For some selected phases the state variables α_x, α_y, α_z, α_0 and χ_0 are extracted for the same points A-F. These state variables can also be found in Appendix E. χ_0 describes the amount of bonding between soil particles and when this variable decreases the size of the intrinsic yield surface in the Creep-SCLAY1S model increases. One thing notable in the stress plots is that during the first 200 year consolidation stage, the mean effective stress p' and the deviatoric stress q both increases about 10-20 kPa for all points. This is due to the dissipation of pore water pressure, independent on what type of model used. During this consolidation phase the anisotropic vector containing $\alpha_{x,y,z}$ values changes less than 0.1%. The value of χ decreases slightly for all points except for point B where it reduces about 40%. Note that in point A in the OCR reset model, most of the stress change is occurring during this pre phase which is realistic so far away from the actual construction. In the sheared model point A is clearly affected by the shearing of piles, yet again implying unrealistic lateral effects due to 2D pile modelling using plate elements.

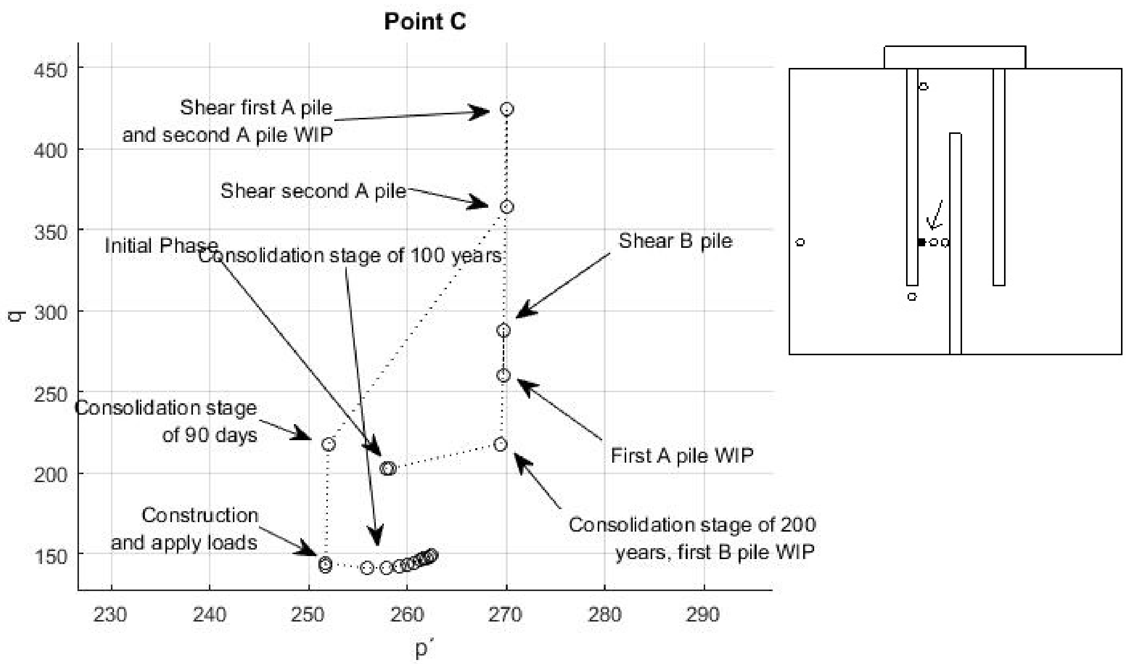

Figure 5.2: Stress plot for point C in a sheared model. The arrows mark phases where the stresses changes significantly. Stresses in [kPa].

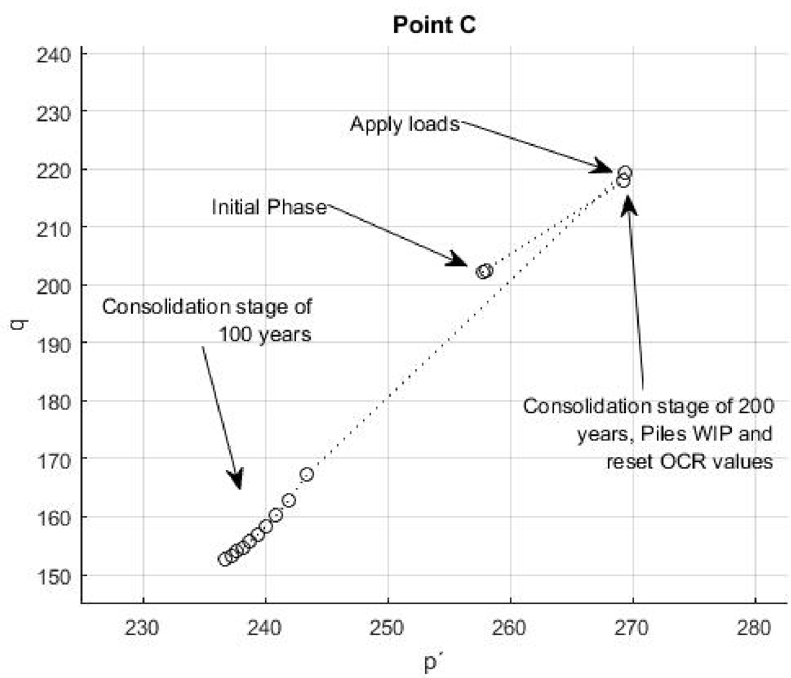

Figure 5.3: Stress plot for point C in a OCR model. The arrows mark phases where the stresses changes significantly. Stresses in [kPa].

The difference in modelling the installation effects becomes clear when comparing the mid phases for the models. In all points in the OCR reset model, the stresses do not change that much compared to the sheared pile model, where the deviatoric stress increases about 100% for points B-E and even more for point F. A reason to why the stresses increase so much in point F is that it is located close to the pile toe where the vertical stresses are large. This is shown in the stress plots in Appendix E where the increase of deviatoric stress in the sheared models is almost vertical for all points.

In the sheared pile model, after the installation phases, there is a 90 day consolidation stage, where there is a decrease in both p' and q for point B-F, implying there is an increase of excess pore pressure close to the piles. This is an effect missed in the OCR reset models.

During these phases in the sheared pile model, points B-F experience a decrease in deviatoric stresses while in the OCR reset model there is an increase. The only thing that changes during this period is an extra load applied to simulate loads from the building. This should result in an increase in the vertical stresses, showing as an increase σ'_1 in Plaxis resulting in an increase in both p' and q. The reason the points in the sheared models show a decrease in stresses is most likely coming from the unrealistic tensile forces created in the piles during the shearing phase.

In the final consolidation stages the points C-E shows a totally different behaviour in the sheared pile model compared to the OCR reset model. In the sheared models the stresses in these point develop similar to the initial 200 year consolidation stage meaning, there is an increase of both deviatoric and effective mean stress, most likely as a result of dissipation of pore pressures. In the OCR model the stresses move in the opposite direction in the p'-q plane where there is an overall reduction of stresses. The effective stresses for point C for both sheared and OCR reset models is illustrated in Figure 5.4 - Figure 5.8. The reference values $\sigma'_{x,y,z}$ are the values taken directly before the consolidation stages. Point B, E and F show similar behaviour since they are also located within the area where the soil is replaced in OCR models.

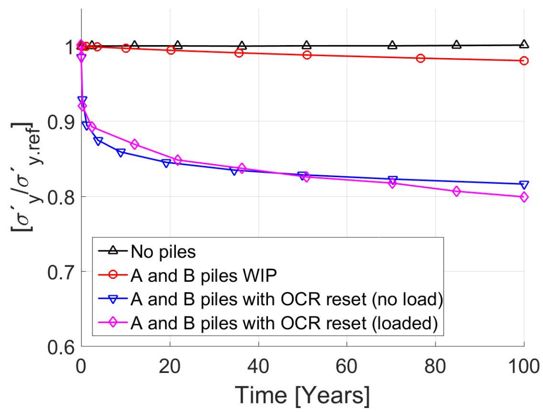

Figure 5.4: The variation of σ'_y during the last consolidation stages at point C, normalized to the initial value $\sigma'_{y.ref}$.

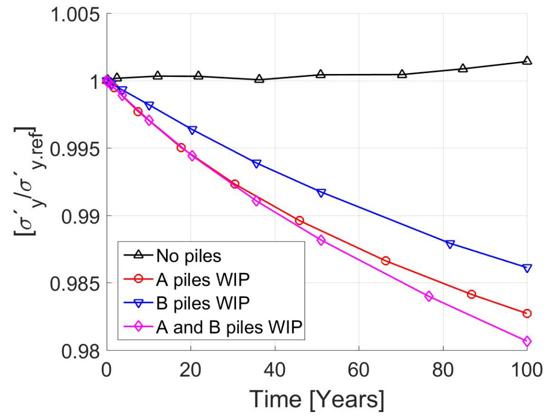

Figure 5.5: The difference in σ'_y when no installation effects are modelled, normalized to $\sigma'_{y.ref}$.

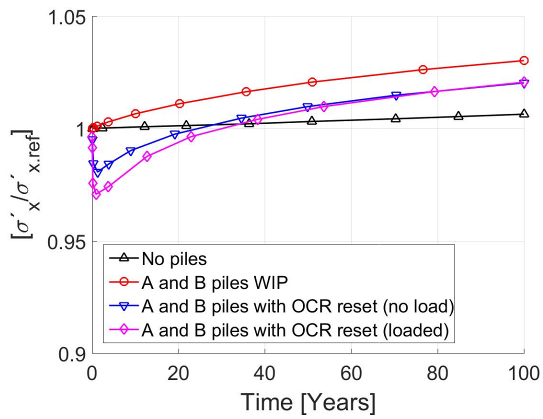

Figure 5.6: The variation of σ'_x during the last consolidation stages at point C, normalized to the initial value $\sigma'_{x.ref}$.

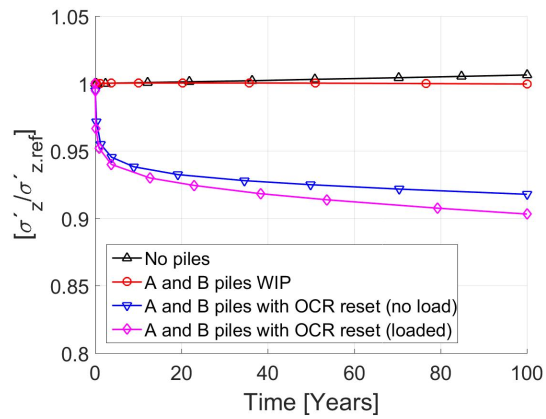

Figure 5.7: The variation of σ'_z during the last consolidation stages at point C, normalized to the initial value $\sigma'_{z.ref}$.

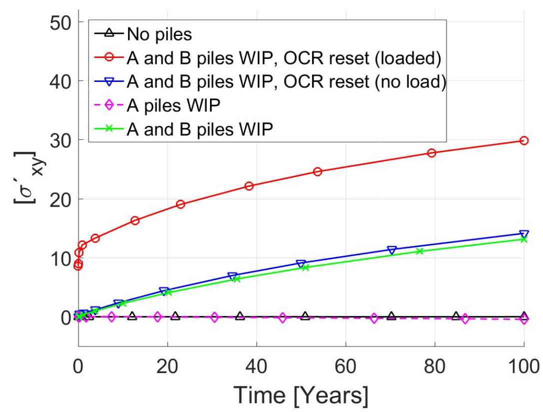

Figure 5.8: The variation of the shear stresses σ'_{xy} during the last consolidation stages at point C.

