The installation effects of screwed displacement piles

Testing and numerical modelling

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Master thesis

by

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Preface

Before you lies my master thesis about the effects of the installation of screwed displacements piles. This thesis is done to obtaining my masters degree in Geo-engineering at the Delft University of Technology. The research is done in collaboration with the municipality of Rotterdam. The aim is to predict the soil displacements during installation and its effects on adjacent structures. I was given the opportunity of combining theoretical and practical aspects into one research. Executing tests in the field taught me that the theory is made so much prettier than the actual practice.

Of course I could not have done this project without the help from a lot of people. First off all I would like to thank my committee for their support and advice. Rodriaan Spruit for providing me with a subject for my thesis and showing me how important practical experience is. Kees Blom for our efficient meetings and help with my report. Wout Broere for all the Plaxis solutions and always having time for me when I needed last-minute help. Ken Gavin for his input during the meetings.

For their help and input during the field tests I need to thank Francois Kamminga and Willem van Bommel. I also would like to thank the Port of Rotterdam, Franki Grondtechnieken and SaVe for making the tests possible.

For the support on the sidelines I need to thank my friends and fellow students. You made studying a lot more fun. Finally, and most important of all, I want to thank my family for their support and patience during my entire study.

'Unfortunately, soils are made by nature and not by man,
and the products of nature are always complex...' - Karl von Terzaghi

F.C.M. van Overstraten Kruijsse
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Abstract

During construction in urban areas often noise and vibrations are not tolerated. For the installation of the foundation piles in these cases there is often chosen for screwed displacement piles. These piles can be installed without nuisance for the vicinity, but do induce large soil displacements during installation. The soil displacements can cause increased soil pressure on adjacent structures. Because urban areas are getting more densely built, these problems will occur more often in the future. The effects can be minimilized when they are known in advanced. This research looks into the possibility of predicting the installation effects of screwed displacements piles on adjacent structures. This is divided in the prediction of the soil displacement in a finite element analysis and in the effect of the soil displacement on adjacent piles. This is done with two case studies. First data from tests in Shanghai is used to create a finite element model. After this measurements are done in Rotterdam to verify this model for Dutch cases.

In the harbour of Shanghai there are 2 test piles installed. During and after the installation the soil displacements are measured with inclinometers. These measurements are used to compare with the modelled displacements. With the finite element program Plaxis a 2D axisymmetric model is made for the installation process of the pile. For this model two methods are tested to model the installation process. First a prescribed displacement is applied to the edge of the pile volume with the magnitude of the radius of the pile. This method resulted in displacements that overpredict the displacements. The other method is to apply a volumetric strain to the pile volume. In theory a volumetric strain of 100% is the correct amount. However the modelled displacements underpredict the measured displacements. To get a better fit with the measured displacement, larger volumetric strains are modelled. A volumetric strain gave a better fit with the measured displacement. The modelled displacements are more uniform over depth than the measured displacement, due to the homogeneity of the model. By using the model in drained and undrained conditions as lower and upper limit, the peaks and troughs in the measured displacements are captured between these displacements.

To see if the suggested model also works for Dutch cases, tests are done during the installation of 5 piles in the harbour of Rotterdam. During these tests inclinometers are used tohe measure the soil displacements and the pile displacements. The inclinometer results for the soil displacements are analysed per soil layer. This showed a clear decrease in the soil displacement over the distance from the installed pile. When comparing these displacements with the theoretical maximal possible displacement, which assumes no volume loss in the soil, all displacements are smaller. The displacements in the clay layers is on average 0.5 smaller than the maximum value. However, the deviation from this average value is large, so for this layer it is safer to use the CEM value. The sandy clay layer is on average 0.15 times smaller and the sand layer 0.06. They do not deviate a lot from their average. Using the CEM value for these layers is too conservative.

The measured displacements are used as comparison for a model of the Rotterdam case. The best method from the Shanghai case is used to see if it can be used for other locations as well. The 150% volumetric strain gave a too large modelled displacements. The volumetric strain is decreased to get a better fit with the measured displacements. This showed that it is not possible to find a volumetric strain to fit all soil layers. The soil displacements in the clay layer are best modelled with 100% volumetric strain and the soil displacements in the sandy clay layer and sand layer with 25% volumetric strain. For the 5 piles a 3D model is made to determine the effects of the soil displacements on the previously installed pile. This model did not give results according to the expectations and should be developed further to get accurate predictions.

Combining all the analysis of the measured and modelled displacements showed that the displacement is depended on the stiffness of the soil layers, the initial displacement at the edge of the pile and the distance from the pile. The soil parameters influence the initial displacements in each soil layers. When the initial displacements are correctly determined with tests, it is possible to predict the soil displacement due to the installation of a screw pile with a finite element model. No conclusion could be made on the effect of the soil displacements on adjacent structures.

Contents

List of symbols & Abbreviations

Symbol		Unit
A	Area	m^3
C	Cohesion	N/m^2
D_{pile}	Diameter pile	m
E	stiffness	Pa
$E50_{ref}$	Secant stiffness in standard drained triaxial test	N/m^2
$Eoed_{ref}$	Tangent stiffness for primary oedometer loading	N/m^2
Eur_{ref}	Unloading / reloading stiffness	N/m^2
e_{init}	Initial void ratio	-
F	Force	N
$G0_{ref}$	Reference shear modulus at very small strains	N/m^2
I	Moment of inertia	
k_x	Permeability in x-direction	m/s
k_y	Premeability in y-direction	m/s
L	Pile length	m
P_0	Initial pore pressure	Pa
P_{excess}	Excess pore pressure	Pa
r_i	Radial coordinate	m
r_0	Initial radius	m
R	Plastic radius	m
T	Temperature	°
S_u	Undrained shear strength	Pa
u_r	Radial soil displacement	m
u_x	Horizontal soil displacement	m
u_y	Vertical soil displacement	m
x	X-coordinate	m
y	Y-coordinate	m
ϵ_x	Volumetric strain in x-direction	m/m
u	Volumetric strain in y-direction	m/m
ϵ_v	Volumetric strain	m/m
$\gamma_{0.7}$	Treshold shear strain at which Gs = 0,722G0	-
γ_{sat}	Saturated volumetric weight	N/m^3
γ_{unsat}	Unsaturated volumetric weight	N/m^3
λ	Frequency	Hz
ν	Poisson's ratio	-
ϕ	Friction angle	°
ψ	Dilatancy angle	°

Abbreviation	
CEM	Cavity expansion method
CPT	Cone penetration test
HB	Hellingsbuis (Inclinometer)
HSS	Hardening small strain
MC	Mohr-Coulom
SPM	Strain path method
SSPM	Shallow strain path method

1 Introduction

Cities are very densely populated and this is still increasing. Not only the population is growing, also industry and tourism have a positive growth rate. To meet all the new demands of a growing city, new buildings, infrastructure and other facilities are required. This is a challenge not only for the available space at surface level, but also for the underground space. If there is space at or above the surface, it does not automatically imply that there is space below the surface as well. Tunnels, foundation, ground anchors, sewers and other obstacles are present everywhere in the subsurface of a city.

A good example where there was enough space above surface but where it has become crowded below surface level is ”Het Collectiegebouw” in Rotterdam (Figure 1.1). Het Collectiegebouw is meant to store the collection which is not being exhibited of Museum Boijmans Van Beuningen. The most logical location is next to the museum, where a large open space is located. However, below a part of this open space is a parking garage. The foundation of the building is at the closest point around 5 meters apart from the parking garage.