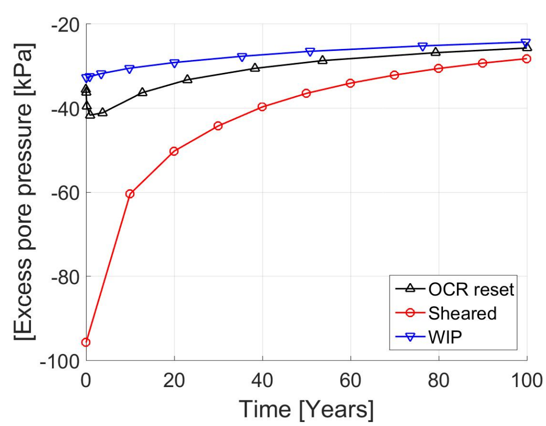

Figure 5.9: The variation of excess pore pressures at point C during the last consolidation stages

The reason of different behaviour between an OCR and a sheared model is due to the drastic decrease in σ'_y and σ'_z as a result from resetting the OCR, see Figure 5.4 and Figure 5.7. Another contributing factor is that the B pile is unloading the soil more over time which can be seen in Figure 5.5 by comparing the A pile system with the overlapping pile system. In Figure 5.9 the excess pore pressure for both an OCR and sheared model are plotted which show, as they should over time, a reduction over time. Also notable is the larger initial excess pore pressure in a sheared model. The shear forces is barely affected by resetting the OCR, see Figure 5.8. The shear forces in point C is getting larger over time in an overlapping pile system which indicates that the load is transferred partially shear. At the same point in an A pile system this shear force contribution is negligible.

In Figure 5.10 and Figure 5.11 the shear forces over depth is plotted for a overlapping and a floating system respectively, using reference spacing. It is obvious that the two systems behaves quite differently as the overlapping pile system develops more shear forces between the piles. This is a wanted effect from the system, illustrating the functionally of the B piles. Additionally, it is possible to see the growth in magnitude of shear forces in the overlapping system over time as the negative skin friction increases. In the same manner Figure 5.12 and Figure 5.13 displays the shear forces for a pile spacing ten times the reference spacing. In this case the curves are more equal to each other for the first 10 years. Afterwards the overlapping system tends to yield more shear forces at larger depths, implying that the forces still is transmitted to the B pile even at a large distance. For the floating system with reference spacing, some sort of plugging of the system is probably occurring as no shear forces are imposed to the soil between the piles.

In Figure 5.14 and Figure 5.15 the shear forces are plotted between the A and B pile at two different depths for ten times the reference spacing. From these plots it is possible to distinguish a point in which the shear forces in the soil switches sign, which is shown to move closer to the B pile as the time proceeds. This would imply that over time more forces are transferred by shear from the A piles to the B piles.

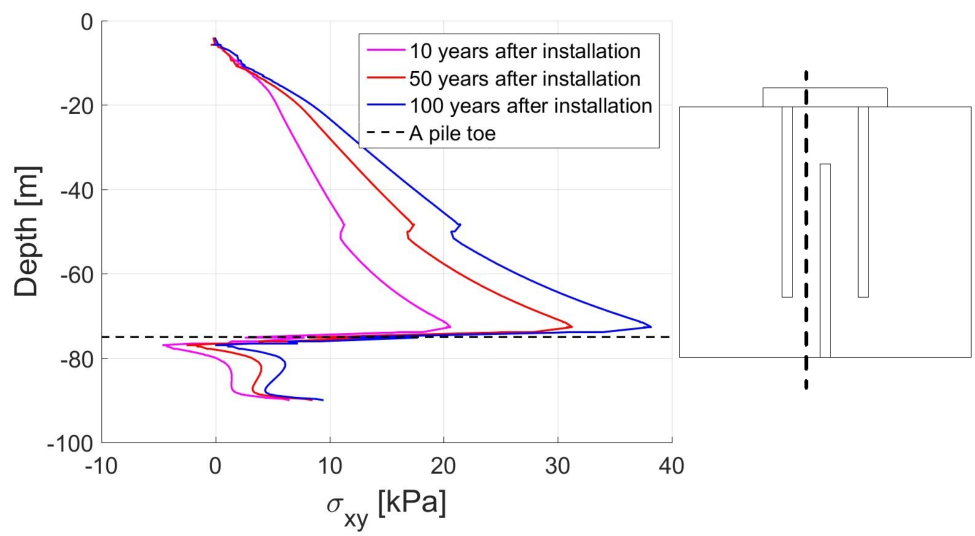

Figure 5.10: Shear force, σ_{xy} distribution between A and B piles in an overlapping pile system, $Sp_{A.ref}$.

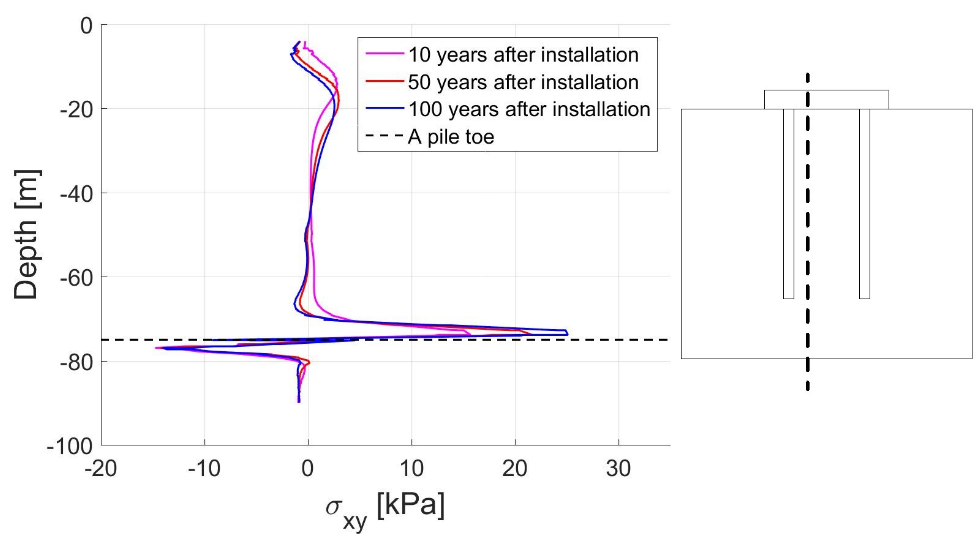

Figure 5.11: Shear force, σ_{xy} distribution in a floating pile system, $Sp_{A.ref}$.

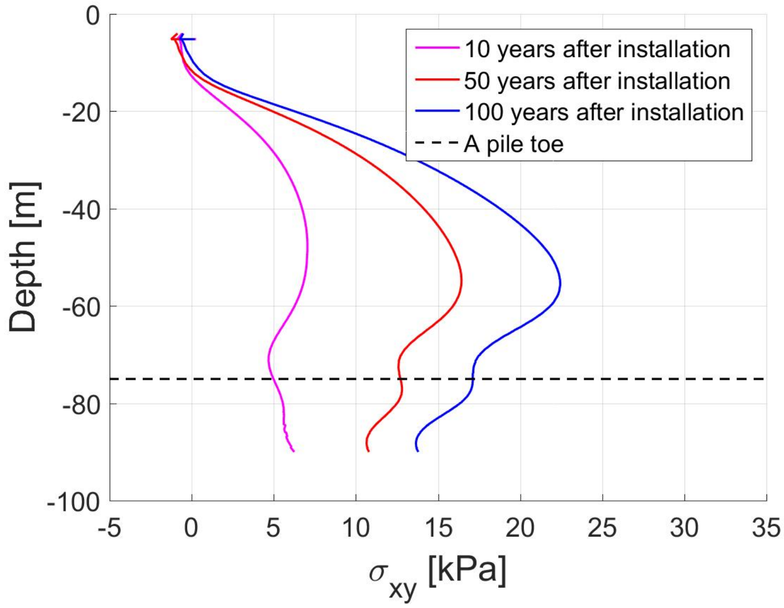

Figure 5.12: Shear force, σ_{xy} distribution between A and B piles in an overlapping pile system, $10 \times Sp_{A.ref}$.

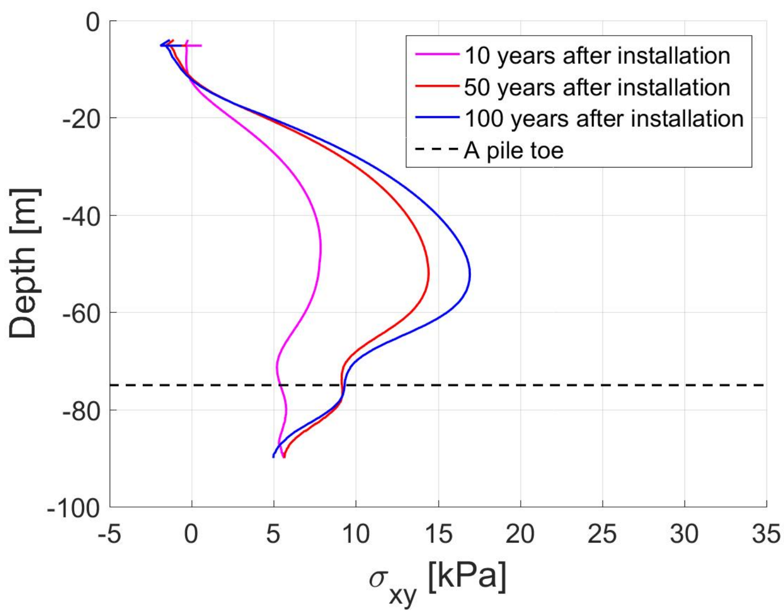

Figure 5.13: Shear force, σ_{xy} distribution in a floating pile system, $10 \times Sp_{A.ref}$.

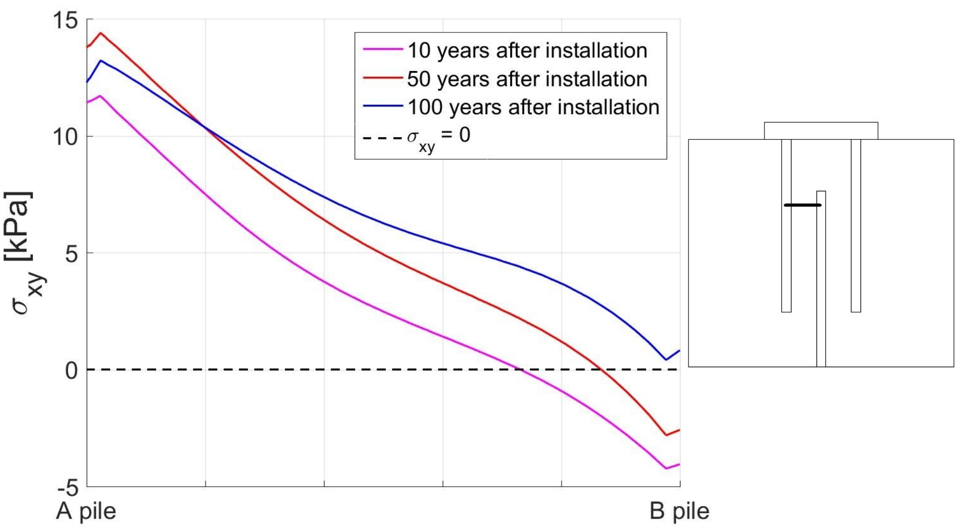

Figure 5.14: Shear force, σ_{xy} distribution between A and B piles at 20 m depth, $10 \times Sp_{A.ref}$.