Figure 1.1: Het collectiegebouw (MVRDV, 2019)

It seems like Het Collectiegebouw just fits in the unused space of the garage, but with the installation of the foundation (screwed displacement piles), large horizontal displacements of the soil occurred. This could cause large forces on the construction of the garage and already installed piles. After this was observed, the construction was stopped until proper measures were taken.

Another problem at Het Collectiegebouw is the small floor area compared to the area of the higher levels of the building. This results in a relatively small area in which the load onto the bearing layers is concentrated, resulting in a lot of piles to be installed. When installing a new pile close to already constructed piles, the horizontal deformations will give an extra lateral load on the already constructed pile. This is not unique to this project, but will occur more often in the future. Buildings are getting higher and higher, so more piles are needed to distribute the load.

1.1. Problem

The displacements of the piles that were measured at Het Collectiegebouw were up to 40 cm. This magnitude of displacements is of such a large order that it could have effect on the bearing capacity of the foundation. Either the capacity of the piles decreases due to the tilted piles, or the piles are not positioned correctly anymore to be connected with the super structure. If the effect of the pile installation are known beforehand, it is possible to determine if it could cause problems for the construction. If problems are expected, it is possible to pre-drill the piles. With pre-drilling the soil is removed from the layers that do not contribute to the bearing capacity of the pile. However, pre-drilling will add a lot of cost to the project. An extra machine an augur, is needed to remove the soil. The soil needs to be transported away from the building site and it will take more time. Pre-drilling just in case, is not a financially suitable solution.

The current guideline for installing screwed displacement piles gives an indication of a minimum centre-to-centre distance of the piles (SBR, 2010). When waiting a day between installations, the minimum distance is 2.25-2.5 times the pile diameter. When more piles need to be installed in a day the minimum distance has to be 4 times the pile diameter. When installing the piles at smaller distances the pile may not reach the desired depth, due to higher resistance of the soil. There is no mention of the extra lateral loads on the piles and surroundings when installing the piles with small centre-to-centre distance. These numbers are only meant for the driveability of the piles, not for limiting the soil displacements. The absence of a guideline or proper predictions for the soil displacement results in contractors acting on there own assessment. Because of the additional costs of the measures against the soil displacement could cause contractors to take a risk and not use any measures.

1.2. Goal

The goal of this thesis is to make a model that predicts the effects of the installation of a screwed pile on the surrounding of the pile. This will be done using the Finite Element program Plaxis. Both the soil displacement and the pile deformation will be looked into.

The research questions are:

Main research question
Can the installation effects of a screw pile be predicted with a finite element analysis?

Sub research questions

- What is the soil displacement as a result of the installation of a screw pile and how is this modelled?
- What is the effect of the soil displacement on adjacent structures and can this be modelled?

1.3. Strategy

This thesis will take the following steps to answer the research questions:

Literature study
The first step is to look up what is already known about this subject. It is important to understand what happens with the soil around the pile during and after installation.

Modelling of one pile in Plaxis 2D
An axisymmetric model of one pile will be used to model the installation process. Different soil displacement methods will be compared with data from Shanghai to determine which one has the best fit with the data.

Field test in the Harbour of Rotterdam
To see whether the model that is made with the measurements from a test in Shanghai are also usable in The Netherlands, tests will be executed in the harbour of Rotterdam to use as a verification of the model.

Modelling of a pile group in Plaxis 3D
The data from the harbour of Rotterdam is used to make a 3D model in Plaxis. 5 piles are modelled in Plaxis and the results will be compared with the test results.

Discussion
With the results from all analysis is the connection between them determined. From this is a recommendation for future projects is made.

Conclusion and Recommendations
The thesis will finish with the conclusions that can be drawn from the different analysis done. From this, recommendations will be made for further research.

1.4. Boundary conditions

All displacement piles will cause problems with the effects of their installation. This thesis will only focus on the effects of the screw piles.
The model created to predict the soil and pile displacement, will only focus on modelling the displacements correctly. Whether the capacity of the modelled piles are correct as well, will not be part of this research.
The model should give realistic values for the displacement, but should be an user friendly and efficient model.
The research is focused on the applicability of this model on Dutch sub soils, with a special interest in the use in the city and harbour of Rotterdam.

2

Literature Study

Many pile foundations systems are developed for different situations. The screw pile is mostly used in urban areas, where nuisance during the installation should be minimilized. Screw piles are already being used since before 1950, but after 1950 several companies started to develop the technique further to the installation techniques we now know (Marinucci and Chiarabelli, 2016). The piles were given different names in Europe and in North-America. In Europe the piles are called Screw piles, as in this report, but in North-America the piles are called Drilled Displacement piles. In this chapter is explained how screw piles are installed and what happens with the soil around the pile during installation. Also methods to predict the soil behaviour around the pile will be discussed.

2.1. Installation of screw piles

The screw pile is the vibration and noise free alternative for driven piles. It is mostly used in populated areas, so there is no nuisance for the surroundings. There are different types of screwed displacement piles(Figure 2.1), but all with the same principle. The tip, with attached tube, is pushed in the soil with a rotational movement. While moving down, the tip pushes the soil to the side. When the desired depth is reached, a reinforcement cage is placed in the tube after which the tube is filled with concrete. While casting the concrete, the casing is retrieved, leaving the tip behind (Figure 2.2). The different screw piles differ from each other in the design of the displacement body. The piles used in this thesis are similar to the SVB, Atlas and Fundex pile.

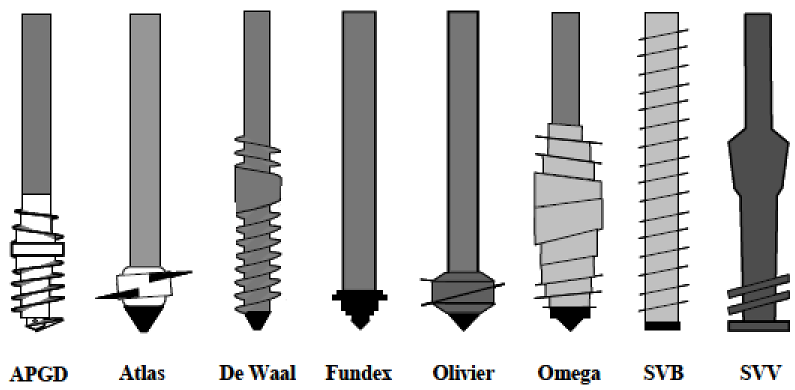

Figure 2.1: Different types of drilled displacement piles (Basu and Prezzi, 2009)

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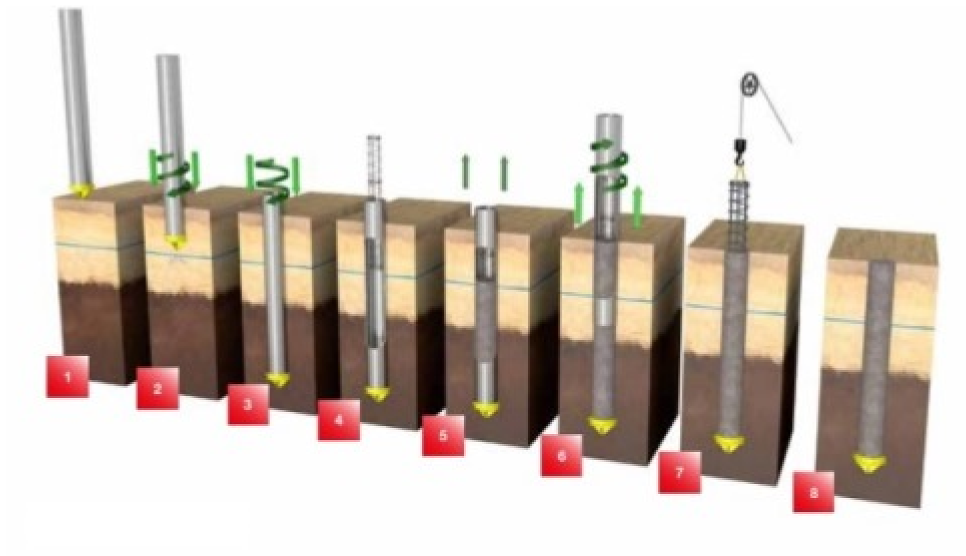


Figure 2.2: Installation process screwed displacement piles (Marinucci and Chiarabeli, 2016)

The screw pile has many advantages over the driven pile, but it also had some disadvantages. Below are the (dis)advantages of the screw pile being summed up (Marinucci and Chiarabeli, 2016).