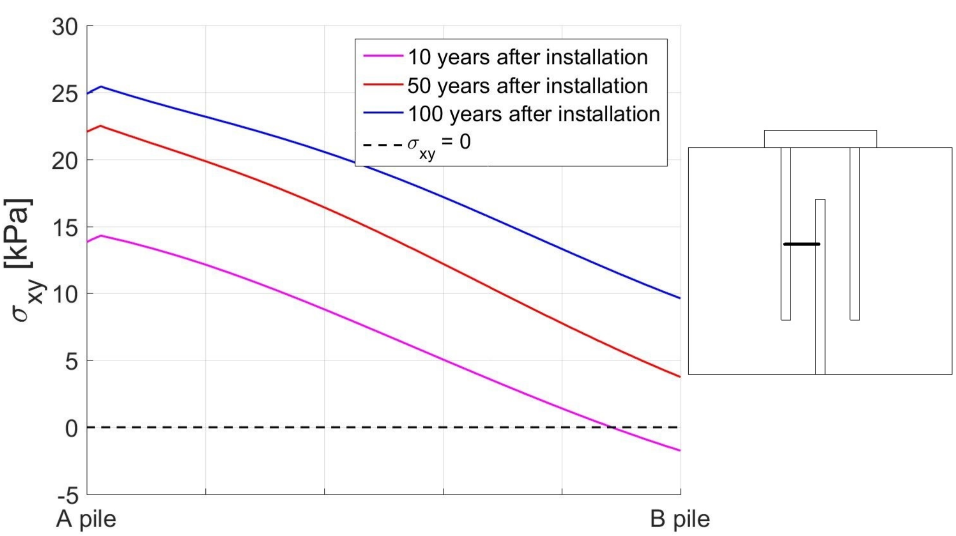

Figure 5.15: Shear force, σ_{xy} distribution at 40 m depth, $10 \times Sp_{A.ref}$.

5.2 Axial forces

The stress distribution in the the A pile in both an OCR reset model and a sheared pile model, after 10 and 100 years, are shown in Figure 5.16 and Figure 5.17. As can be seen the forces in the A pile in a sheared model are tensile at the bottom part which are present even after 100 years. In Figure 5.18 and

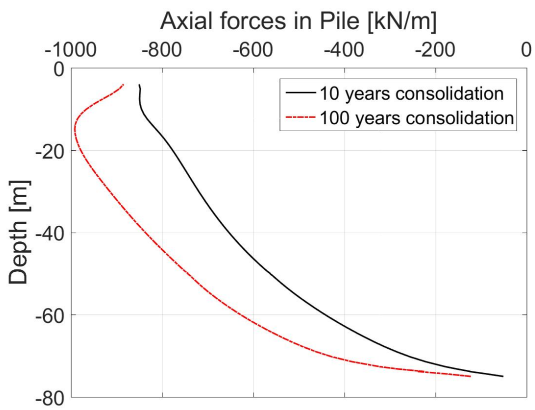

Figure 5.16: A piles, OCR Model

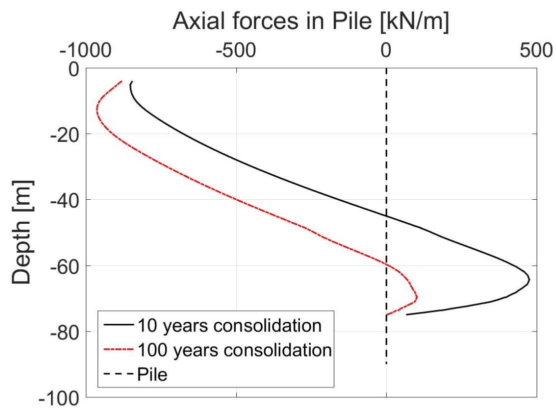

Figure 5.17: A piles, Sheared Model

Figure 5.19 the tensile force distribution in a B pile after some time using both shear and soil replacement as installation technique can be seen. In the latter models large tensile forces are built up in the piles which would, as previously stated, cause an unrealistic load transfer from the soil to the piles and vice versa.

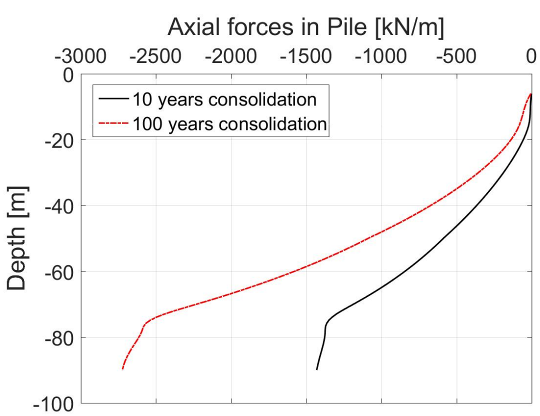

Figure 5.18: B piles, OCR Model

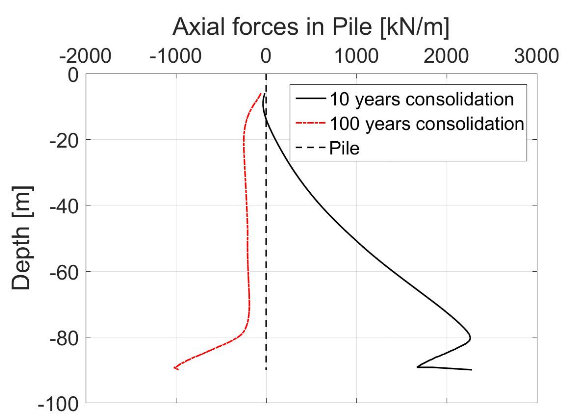

Figure 5.19: B piles, Sheared Model

The large tensile forces in the piles is the reason why models where the OCR have been reset are used for further analysing the of forces in the piles. When Sp_A is altered the axial forces in the piles changes. The results for the left A pile for both an overlapping pile system and a floating pile system, at two depths, are presented in Figure 5.20 - Figure 5.23. The spacings are normalized to the reference spacing $Sp_{A.ref}$ = 7 m. Figure 5.20 and Figure 5.21 shows the axial force at 73 m depth i.e. 2 m above the pile toe. Figure 5.22 and Figure 5.23 shows the axial force at 40 m depth which is one of the shallowest depths where a difference between the two can be visually distinguished. As can be seen in Figure 5.20

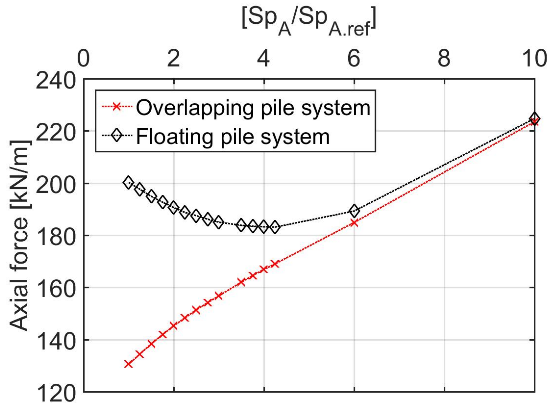

Figure 5.20: Force in left A pile at 73 m depth after 10 years

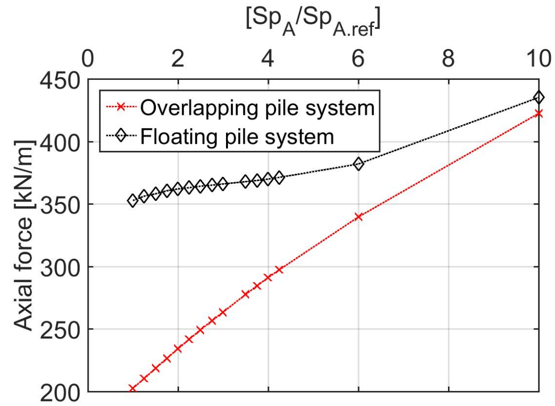

Figure 5.21: Force in left A pile at 73 m depth after 100 years

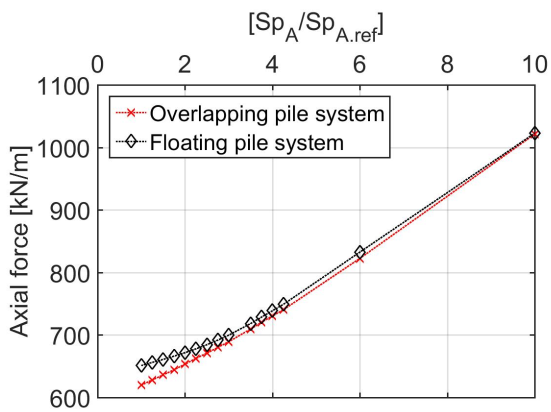

Figure 5.22: Force in left A pile at 40 m depth after 10 years

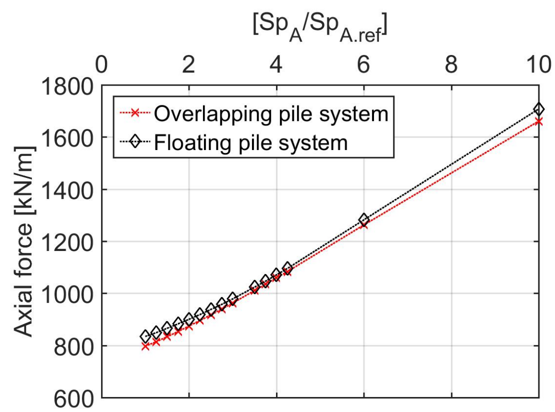

Figure 5.23: Force in left A pile at 40 m depth after 100 years

- Figure 5.23, the A piles in a floating pile system experiences larger axial forces at greater depths than the same piles in an overlapping pile system implying that the B pile is unloading the A piles. At 40 m depth this unloading effect is smaller and at depths above 40 m it barely influences the A piles. When Sp_A increases to above 6 times the reference spacing the effect of the B piles reduces even at larger depth. The magnitude of maximum axial force in the B pile is illustrated in Figure 5.24. By analysing the location of the maximum forces in the A piles, it is possible to determine the location of the neutral plane. Above this point negative skin friction increases the forces in the pile, and below this point the pile is unloaded. The location of the neutral plane is lowered as the soil settlements progresses and the negative skin friction hence increases. By comparing the size of the axial pile force in Figure 5.20 and Figure 5.21 it can be stated that the amount of negative skin friction developed in an A pile, over 100 years is about 150 [kN/m] for a floating pile system with reference spacing and about 70 [kN/m] for an

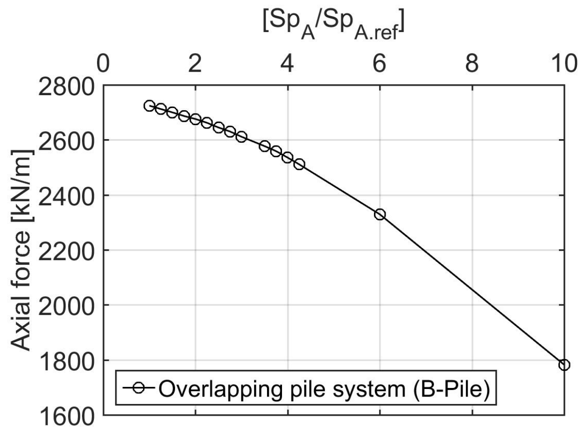

Figure 5.24: The maximum axial force in the B pile after 100 years

overlapping pile system with the same spacing. This behaviour is observed for larger spacing as well, again showing the unloading effect of the B pile. When the overlapping length L_o is reduced the axial forces in both A and B piles are decreasing.