Advantages:

- Due to the displacement of the soil, higher shaft friction and end bearing capacity can be achieved than with non-displacement piles (bored piles).
- The installation causes low vibration and noise, so is suitable for crowded areas.
- It is environmentally friendlier than soil replacement piles. No soil needs to be transported, less concrete is necessary and no bentonite spoil.

Disadvantages:

- Because of the soil displacement the horizontal soil pressure in the vicinity of the piles increase significantly.
- Screwed displacement piles are usually chosen for crowded areas because of their low vibration impact, but at these places there is less tolerance for the horizontal deformations of the soil.

The large soil displacements can be mitigated by pre-drilling the soil layers that deliver no positive bearing capacity. With an auger, with the same diameter as the final piles, the soil is removed. Due to the removal of the soil, the soil displacement is minimised over the pre-drilled soil layers. For pre-drilling an extra machine is required and the excess soil needs to be transported, so it is more expensive and time costly than installing the piles without pre-drilling.

2.2. Soil displacement

According to Massarsch and Wersäll (2013) the displacement fields around a pile can be divided in 6 areas (Figure 2.3).

1) Around the tip of the pile is an elliptical zone with disturbed soil. Ni et al. (2010) determined the area with laboratory tests and Particle image velocimetry analysis. The path of a single soil grain around the pile tip is described by (Baligh, 1985) with the Strain Path Method. The dominant direction is horizontal, but the grains also experience different vertical displacements.
2) Around the whole perimeter of the pile is a smear zone. In this zone the entire soil structure is disturbed. The width of this zone is thin and independent of the diameter of the pile.
3) Massarsch (1976) determined that a zone, up to one diameter around the shaft, is highly disturbed by the

installation of the pile. In this zone the undrained shear strength is reduced. The dominant soil displacement is in a horizontal direction.
4) This zone gives passive earth pressure against the elliptical zone around the pile tip while the pile is installed to its desired depth. The displaced volume of the pile is pushed in this zone, which results in dominant horizontal displacements. This zone, however, has also vertical soil displacements. The closer to the surface, the larger the vertical movements are.
5) Near the surface the resistance in the horizontal direction is larger than the vertical direction, so the soil moves dominantly in the vertical direction. This results in surface heave around the installed pile.
6) When the pile is installed, the downward movement is dragging down the soil directly adjacent to the pile. This results in a small gap between the pile and the soil at the surface.

For each zone, different methods have been developed to predict the soil movements. The methods for zone 1, 4 and 5 are further elaborated, because these zones are the most important for this research. Zone 2,3 and 6 are in such a close distance to the pile, that this behaviour will not have any influence on adjacent structures.

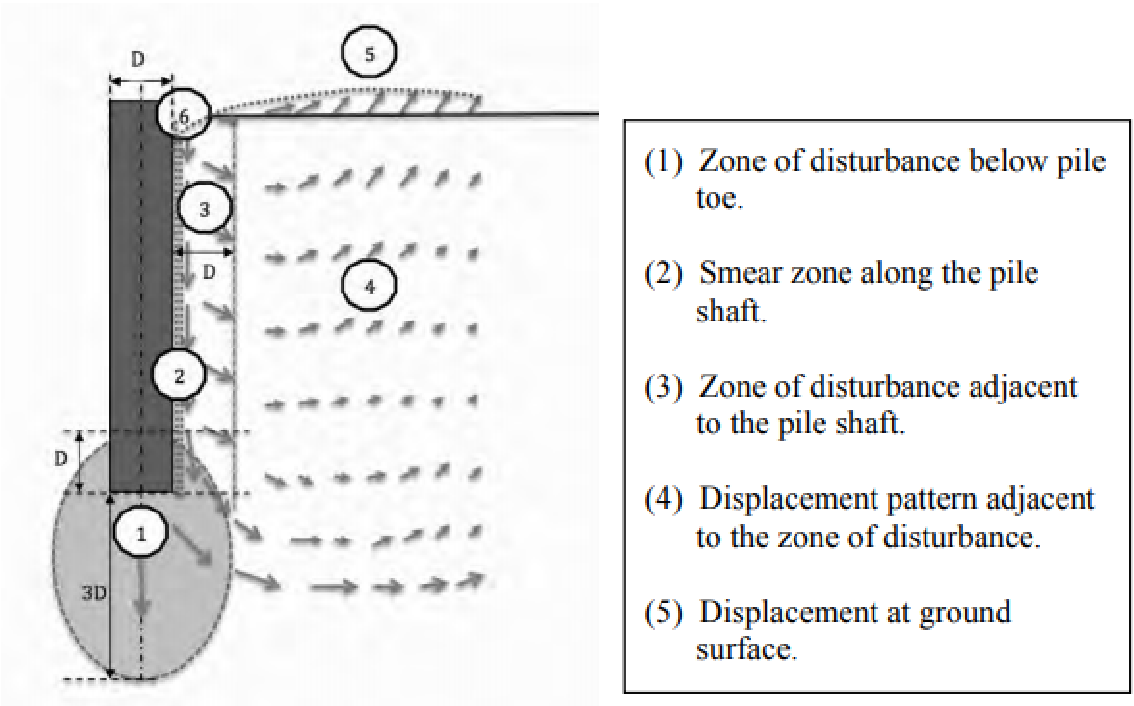


Figure 2.3: Schematic illustration of the displacement field and zones of disturbance during pile installation. (Massarsch and Wersäll, 2013)

2.2.1. Cavity Expansion Method

For the horizontal displacement around the mid-height of the shaft (zone 4), the Cavity Expansion Method (CEM) has been defined. The CEM is about the expansion of an already existing cavity. It is derived from the pressuremeter test, which measures the shear strength of a soil. By expanding an existing cavity, the relation between the applied pressure and the increasing volume of the cavity is retrieved. Palmer (1971) uses this correlation to create the stress strain relation of a soil, which is usable for the Cavity Expansion Method. The CEM assumes a plain strain in an axial symmetrical situation. This gives a 1D prediction of the soil movement on the axis of the radius. Near to the expanding cavity, the soil is heavily disturbed and experiences plastic strain. Around the plastic zone there is a zone which only experiences elastic strain(Figure 2.4). The deformations in the elastic part are considered to be negligible small, so it is assumed that only in the plastic part soil deformations occur (Carter et al., 1986).

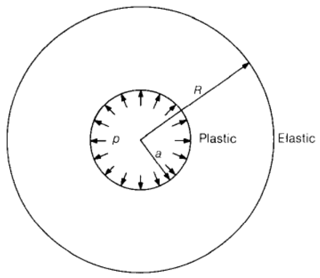


Figure 2.4: Zones around cavity expansion (Carter et al., 1986)

Yu (2000) determined a solution to calculate the plastic radius for the installation of a pile. Which starts from zero radius and is expanded in an infinite medium.

$$R = a\left[\frac{E}{Em + 2(1-m)(1+\nu)S_u}\right]^{\frac{1}{2(1-m)}} \tag{2.1}$$

$$\text{With} \qquad m = \frac{2(1+\nu)(1-2\nu)S_u}{E} \tag{2.2}$$

When installing a pile there is no pre-existing cavity, so the CEM is not fully applicable for pile installation. However, Carter et al. (1979) have described how the CEM can be used for pile installation. They have showed that the consolidation process around a pile is the same for an expansion from 0 to r0 as compared to an expansion from 0.575r0 to 1.15r0 (Figure 2.5).

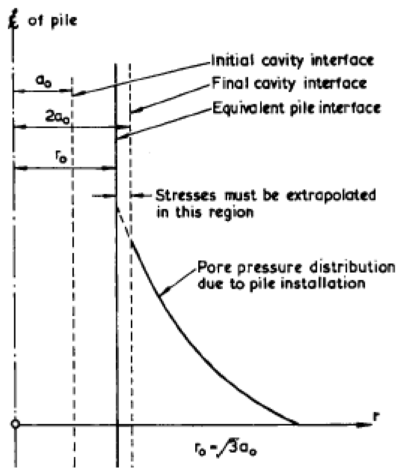


Figure 2.5: Pile installation vs Cavity expansion (Carter et al., 1979)

Randolph et al. (1979) conducted laboratory tests with closed-ended piles to measure the soil displacement around the pile after installation. They matched the measured data with their theoretical equation (Equation 2.3) deduced from the CEM method. For this theory it is assumed that the soil is incompressible and undrained, so the volume stays constant. The measured soil displacement at the tip of the pile and at mid-height of the pile matches reasonably well with the equation, but the soil at 2r below the surface has lower measured displacements than the suggested equation. This is due to vertical movement of the soil, which is not taken into account in this method.

$$\frac{u_r}{r_0} = \left(1 + \frac{r_i^2}{r_0^2}\right)^{1/2} - \frac{r_i}{r_0} \tag{2.3}$$

2.2.2. Strain Path Method

For the soil movement around the tip of the pile (zone 1), Baligh (1985) developed the Strain Path Method (SPM). During tests the path of the soil has been monitored. These test have shown that a particle is initially pushed down and after the tip of the piles has passed, returns to its original height (Figure 2.6). This proves that the soil indeed experiences also displacements in the vertical direction. This method is only applicable for deep penetrations, so it cannot be used near the surface. This is because of the stress free conditions at the surface, which are not accounted for in this model.

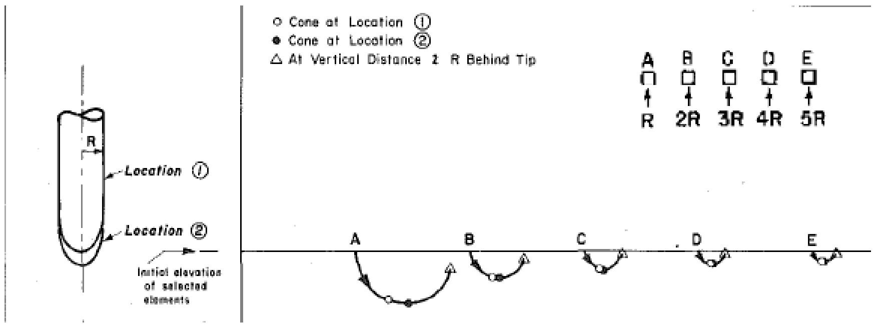


Figure 2.6: Particle movement around the tip of a displacement pile according to the SPM (Baligh, 1985)

2.2.3. Surface heave

Sagaseta and Whittle (2001) developed the SPM further to the Shallow Strain Path Method (SSPM), which is applicable for near surface soil displacements, because it takes the stress free surface into account. The free surface increases the complexity significantly. Solutions from fluid mechanics are adopted to solve this problem. The method uses sinks to represent volume loss or gain in the soil. In case of displacement piles volume is added to the soil. The analysis takes 3 steps. In step 1 the free surface is neglected and a sink is placed in an infinite medium. In step 2 a positive or negative image sink is placed on the opposite side of the surfaces. This is to cancel the stresses and strains on the surface, which are created by step 1. In step 3 the remaining stresses or strains (depending if a positive or negative image sink is used) are evaluated and removed. As result, a realistic representation of the soil situation is obtained. This method can be adapted for different situations in the soil; tunnelling, sheet pile removal, pile driving etc.. For pile driving, an equation for the horizontal and vertical soil movement at surface level is determined by Sagaseta et al. (1997) (Equation 2.4 and 2.5).

$$u_y(r,0) = \frac{r_0^2}{2} * \frac{L}{r * \sqrt{r^2 + L^2}} \tag{2.4}$$

$$u_r(r,0) = -\frac{r_0^2}{2} * \left(\frac{1}{r} - \frac{1}{\sqrt{r^2 + L^2}}\right) \tag{2.5}$$

The previously mentioned methods predicts at the soil movement at a very local level. When regarding the movement around the whole pile, two processes act simultaneously; 1) The drag around the pile. 2) The cavity expansion (Chong, 2013). During pile installation, large shear strain around the pile drags the soil around the pile downward. At the same time the soil is laterally displaced, which also results in a vertical movement upward, because that is the way of least resistance. These 2 components result in a soil movement as shown in Figure 2.7.

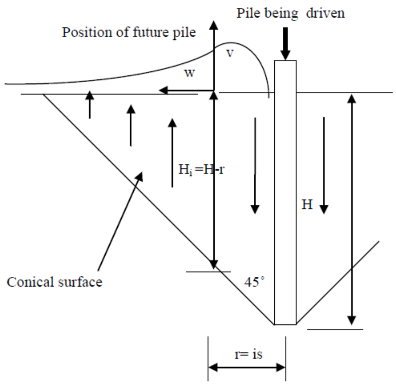


Figure 2.7: Soil movement around a displacement pile (Chong, 2013)

2.2.4. Influence on adjacent piles

The displacements induced by the pile installation can be of great influence on previously placed piles. Depending on the distance between the piles, the previously installed pile can displace due to the installation of a new pile and encounter extra loads (Figure 2.8). Due to the vertical soil displacement around the pile the adjacent pile could heave as well, which reduces the bearing capacity of the pile. It can also result in (ex)tension cracks if insufficient reinforcement is present in the previously installed pile.

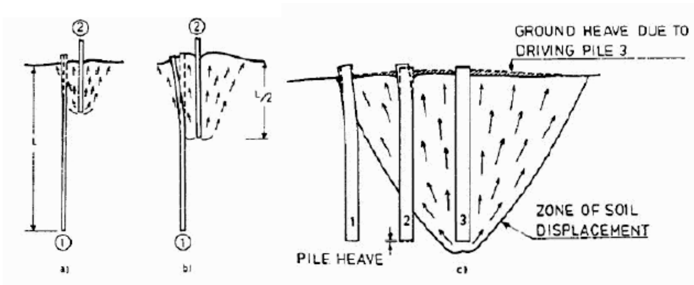


Figure 2.8: Displacement of adjacent piles (Hagerty and Peck, 1971)

Oostveen and Kuppers (1985) measured this phenomenon during the construction of the airport in Bagdad. Several piles were installed at different distances to measure heave of the surface and the pilehead. For the heave of the surface they developed a analytical solution, which is depended on the compressibility of the soil, porosity, saturation and pore pressure. The measured surface heave fits the calculated heave. Also a clear heave is measured in the pile heads. The maximum heave was 28 mm at an R/R0 distance of 6.