5.3 Settlement of the pile system

Increasing the number of piles would make the soil stiffer, but also a more expensive solution. Without going too deep into the economic parts of pile driving, it is safe to state that longer and deeper piles will cost more and for end bearing piles the risk of failure during installation is increased. This is why during the analysis of settlements two additional models are created in order to compare if an additional A pile, instead of the B pile, would reduce the settlements in a similar way. These first of these two models is a model with three floating piles of equal length, creating a solution with less amount of total pile meters compared to the overlapping pile system. The second model has the same amount of pile meters as the overlapping system, but the B pile is replaced with an equally long A pile. In Figure 5.25 and Figure 5.26 settlements are illustrated for an overlapping pile system, a floating pile system with two piles and two systems with the additional floating pile installed. All settlements are taken at the pile head of the left A pile. If the spacing between A piles is small, there is no apparent difference to install an extra A pile, unless it is longer. The elongated A pile does reduce the settlements, but not to the same amount as a B pile would do. If only the overlapping pile system and the system with two floating piles are considered, it is possible to see at what spacing Sp_A an overlapping pile system would settle equally as a floating pile system with reference spacing $Sp_{A.ref}$. This is illustrated in Figure 5.27 where the settlements are normalized against the obtained settlements, $u_{y.ref}$ from a floating pile system with reference spacing $Sp_{A.ref}$. The reason for the settlements exceeding the floating reference system can be explained by the increase of negative skin friction that arises from the piles being placed further apart, increasing the influence area. The settlements, as shown in Figure 5.27, becomes equal between the reference floating pile system when the spacing of an overlapping pile system is about 12-13 larger.

Next we look at how the overlapping length, L_o, affects the settlements. The amount of allowed settlements is obtained from a floating pile system with reference spacing $Sp_{A.ref}$ after 100 years which

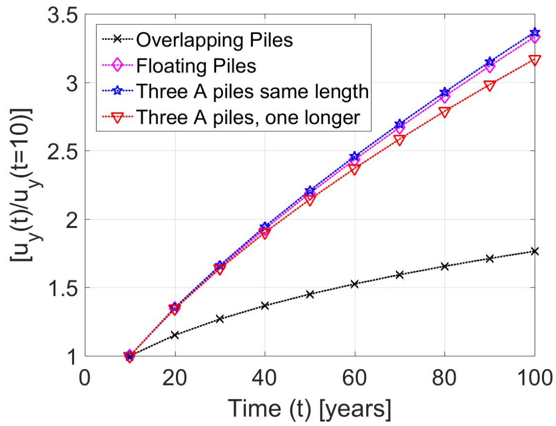

Figure 5.25: Total settlements of different pile systems with $Sp_A = 7$ m.

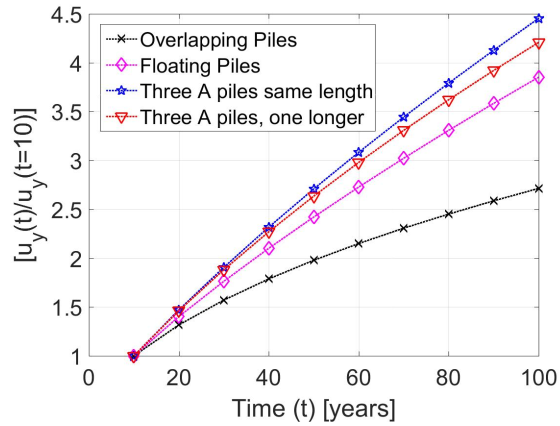

Figure 5.26: Total settlements of different pile systems with $Sp_A = 42$ m

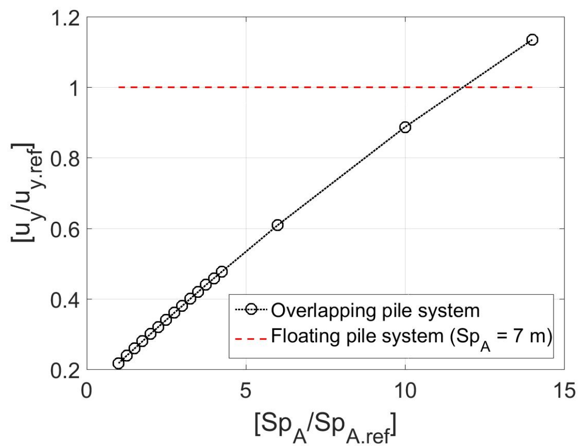

Figure 5.27: Normalized Settlements u_y plotted against different normalized spacings Sp_A

would be about 1.3 [mm/year]. When L_o is reduced the amount of settlement increases and this is illustrated in Figure 5.28 where the settlement is normalized with the settlement obtained in the reference system, $u_{y.ref} = 1.3$ mm. According to the results shown in Figure 5.28 it is possible to make the A piles shorter when installing a B pile, and an overlapping length of about 32 m will have settlements of the same magnitude as for an floating pile system.

When comparing the settlement behaviour between sheared pile models and OCR reset models the difference is quite small, for both overlapping and floating piles, when Sp_A is small. As the spacing Sp_A increases, the influence from the way of modelling installation effects become larger. This is illustrated in Figure 5.29 - Figure 5.31.

The settlement analyses suggest that if overlapping piles are used, the A piles could be shortened from the initial 75 m to around 34 m. An overlapping system with short piles and one B pile would result in a total length $L_{tot} = 152$ m. The floating pile system with two longer A piles has a total pile length $L_{tot} = 142$ m. This is a small difference but if the benefits from using a B pile can be seen in an equal magnitude in the out of plane direction, the overlapping piles might exceed the performance of the floating pile

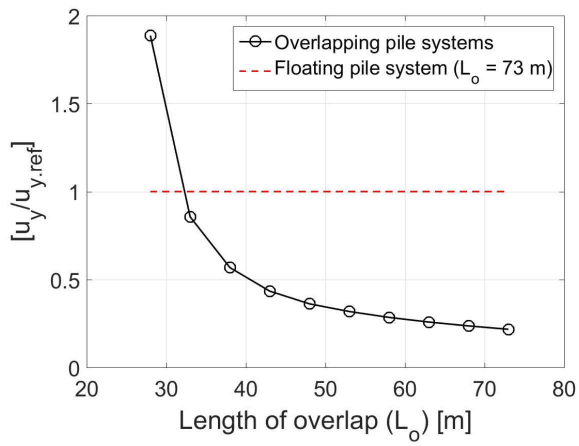

Figure 5.28: Settlements overlapping systems with various length of the overlap L_o.

system. It is also shown that the installation effects have a influence on the final settlements. It can be seen that shearing tends to result in lower final settlements as compared to resetting the OCR.

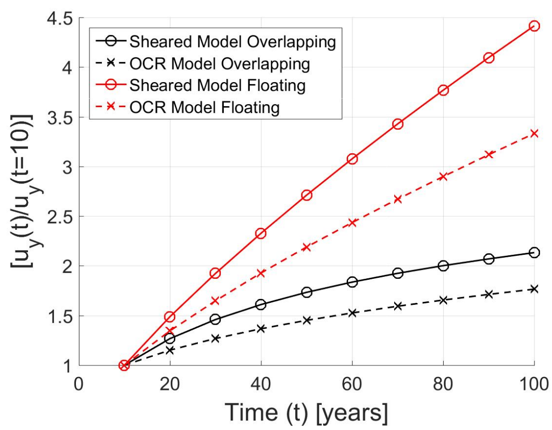

Figure 5.29: Settlement at the pile head of the left A pile, $Sp_A = 7$ m.

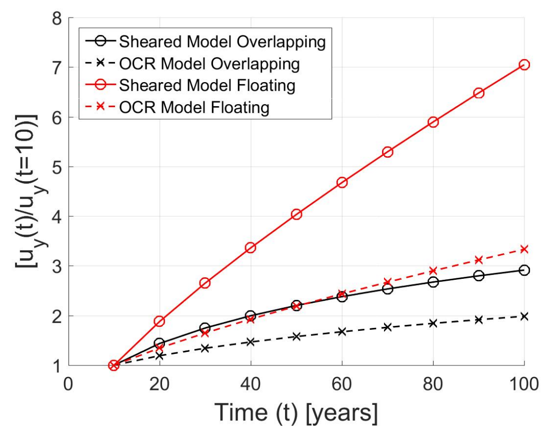

Figure 5.30: Settlement at the pile head of the left A pile, $Sp_A = 14$ m.

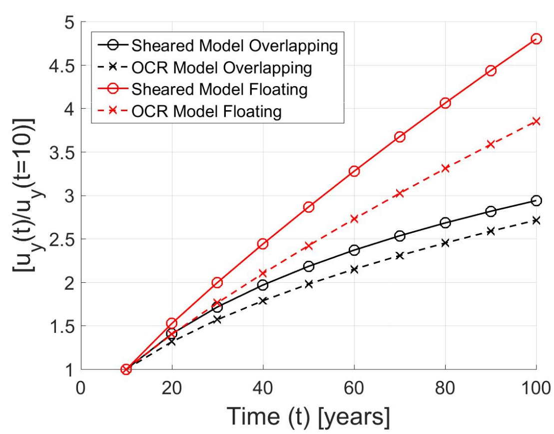

Figure 5.31: Settlement at the pile head of the left A pile, $Sp_A = 42$ m.

6 Concluding remarks

This thesis treats the basics of overlapping pile behaviour, from the time of installation to the end of design life of an overlying structure. The behaviour is analysed using Plaxis 2D and the user defined soil model, Creep-SCLAY1S that includes long term effects. A suitable way of modelling basic overlapping pile systems, able to capture some of the effects from installation, is determined. This system are compared to the more traditional floating pile system in terms of settlements and load transfer. Additionally the limitations of the overlapping system is investigated, as well as the soil response, in order to show the mechanisms.

When modelling an overlapping pile system in Plaxis 2D, plane strain is the only realistic option. As a result the most suitable way of modelling the piles, in this case, is with plate elements see, Section 4.3.

The spacing between the A piles in an overlapping pile system can be increased about 6 times compared to an floating pile system before the positive effects from the B pile diminishes. Additionally it is shown that the A piles could be reduced to around half their initially intended length before showing an inferior behaviour to a floating pile system, see Chapter 5. However, keep in mind that the wall effect from modelling the pile with plate elements, discussed in Section 4.3, might enhance these positive unloading effects in a way that might not be realistic. The wall effect causes an irrational response in both the pile as well as the soil. Installation effects and load transfer affects an area many times more distant than what a single pile or a pile row would do as shown by both the axisymmetric reference model, see Section 4.4, and previous studies.

For the analysis, the user defined soil model Creep-SCLAY1S is implemented in Plaxis. This soil model offers the user the ability to model soil with consideration taken to for example anisotropy and bond degradation. The model will have to be considered as one of the more advanced which in hindsight might not have been the optimal choice for a first FE-interpretation of a relatively new foundation technique. Disregarding the material model's capacity and accuracy it may create more uncertainty for inexperienced users in a problem where limited research has been made. Hence, some effects in soil response, stated to be effects of overlapping piles might as well be a direct or indirect consequence of the material model itself or the parameters inserted to it.

Regarding the parameters, presented in Section 4.1, some weaknesses can be pointed out. Although the CRS and triaxial tests along with the ongoing settlements in the area has been reasonably matched, see Section 4.2, the parameters that was matched by trial and error poses for some inaccuracy. The curves from the conducted triaxial and CRS tests can be matched with other parameter values than the ones used in the continued modelling. This in turn means that the actual soil response to the pile installation could be different from the one presented in the results. Furthermore the triaxial tests are only captured until 2% which is approximately the top of the ϵ - q stress curve. This should only affect the outcome of the model in cases of high strains i.e. strains above 2% but some reservation for inaccuracy will still have to be noted. However, in the comparative studies the potential errors due to the soil model and parameters will affect both the overlapping and the floating pile system in the same way.

In this thesis and with the models used, the way of capturing installation effects are only reflecting the reality to a certain extent. First of all, by not being able to volume expand the piles in a satisfactory manner, the fact of soil being pushed away from the piles in the lateral direction is overlooked. For the shearing, some issues can be pointed out such as the unreasonable axial forces in the piles which influences the analysis of force transmission between the A and B piles.