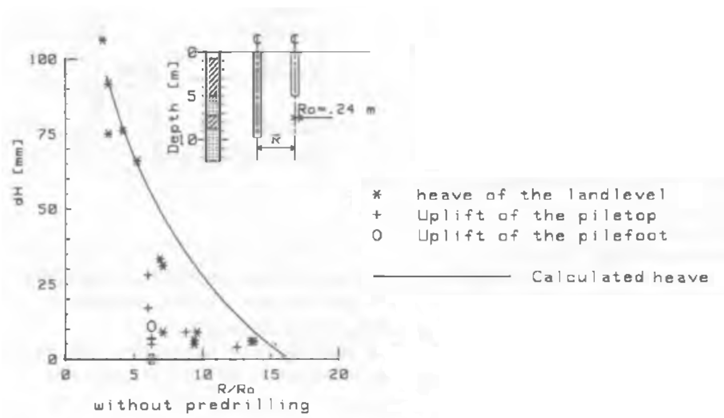


Figure 2.9: Pile heave of adjacent piles Bagdad (Oostveen and Kuppers, 1985)

Besides the heave of the adjacent piles, the soil displacements also cause extra horizontal stresses in the adjacent piles. There are many different situations in which piles are subjected to soil displacement; failing or creeping slopes, adjacent embankments, pile installation. There is not a single method to describe the pile interaction with the soil displacement for all these situations (Chen and Poulos, 1997). To measure the horizontal load that moving soil applies to piles, Pan et al. (2002) did laboratory tests. A fixed pile is placed in a box filled with stiff clay. The box with the soil is displaced so the soil moves against the pile. The applied soil pressure on the pile versus the movement of the box was monitored (Figure 2.10). They concluded that the maximum soil pressure on a passive pile is 10 times the undrained shear strength of the soil.

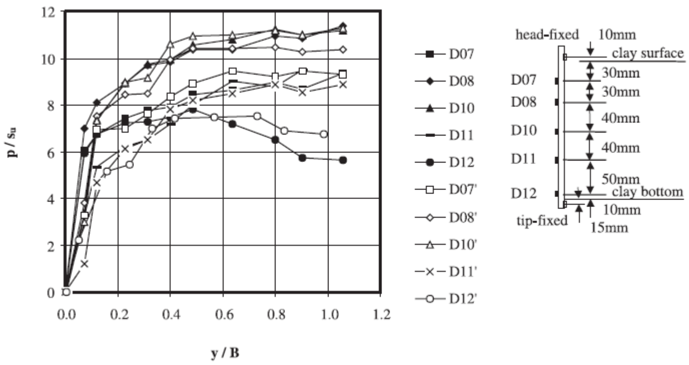


Figure 2.10: Normalised soil pressure vs soil displacement (Pan et al., 2002)

3

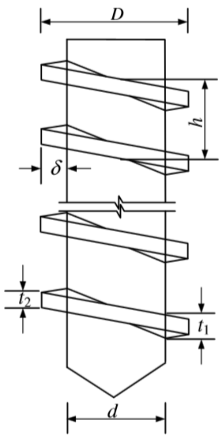
Case study harbour of Shanghai

In this chapter the possibilities of modelling the installation effects of a screw pile are discussed. For the verification of the model, field measurements are used from a case study in the harbour in Shanghai. First these measurements will be described. Thereafter the different modelling methods will be described for the installation of a screw pile. The result of these methods are compared with the measured displacement and a recommendation is made for the best method to use.

3.1. Measurements Harbour of Shanghai

In the harbour of Shanghai, Meng et al. (2015) executed soil displacement measurements during the installation of two screw piles. The goal of these tests was to measure the effect of the installation of screw piles. Two piles, DSP1 and DSP2, were installed with a distance of 25 m between them.

(a) Excavated test pile



(b) Schematic

Figure 3.1: Design test piles (Meng et al., 2015)

The piles used for these field tests are a newly developed type of pile in China. The system is a combination of the SVB-pile and the Atlas-pile (see section § 2.1). The pile has a thread along the entire length of the tube, but with a thicker thread than the normal SVB pile (Figure 3.1). The imprint of the thread of the pile will remain in the soil, like what happens with an Atlas pile. When the tube is retrieved, the remaining cavity is filled with concrete. The diameter of the tip is 370 mm and the diameter of the thread on the tube is 500 mm. The effective length of the pile is 20.4 meters.

The subsoil at the test location exist mostly out of layers with very porous clay (Figure 3.2). The properties of the different layers are determined with field and laboratory tests. The grondwater is at a level of -1.4 m below surface level.

Number	Soil layer	H (m)	γ (kN/m^3)	e	w_n (%)	w_l (%)	w_p (%)	c_{cu} (kPa)	ϕ_{cu} (°)	E_s (MPa)	q_c (MPa)	k_v (cm/s)	k_h (cm/s)
1	Filled soil	1.6	—	—	—	—	—	—	—	—	—	—	—
2	Clay	0.9	18.6	0.89	30.7	40.8	21.4	20	18.0	4.21	0.85	1.4×10^{-7}	3.0×10^{-7}
3	Silty clay	4.6	17.5	1.13	39.8	36.2	20.8	10	20.5	2.6	1.09	1.8×10^{-6}	6.9×10^{-6}
4	Very softsilty clay	8.7	16.6	1.43	50.4	44.4	23.5	12	15.6	1.89	0.51	2.4×10^{-7}	6.2×10^{-7}
5	Clay	3.5	17.9	1.05	36.6	40.5	21.9	9	16.2	3.47	1.14	1.4×10^{-6}	2.5×10^{-6}
6	Hard clay	6.7	18.3	0.93	32.1	36.8	20.7	14	20.5	7.71	3.33	4.3×10^{-7}	8.3×10^{-7}
7-1	Sandy silt	8.9	20.0	0.63	21.4	36.6	18.6	—	—	12.93	10.4	4.9×10^{-4}	7.5×10^{-4}

Note: c_{cu} = cohesive strength from CU test; E_s = deformation modulus from Oedometer test (100–200 kPa); e = void ratio; H = soil thickness; k_v, k_h = vertical and horizontal permeability coefficients; q_c = cone tip resistance of cone penetration test; w_l = liquid limit; w_n = water content; w_p = plastic limit; ϕ_{cu} = friction angle from CU test; γ = unit weight, respectively.