6.1 Continued Research

As this thesis only covers the basic modelling of piles in an overlapping system, a few areas, comparisons and ideas remains untouched. Following are some ideas and directions where the continued research, in analysing overlapping pile systems, could progress.

- One large limitation is that overlapping piles is a 3D problem but modelled as 2D in this thesis. A few drawbacks of modelling in 2D, in the way it was done in this thesis, is that the soil response might not always be reasonable due to, for example wall effects and anisotropy of soil. The next step would be to create 3D models to verify similar behaviour.
- Real measurements of an overlapping pile system is crucial to validate any way of modelling this problem. Ideally this would include both early measurements from installation as well as long term measurements. The early measurements might include pore pressure monitoring, soil movements and force distribution in the piles.
- A detailed study should be done on piling economy since this is often a deciding factor when designing and selecting a foundation system. For sure three A piles are less expensive than a system with two A piles and one B pile of equal total pile length, but if the A piles can be shortened and the spacing increased, overlapping piles might be a valid option.
- In this thesis solely the ongoing settlement rate at Regionens Hus has been studied. Further studies might regard how different soils affects the efficiency of overlapping piles. As negative skin friction will increase with larger settlement, the efficiency of the system might increase.
- How will alternative configurations of overlapping piles behave? In the thesis just a few different pile set ups has been studied, but further studies might treat what happens if the system is asymmetric, or if for example B piles are placed outside the A piles. Larger systems should also be studied in order to include group effects and to find the least amount of B piles needed.
- This thesis has aimed to study the benefits of overlapping piles with regards to settlements. Can the use of B-piles have other positive effects, such as reduced settlements for adjacent structures? One further topic could be to assess how B piles could work as load absorbers reducing loads to spread out.

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A Pile layout

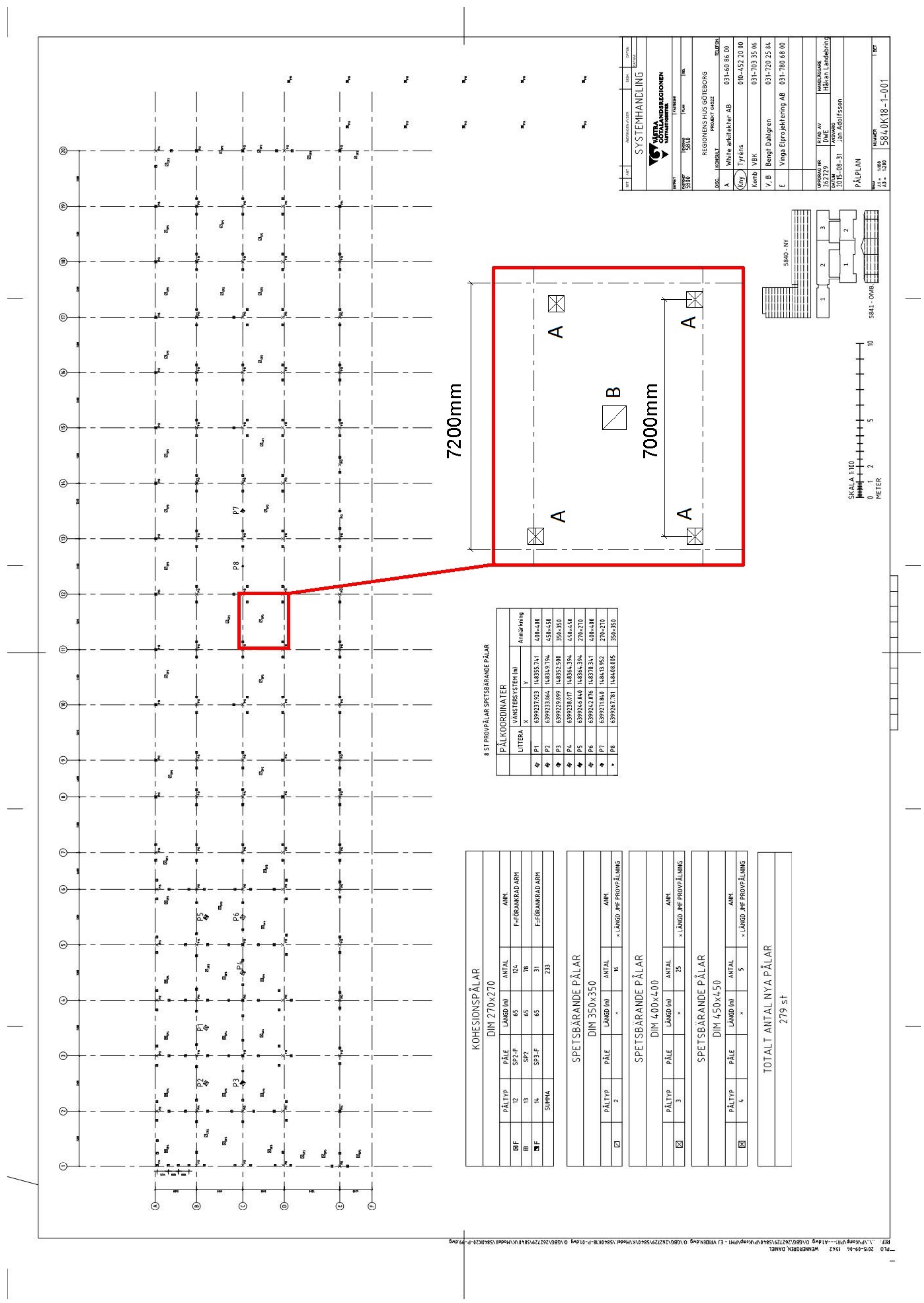

Figure A.1: Drawing over pile locations for Regionens Hus

B Determination of OCR and POP

The effective stresses are calculated with a soil density from lab tests conducted by BohusGeo and the assumption of the water table being located at 2 m depth from surface. The pre consolidation pressure is based upon available CRS tests by BohusGeo together with the Sällfors correction. The values are plotted in fig. B.1. The OCR values are calculated as $\frac{\sigma'_c}{\sigma'_v}$ and the POP values are the difference $\sigma'_c - \sigma'_v$.

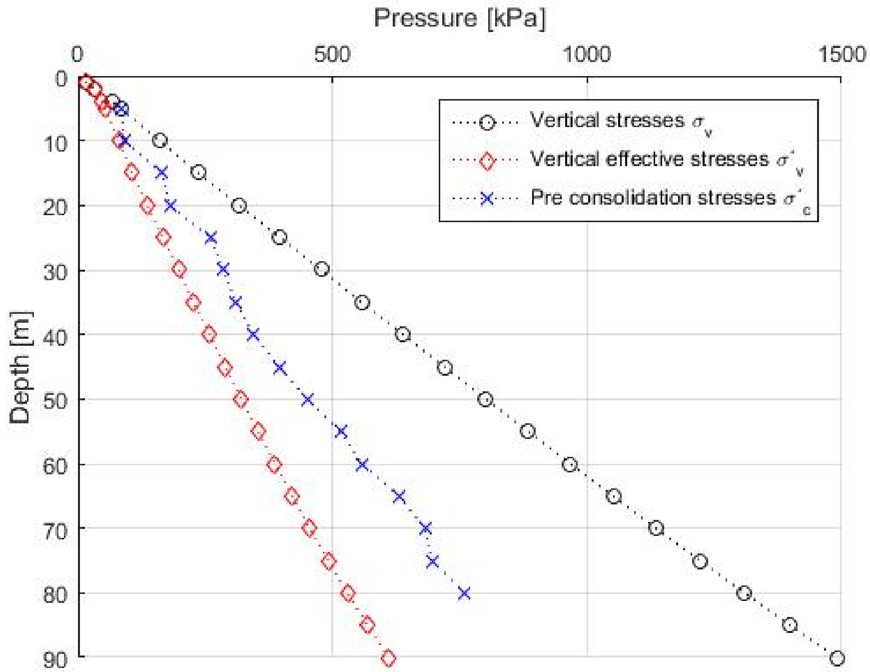

Figure B.1: Vertical, total and effective, stresses together with pre consolidation stresses.

C Model parameters

Table C.1: Parameters through depth

Parameter type	Parameters	Symb.	Layer	
			Upper	Lower
Isotropic	Modified swelling index	κ^*	0,016	0,021
	Intrinsic compression index	λ_i^*	0,09	0,098
	Poissions ratio	ν'	0,18	0,18
	Friction angle	ϕ'_{cv}	34	31
	Stress ratio at critical state in compression	M_c	1,375	1,244
	Stress ratio at critical state in extension	M_e	0,943	0,879
Anisotropic	Initial inclination of yield surface	α_0	0,527	0,474
	Absolute effect. in rotational hardening	ω	35	35
	Relative effect. in rotational hardening	ω_d	0,928	0,807
Destructuration	Initial bonding	χ_0	19	17
	Absolute rate of destruction	ξ	9,4	9
	Relative rate of destruction	ξ_d	0,2	0,2
Viscous	Intrinsic creep coefficient	μ_i^*	0,00125	0,00125
	Reference time	τ_d	1	1
Initial stress	Buoyant unit weight [kN/m^3]	γ'	16,1	17,1
	Lateral earth pressure at rest (Normally consolidated soil)	K_0^{NC}	0,441	0,485
	Over-consolidation ratio	OCR	1,28	1,26
	Initial void ratio	e_0	1,65	1,43
	Permeability [$*10^{-5}\ m/day$]	k_{xy}	1,5	1,0

D Modelling phases

The modelling phases in Plaxis are described here for both shear and OCR models respectively.

Shear Models

• Phase 1 (Initial phase)	K_0 procedure.
• Phase 2 (Placement of fill)	A 4 m layer of fill material is placed.
• Phase 3 (Consolidation)	200 years consolidation, which is the time from when the fill was placed until present.
• Phase 4 (Load - fill)	The fill layer is removed and replaced with an equivalent load. This step is to keep the modelling simple.
• Phase 5 (B pile WIP)	B pile is wished in place.
• Phase 6 (Shearing)	Pile is sheared until failure by applying a line displacement on the pile. Also, the bottom boundary is temporarily opened allowing the pile(s) to penetrate.
• Phase 7 (Locking of B pile)	B pile is locked in position by removing the line displacement and closing the bottom boundary.
• Phase 8 (A Piles WIP)	A piles are wished in place.
• Phase 9 (Shearing)	A piles are sheared until failure by applying a line displacement on the piles.
• Phase 10 (Consolidation)	90 days consolidation still keeping the line displacement on the A piles.
• Phase 11 (Release line displacements)	The line displacements on the A piles are removed.
• Phase 12 (Resetting displacements)	The displacements are reset.
• Phase 13 (Construction of bottom plate)	The plate is modelled as a very stiff plate without any density.
• Phase 14 (Loading)	A uniformly distributed load is applied on the building.
• Phase 15 (Consolidation)	100 years consolidation representing the life span of the construction.

OCR Models

• Phase 1 (Initial phase)	K_0 procedure.
• Phase 2 (Placement of fill)	A 4 m layer of fill material is placed.
• Phase 3 (Consolidation)	200 years consolidation, which is the time from when the fill was placed until present.
• Phase 4 (Load - fill)	The fill layer is removed and replaced with an equivalent load. This step is to keep the modelling simple.
• Phase 5 (B pile WIP)	B pile is wished in place.
• Phase 6 (First A pile WIP)	The first A pile is wished in place.
• Phase 7 (Second A pile WIP)	The second A pile is wished in place.
• Phase 8 (Resetting OCR step 1)	Soil around the piles is substituted to new soil which deletes state parameter values.
• Phase 9 (Resetting OCR step 2)	Soil around the piles is again substituted to new soil with updated OCR.
• Phase 10 (Resetting displacements)	The displacements are reset
• Phase 11 (Construction of bottom plate)	The plate is modelled as a very stiff plate without any density.
• Phase 12 (Loading)	A uniformly distributed load is applied on the building.
• Phase 13 (Consolidation)	100 years consolidation representing the life span of the construction.