Figure 3.2: Soil properties Shanghai

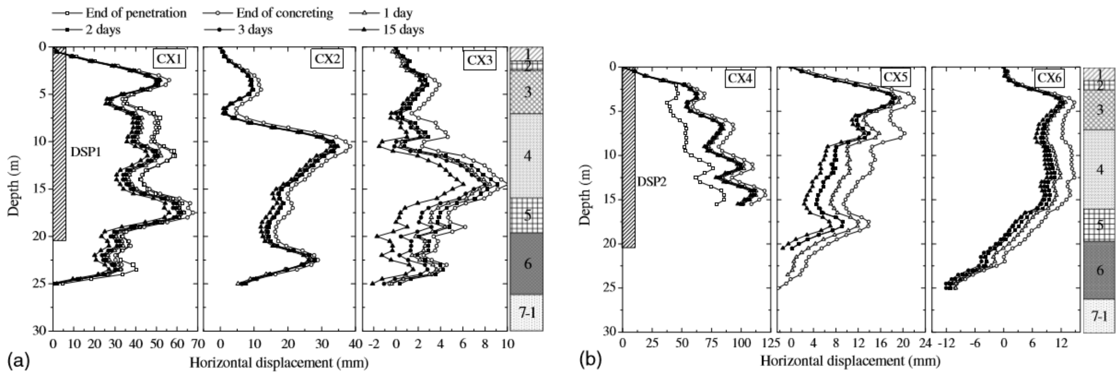
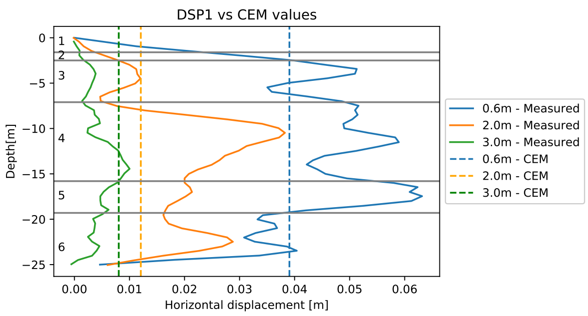
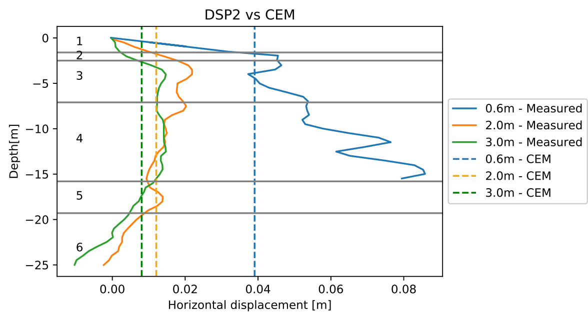


Figure 3.3: Horizontal soil displacement field tests Shanghai harbour (Meng et al., 2015)
a) Displacement pile 1 b) Displacement pile 2

Both piles had three inclinometers at distances of 0.6 m, 2.0 m and 3.0 m. The inclinometers were measured at different times during and after installation of the piles (Figure 3.3). Over the time of 15 days they also measured the pore pressures at different distances and depth. The measured displacement shows a clear displacement in the soil. The displacements decrease the farther away from the pile, as was suggested in the literature. However, the amplitudes of the displacement does not match with the discussed Cavity Expansion Method (subsection § 2.2.1). This makes the assumption that the soil has a constant volume while it is being pushed into the adjacent soil. It is expected that in reality the soil volume does not remain constant, because the soil compresses due to the extra pressure. It depends on the soil parameters and drained behaviour of the soil how much a soil compresses. Because of this the measured displacements should be equal or smaller than the CEM value. In the case of Shanghai, most displacements are larger than the CEM values (Figure 3.4). This suggest that more volume is displaced than only the volume of the pile. From the measured displacement it is also visible that the displacements are not uniformly distributed along the depth of the pile. This suggests that soil parameters indeed play a role in the size of the soil displacements.



(a) DSP1



(b) DSP2

Figure 3.4: Measured displacement vs Cavity Expansion Method values
Soil layers indicated with numbers

3.2. 2D axisymmetric model

3.2.1. Method

Phases

It is not possible in Plaxis to model the fluent and continuous process of the installation of the tube (Dijkstra et al., 2006). So the installation process needs to be modelled in steps. Engin et al. (2015) used the Press-Replace technique for the installation phase. The PR technique divides the pile in small horizontal layers. First the soil is displaced in one step (Press) and then soil is replaced with pile material (Replace). This is continued until all steps are pressed and replaced. This method, however, takes 6 hours calculation time per meter penetration. One of the boundary conditions of this research is a user-friendly model for a group of piles. The PR technique would take too much time for a group of piles, so it is decided not to use it. Broere and van Tol (2006) used just one step for the soil displacement phase, which is a more time efficient methods. For this reason, this will also be used in this research.

For this problem an axisymmetric model is chosen, because the soil displacements around the pile is expected to be the same in all directions. The modelling of the installation of one pile is divided into four phases (Figure 3.5).
1) The first phase is the initial phase in which the initial stresses with the K0 procedure are calculated.
2) The second phase is the expansion phase. It models the installation of the pile in one step.
3) In the third phase the pile is modelled with reinforced concrete.
4) The final phase is to model the consolidation of the soil around the pile over 15 days.

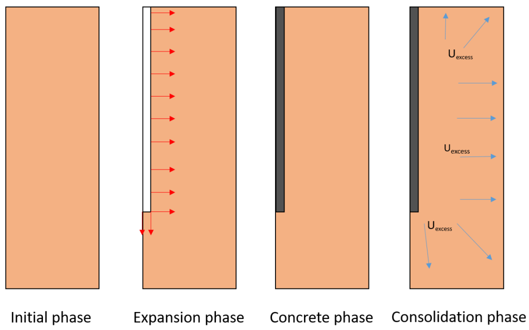


Figure 3.5: Schematic representation of the calculation phases

Soil parameters

For this model the hardening soil small strain model (HSS model) is chosen to be used. The HSS model uses stress depended stiffness, which gives a more realistic soil response than the Mohr-Coulomb soil model (MC model). Large displacements are expected near the installed pile, but farther away there are also small strain expected, which are modelled the best with HSS model. The soil properties for the different soil layers are obtained from Meng et al. (2015). Not all the parameters required for the HSS model are retrievable from the publication. Wang et al. (2017) did research for the parameters for the HSS model in the Shanghai clay layers, so the missing parameters are supplemented with the results from Wang et al. (2017). All the parameters used are noted in Appendix A.

	Soil layer	H [m]
1	Filled soil	1.6
2	Clay	0.9
3	Silty clay	4.6
4	Very sof silty clay	8.7
5	Clay	3.5
6	Hard clay	6.7
7	Sandy silt	8.9

Dimensions and structures

The model has a dimension of 20 m width and 30 m depth. The dimensions are taken extra large, so the boundaries will not have influences on the displacements. The pile has a diameter of 0.44m and a length of 20.4m. The chosen diameter is the average of the tube diameter and outer diameter of the thread (D and d in

Figure 3.1b). To determine the soil displacements in the soil, plates are used. These are placed at the same distance as the inclinometers used during the field tests (0.6m, 2m and 3m).

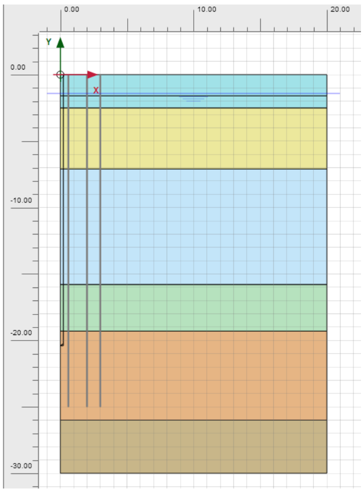


Figure 3.6: Dimensions Shanghai model

Boundary conditions

All the boundary conditions used are the default conditions. This means that the displacements in the x-direction in the vertical boundaries are fixed. The displacements in the y-direction are free. For the horizontal boundaries there is a difference between the top and bottom boundary. At the top boundary (surface level) all the displacements are free. For the bottom boundary all the displacements are fixed. The waterflow is open in all direction except in the bottom horizontal boundary (Plaxis, 2018).

Mesh

For the elements used in the 2D model there are two options. The element can have either 6 nodes or 15 nodes. The 15-node element is a 4^{th} order interpolation for displacements. The 6-node element is only a 2^{nd} order interpolation. This results in more accurate results when using the 15-node elements, so this is used in the model. The size of the mesh is medium. The generated mesh is shown in Figure 3.7. The model has 1467 elements and 11987 nodes.