E Stress path interpretation

Stress plots for both OCR and sheared models for all points A-F can be seen in this Appendix. Finally a table with all the state variables for some selected phases corresponding for the same points A-F is presented.

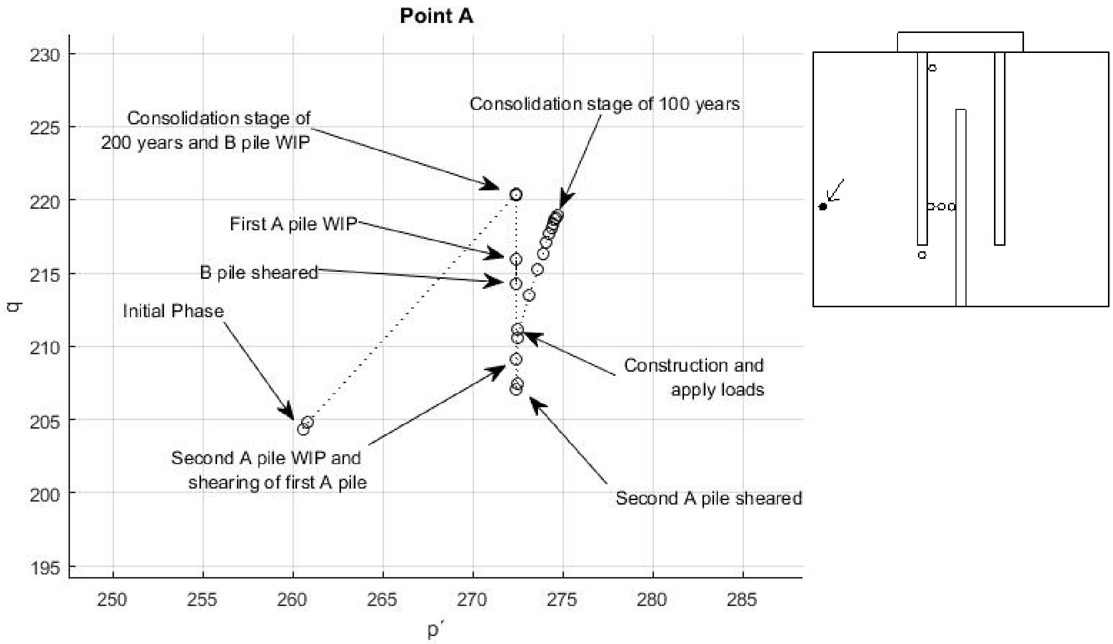

Figure E.1: Stresses in point A, sheared model

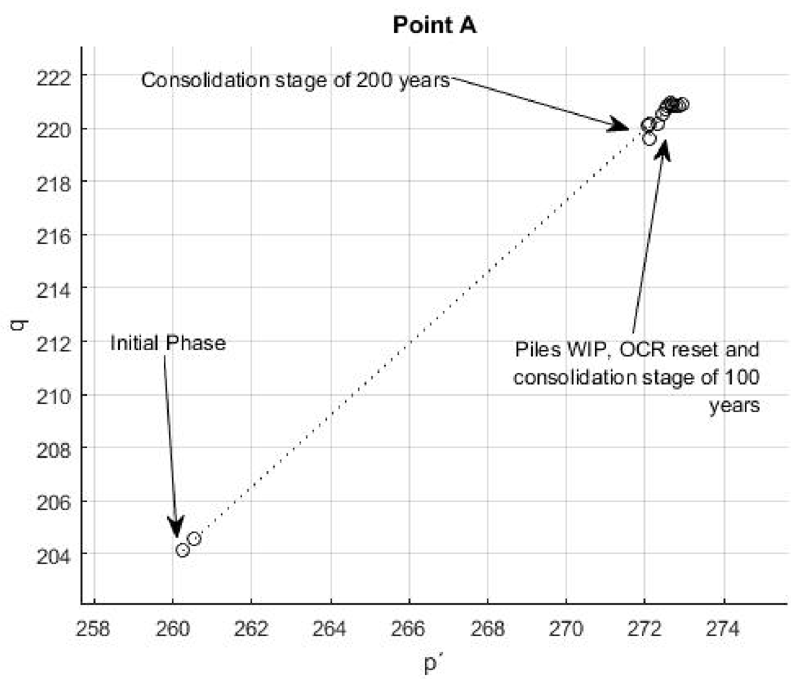

Figure E.2: Stresses in point A, OCR model

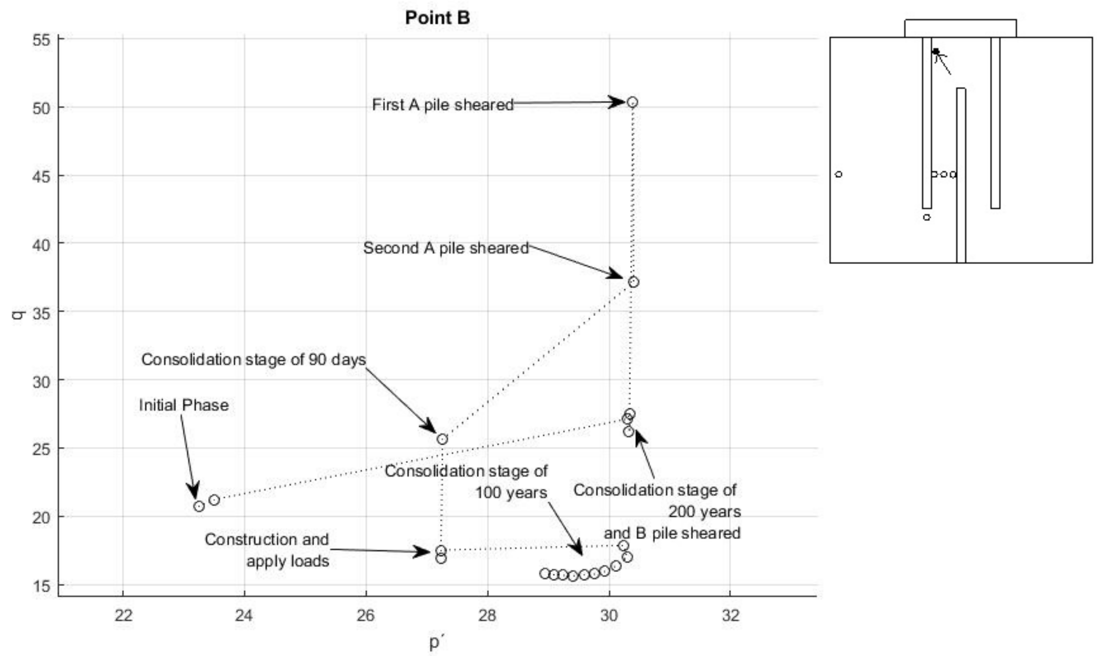

Figure E.3: Stresses in point B, sheared model

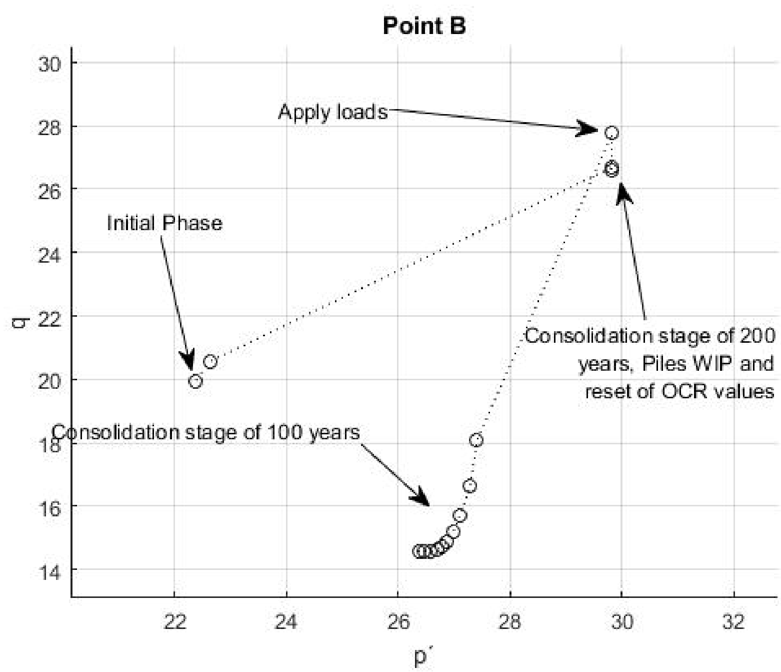

Figure E.4: Stresses in point B, OCR model

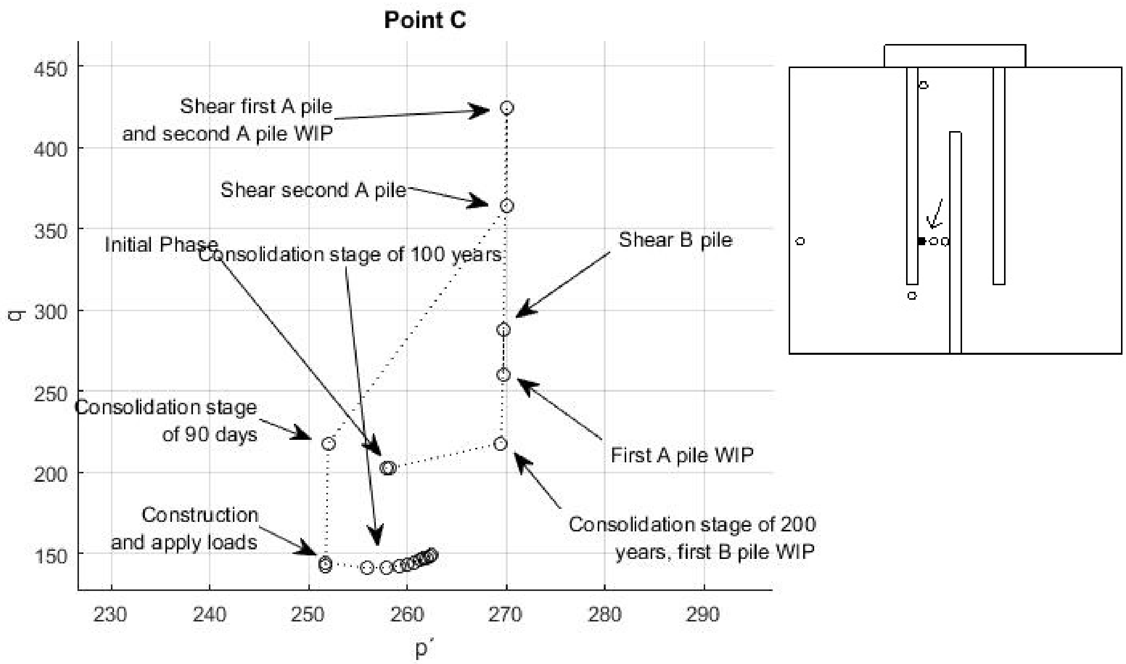

Figure E.5: Stresses in point C, sheared model

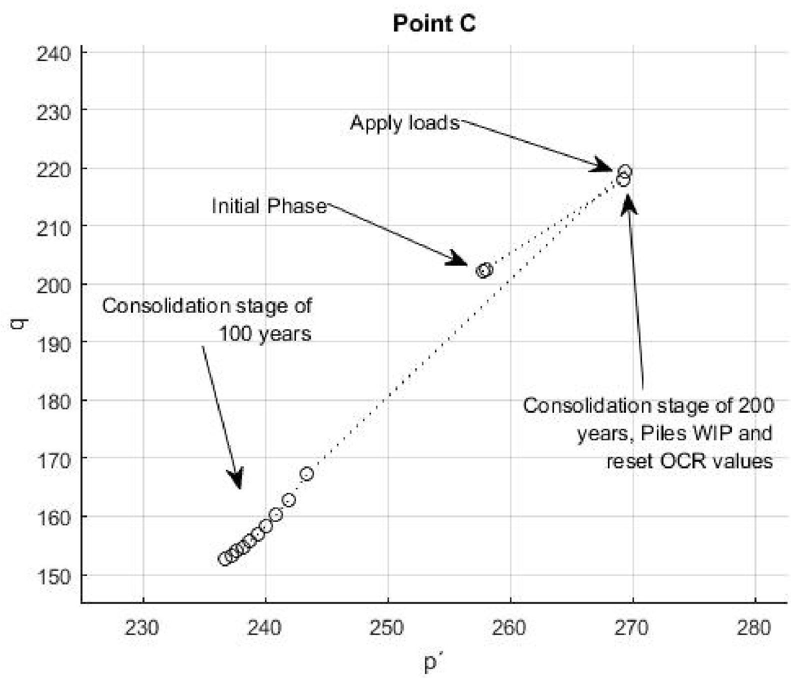

Figure E.6: Stresses in point C, OCR model

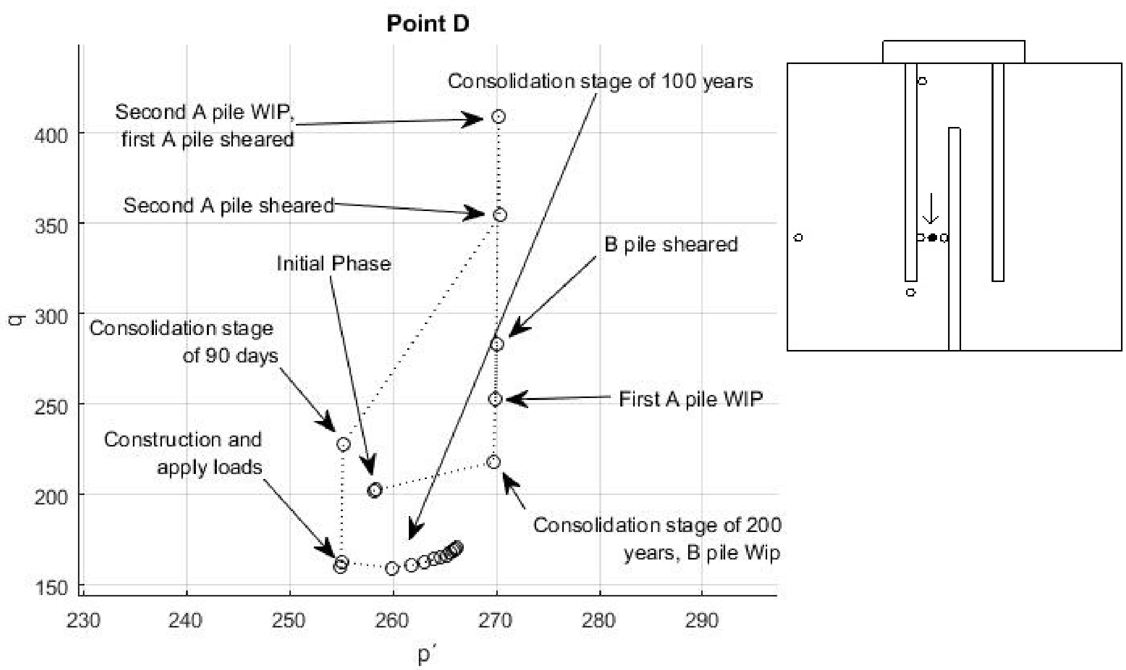

Figure E.7: Stresses in point D, sheared model

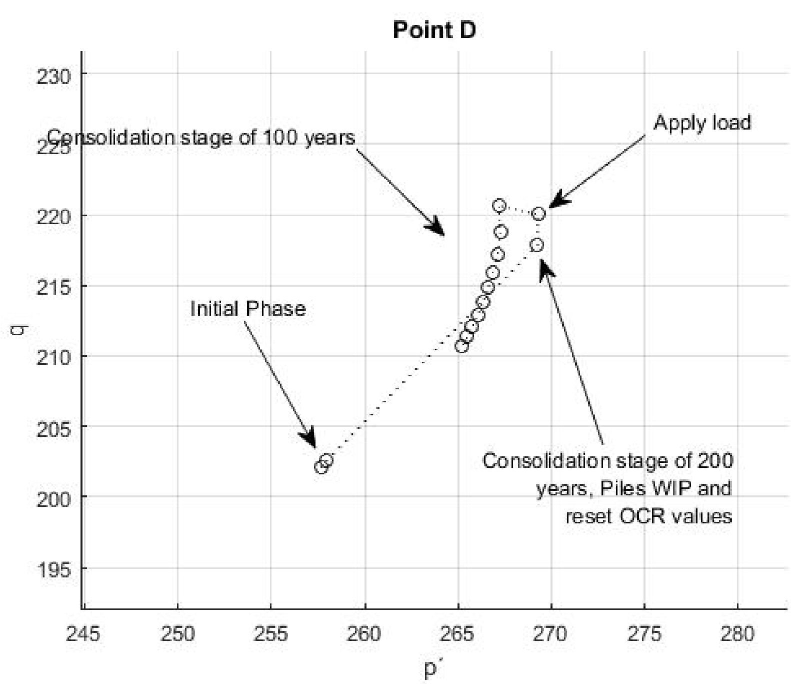

Figure E.8: Stresses in point D, OCR model

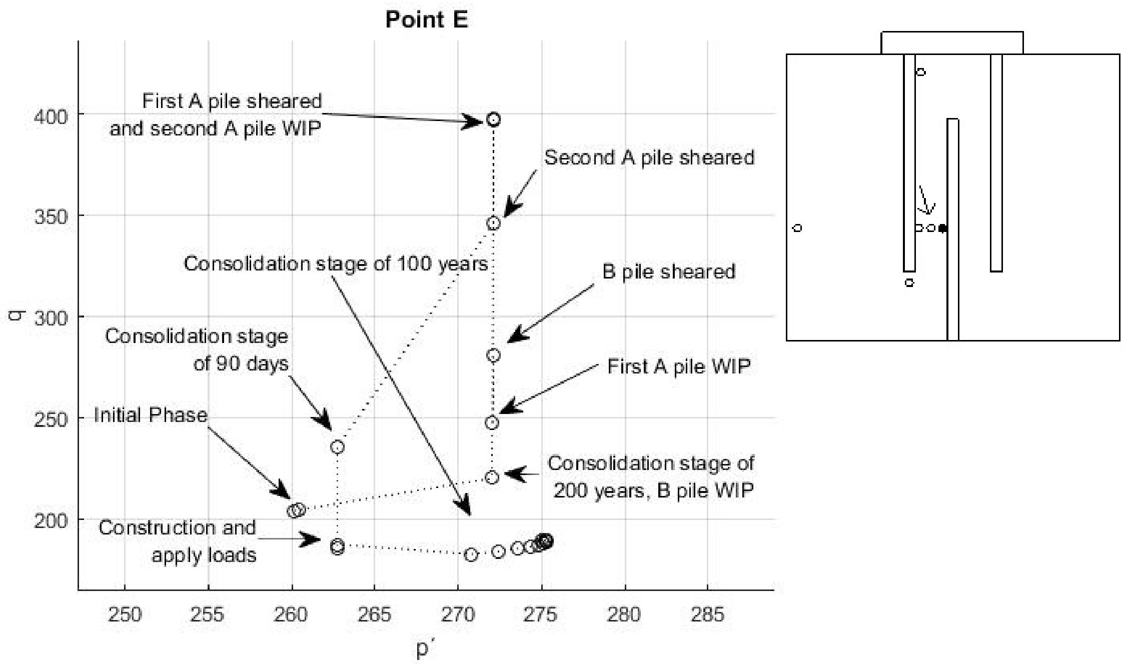

Figure E.9: Stresses in point E, sheared model

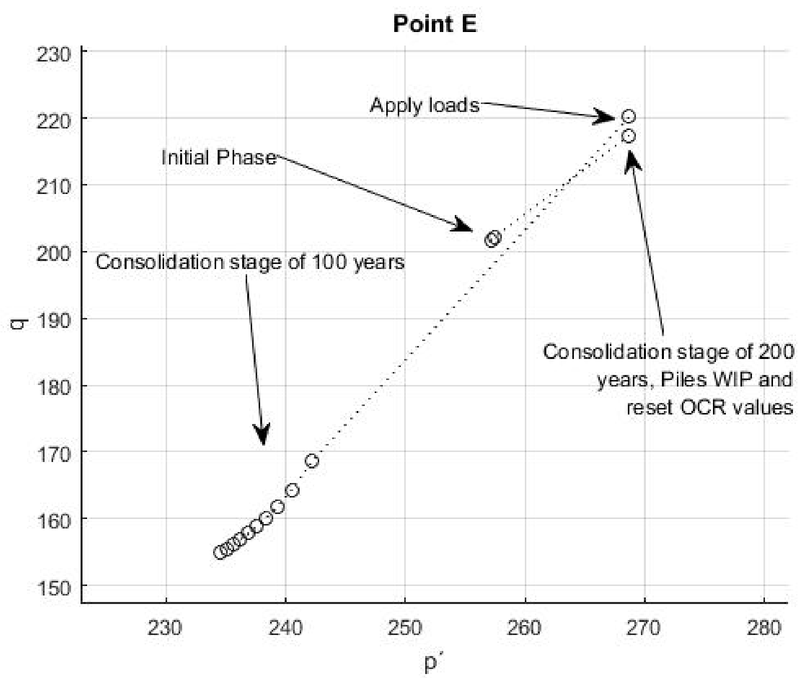

Figure E.10: Stresses in point E, OCR model

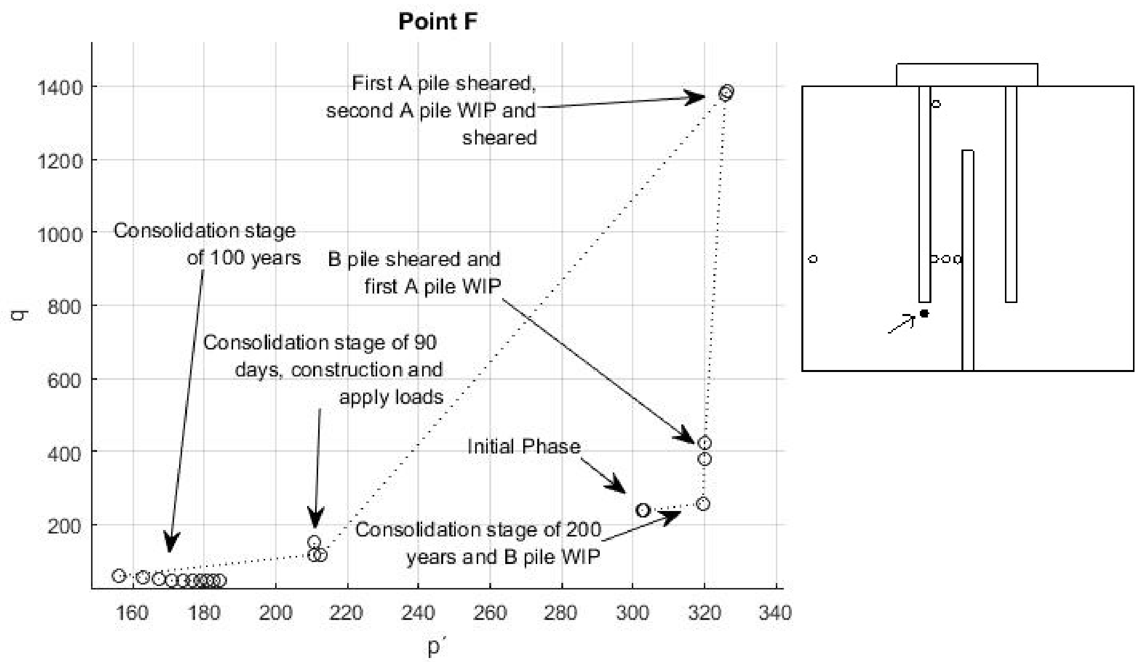

Figure E.11: Stresses in point F, sheared model

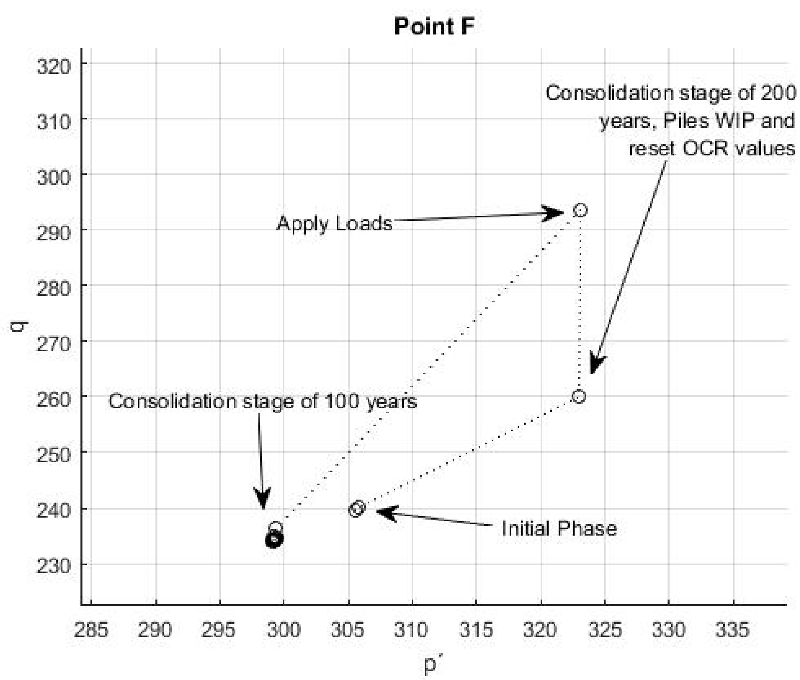

Figure E.12: Stresses in point F, OCR model

Table E.1: Point A Soil replace

Point A (Soil replace)	α x	α y	α z	α 0	χ
Initial	0.00000	0.00000	0.00000	0.00000	0.00000
Placement of fill	0.84203	1.31593	0.84203	0.47390	17.00000
Consol. 200y	0.84208	1.31584	0.84208	0.47376	16.81838
Installation					
Consol. 10y	0.84208	1.31584	0.84208	0.47375	16.80628
Consol. 100y	0.84209	1.31577	0.84214	0.47366	16.68166

Table E.2: Point A Shear

Point A (Shear)	α x	α y	α z	α 0	χ
Initial	0.00000	0.00000	0.00000	0.00000	0.00000
Placement of fill	0.84203	1.31593	0.84203	0.47390	17.00000
Consol. 200y	0.84208	1.31584	0.84208	0.47376	16.81442
Installation					
Consol. 10y	0.84208	1.31584	0.84208	0.47376	16.81432
Consol. 100y	0.84222	1.31574	0.84204	0.47360	16.74588

Table E.3: Point B Soil replace

Point B (Soil replace)	α x	α y	α z	α 0	χ
Initial	0.00000	0.00000	0.00000	0.00000	0.00000
Placement of fill	0.82443	1.35113	0.82443	0.52670	19.00000
Consol. 200y	0.82440	1.35096	0.82464	0.52645	12.10854
Installation					
Consol. 10y	0.83538	1.33871	0.82591	0.50863	18.13231
Consol. 100y	0.85328	1.32184	0.82487	0.48426	17.56646

Table E.4: Point B Shear

Point B (Shear)	α x	y	z	0	χ
Initial	0.00000	0.00000	0.00000	0.00000	0.00000
Placement of fill	0.82443	1.35113	0.82443	0.52670	19.00000