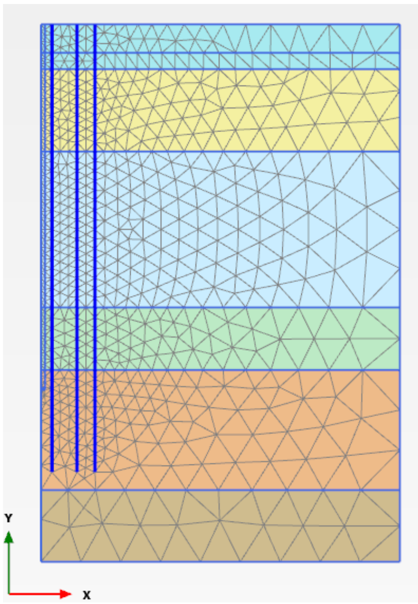


Figure 3.7: Mesh Shanghai model

Modelling expansion phase

The expansion of the soil can be modelled in two ways. Either with a prescribed displacement or with a volumetric strain. For the prescribed displacement two options have been modelled. 1) A displacement with the length of the radius at the edge of the pile. 2) the CEM of Carter et al. (1979)(see subsection § 2.2.1). All two methods are implemented and compared with data from Shanghai to see which method fits the best. Figure 3.8 shows all the explored methods, which are further explained in this chapter. Both drained and undrained conditions have been used for all methods. The installation of the pile took less than 15 minutes. Thus it could be assumed that the response of the soil is undrained. On the other hand the soil is very porous, so this might result in a drained response.

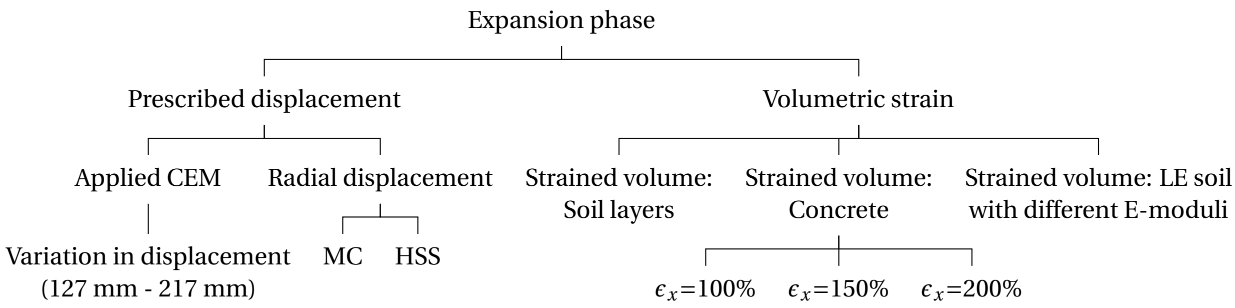


Figure 3.8: Explored options

3.2.2. Results

Prescribed displacement

With a prescribed displacement a line or point is displaced in a prescribed direction and a prescribed distance. The prescribed displacement is used in two methods. In the first method the prescribed displacement is applied to the edge of the pile with a distance of the piles radius. During installation all the soil in the volume of the pile is pushed to the side. At the edge this is done with the magnitude of the radius of the pile. The other method uses the CEM method of Randolph and Wroth (1979) as discussed in subsection § 2.2.1. Both methods are further discussed and analysed in this section.

Radius prescribed displacement

The expansion phase is modelled by horizontal and vertical prescribed displacement at the edge of the later location of the pile (Figure 3.9). The horizontal prescribed displacement is equal to the radius of the pile (220 mm), this is is similar to the full displacement of the pile due to installation. The vertical displacement is set to 0.5 m, this is to model the tip being pushed into the soil.

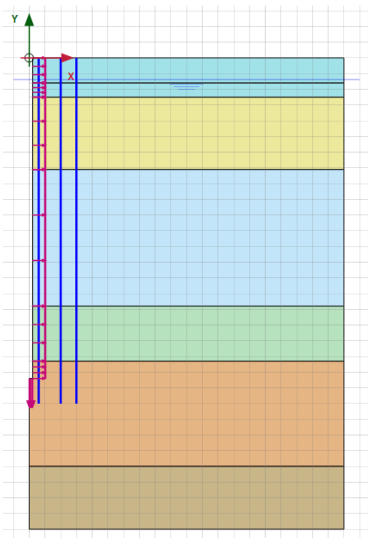
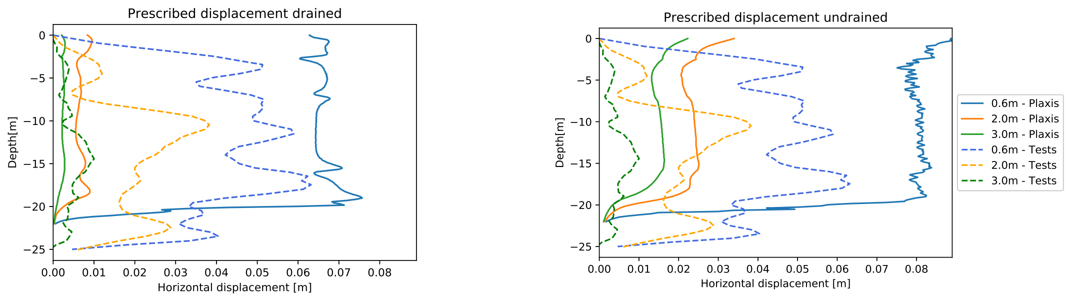


Figure 3.9: Prescribed displacement

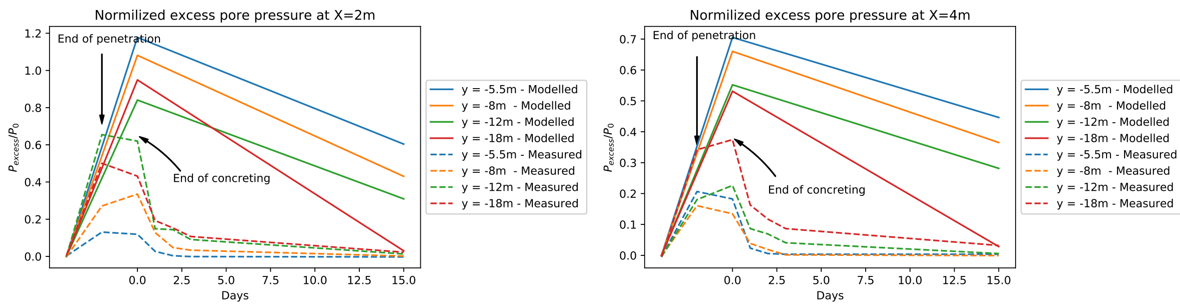
The model has been run in drained and undrained conditions. The results from these models have been compared to the measured field data (Figure 3.10a and 3.10b). The displacements of the drained model are an overprediction for the nearby displacements, but too low for the measurements farther away from the pile. The drained model shows different displacement in each soil layers. The displacements in the undrained model are a lot higher and similar for all the different soil layers, which is easily explained by the constant volume of the soils. The undrained model grossly overpredicts the displacements.