Consol. 200y	0.82456	1.35102	0.82442	0.52654	12.82004
Installation					
Consol. 10y	0.86000	1.28882	0.85118	0.45299	12.24638
Consol. 100y	0.87014	1.28045	0.84941	0.44181	12.03018

Table E.5: Point C Soil replace

Point C (Soil replace)	α x	y	z	0	χ
Initial	0.00000	0.00000	0.00000	0.00000	0.00000
Placement of fill	0.84203	1.31593	0.84203	0.47390	17.00000
Consol. 200y	0.84208	1.31584	0.84208	0.47377	16.82309
Installation					
Consol. 10y	0.84398	1.31307	0.84296	0.46965	16.52429
Consol. 100y	0.84827	1.30804	0.84370	0.46229	16.25921

Table E.6: Point C Shear

Point C (Shear)	α x	y	z	0	χ
Initial	0.00000	0.00000	0.00000	0.00000	0.00000
Placement of fill	0.84203	1.31593	0.84203	0.47390	17.00000
Consol. 200y	0.84208	1.31584	0.84208	0.47376	16.81979
Installation					
Consol. 10y	0.86547	1.26315	0.87138	0.41018	16.14089
Consol. 100y	0.86547	1.26314	0.87138	0.41017	16.14054

Table E.7: Point D Soil replace

Point D (Soil replace)	α x	y	z	0	χ
Initial	0.00000	0.00000	0.00000	0.00000	0.00000
Placement of fill	0.84203	1.31593	0.84203	0.47390	17.00000
Consol. 200y	0.84208	1.31584	0.84208	0.47377	16.82319
Installation					
Consol. 10y	0.84207	1.31580	0.84212	0.47370	16.80712
Consol. 100y	0.84259	1.31487	0.84254	0.47232	16.70396

Table E.8: Point D Shear

Point D (Shear)	α				χ
	x	y	z	0	
Initial	0.00000	0.00000	0.00000	0.00000	0.00000
Placement of fill	0.84203	1.31593	0.84203	0.47390	17.0000
Consol. 200y	0.84208	1.31584	0.84208	0.47376	16.81604
Installation					
Consol. 10y	0.85595	1.28005	0.86400	0.42779	16.31100
Consol. 100y	0.85595	1.28004	0.86400	0.42778	16.31010

Table E.9: Point E Soil replace

Point E (Soil replace)	α				χ
	x	y	z	0	
Initial	0.00000	0.00000	0.00000	0.00000	0.00000
Placement of fill	0.84203	1.31593	0.84203	0.47390	17.00000
Consol. 200y	0.84208	1.31584	0.84208	0.47377	16.82441
Installation					
Consol. 10y	0.84375	1.31313	0.84312	0.46975	16.51903
Consol. 100y	0.84848	1.30724	0.84428	0.46117	16.22608

Table E.10: Point E Shear

Point E (Shear)	α				χ
	x	y	z	0	
Initial	0.00000	0.00000	0.00000	0.00000	0.00000
Placement of fill	0.84203	1.31593	0.84203	0.47390	17.00000
Consol. 200y	0.84208	1.31584	0.84208	0.47376	16.81808
Installation					
Consol. 10y	0.84638	1.30411	0.84951	0.4573	16.56200
Consol. 100y	0.84659	1.30391	0.84950	0.45702	16.54689

Table E.11: Point F Soil replace

Point F (Soil replace)	α				χ
	x	y	z	0	
Initial	0.00000	0.00000	0.00000	0.00000	0.00000
Placement of fill	0.84203	1.31593	0.84203	0.47390	17.00000
Consol. 200y	0.84213	1.31574	0.84213	0.47361	16.64983
Installation					
Consol. 10y	0.84213	1.31574	0.84213	0.47361	16.64926
Consol. 100y	0.84213	1.31573	0.84214	0.47360	16.64777

Table E.12: Point F Shear

Point F (Shear)	α x	α y	α z	α 0	χ
Initial	0.00000	0.00000	0.00000	0.00000	0.00000
Placement of fill	0.84203	1.31593	0.84203	0.4739	17.00000
Consol. 200y	0.84213	1.31573	0.84213	0.4736	16.64782
Installation					
Consol. 10y	0.83487	1.22770	0.93743	0.39531	10.99090
Consol. 100y	0.89348	1.16569	0.94083	0.28987	10.53766

F Sensitivity analysis

Commonly a sensitivity analysis is performed to see how much influence different variables, for example mesh size ,loading or soil parameters, have on the result. The way the sensitivity analysis is conducted in this thesis is slightly different since there is no measurements that could work as reference to the model results. The behaviour of overlapping piles is being evaluated mostly by comparing it to other systems of floating piles. If some model parameters would change, all different models would be affected in a similar manner. Hence the sensitivity analysis is done already when the model parameters was adjusted to fit the available triaxial and CRS tests. The model variables are tested to see how they influence the results from the soil test facility and the summary can be found in table F.1.

Table F.1: Parameters through depth

Parameters	**Symb.**	**Ref. value**	**Modified values**		**Change**
			Smaller	Larger	
Modified swelling index	κ^*	0.015	0.005	0.02	Change in inclination of the elastic region
Intrinsic compression index	λ_i^*	0.085	0.075	0.1	Change in the slope of the plastic region and end-point of the re-compression
Initial bonding	χ_0	19	10	28	Minor change slope of plastic region and the creep rate
Absolute rate of destruction	ξ	10	8	12	Significant change to the plastic region
Relative rate of destruction	ξ_d	0.2	-	0.3	Minor change in the plastic region
Intrinsic creep coefficient	μ_i^*	0.006	0.002	0.01	Significant change in the position of the elasto-plastic region
Reference time	τ_d	1	-	3	Minor change in the position of the elasto-plastic region
Pre-overburden pressure	POP	1	-20%	+20%	Change in the position of the elasto-plastic region

A small mesh study is presented in fig. F.1 where it is clear that the mesh size has small effect on the final settlements. A medium sized mesh can be used to save some calculation time but a finer mesh should still be used around and close to the piles in order to have sufficient stress point and nodes.

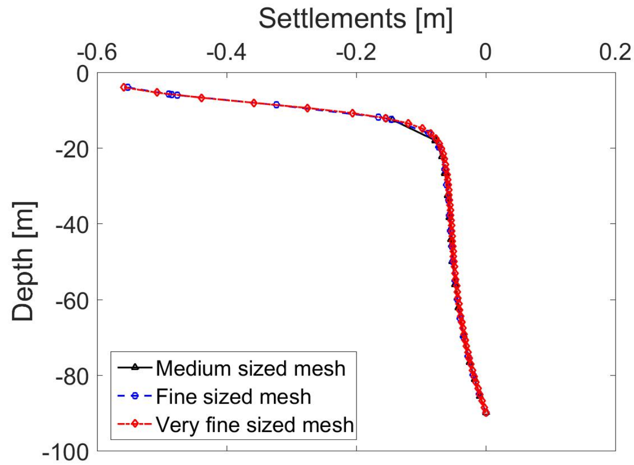

Figure F.1: Three different mesh sizes compared with regards to settlements of the soil.