(a) Drained (b) Undrained

Figure 3.10: Field data compared with prescribed displacement method

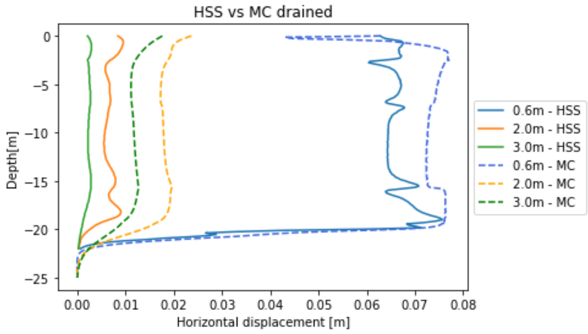
Besides the soil displacement, Meng et al. (2015) also measured the water pressures during installation of the piles. The excess pore pressures were monitored for 15 days. In the field almost all the excess pore pressures dissipated over those 15 days, due to the very porous soil layers. In the Plaxis model this is only the case in the deeper soil layer (-18m) . In all the other layers the excess pore pressure is still present (Figure 3.11). Also the pore pressures right after installation are much higher than the measured excess pore pressures. Because pore pressures are difficult to measure correctly and information on how they were measured is absent, no conclusions can be drawn from comparing the results with the Plaxis model.



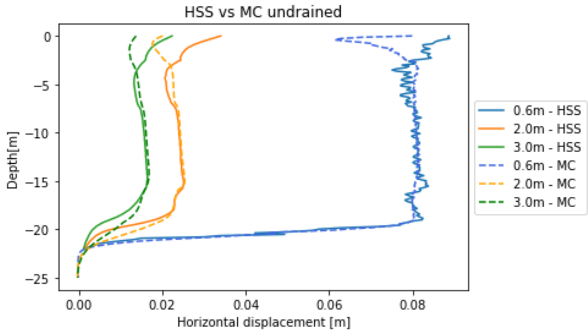
(a) X=2m (b) X=4m

Figure 3.11: Normalised excess pore pressure of modelled and measured data

The 2D model computes fast using the HSS model. It only takes a few seconds to run. However,the expectation is that it will give long run times in the 3D model. Especially when several piles are modelled. Using the Mohr-Coulomb (MC) soil model instead of the HSS model tends to give shorter run times. This is a linear elastic perfect plastic model and is often used as a preliminary analysis of a problem. The expectation is that the displacement will be larger with the MC model, because it has no stress depended stiffness. To see the effect of the MC model it is plotted against the results of the HSS model (Figure 3.12). This has been looked at for both drained and undrained conditions. In drained conditions the soil displacements with the MC model are indeed larger than the displacements with the HSS model. In undrained conditions the soil displacements are almost similar between the two soil models. This is due to the constant volume of the soil in the undrained conditions, so the stress depended stiffness does not play a role. Another effect of the MC model is displacements at larger distance from the pile. The constant stiffness of the soil causes less damping in the soil, which generates displacements to continue for a larger distance. This is visible in the model, because the MC displacements are only 1.3 times the HSS displacements at 0.6m distance from the pile, but the MC displacements are 6 times the HSS displacements at 3m. The displacements from the MC model could be used as a conservative estimation of the soil displacement if the time savings are significant.



(a) Drained



(b) Undrained

Figure 3.12: Mohr-Coulomb vs Hardening Small strain modelled displacements

Applied CEM

In subsection § 2.2.1 the adapted CEM by Randolph and Wroth (1979) for pile installation has been discussed. They state that expanding an existing cavity of $0.575r_0$ to $1.15r_0$ is similar to creating a cavity from 0 to r_0. Applying this to the pile in the harbour of Shanghai gives an expansion phase with an existing cavity with an radius of 127mm expanding to a radius of 254mm. For the first analysis this method is only done in undrained conditions, to get a first impression of the modelled displacements. If these results are promising, the method is also run under drained conditions. The modelled soil displacements are shown in Figure 3.13

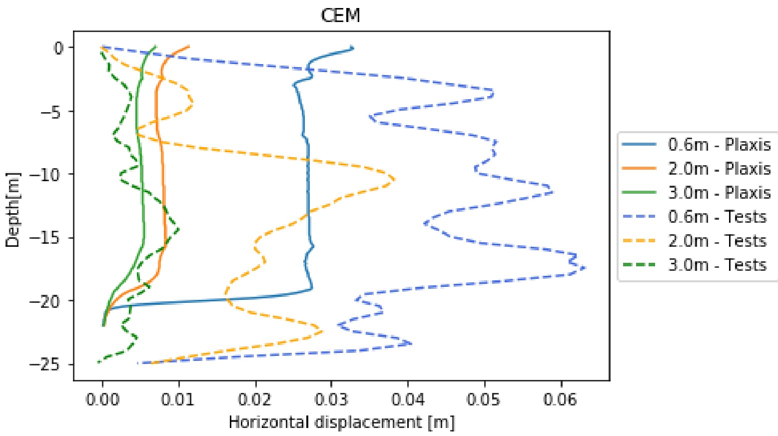


Figure 3.13: Field data compared with applied CEM model

The displacements in the model are less than measured. This could be because this method is for linear elastic soils, so the HSS model will show smaller displacements. The method might be adaptable for HSS model by increasing the prescribed displacement. The prescribed displacement is increased in steps of 10 mm to find a better fit (Figure 3.14). The displacements increase linearly with the increased prescribed displacement. With a prescribed displacement of 217mm, the modelled soil displacement is similar to the average of the measured displacement. With increasing the prescribed displacement, this method looks a lot like the method with a prescribed displacement with the magnitude of the radius. Continuing with this method has no added value, so the drained model or options to optimise the method have not been looked into.

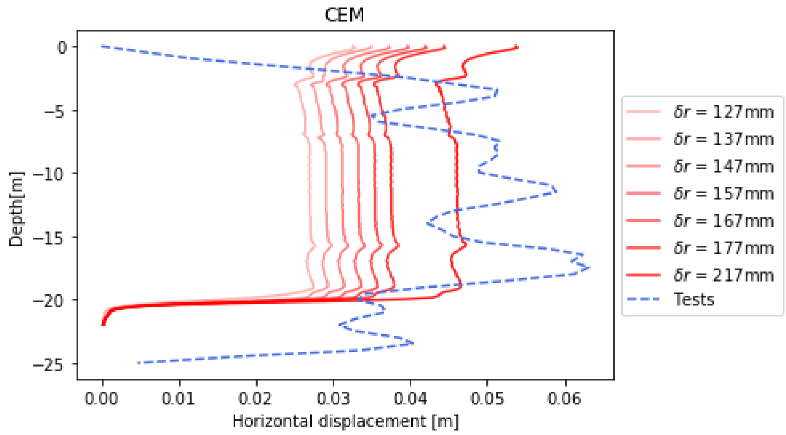


Figure 3.14: Field data compared with different cavity expansions at 0.6m distance undrained

Volumetric strain

In this model the soil is displaced by applying a volumetric strain to a soil volume. The soil volume has the dimensions of the desired pile. During installation of the pile, the entire volume of the soil is being pushed to the side. This could be compared with the volume of the pile having a volumetric strain of 100%. For the volumetric strain it is possible to determine the direction of the strain. In the 2D model this is in either the X or Y-direction or in both. In the initial attempt, the volumetric strain was applied for both directions. However, this caused large deformations in the y-directions, because this direction has less resistance than the X-directions. Because the dominant soil movement around the pile is in horizontal direction, it is chosen to apply the volumetric strain only in X-direction. This has consequences for the deformations around the tip of the pile. Around the tip there is also a dominant movement in the y-direction. This is not modelled when using only volumetric strain in the x-direction. Another problem with the volumetric strain, is that it causes soil failure in the top layer, because at the top near the pile large vertical displacements are generated. To solve this, there is no volumetric strain applied in the top layer. In the field the top layer is strongly influenced by the activities at the surface, so it is always difficult to say what causes the deformations. Beside this, the inclinometers are fixed at the top, so this influences the measuring of the displacements in the top layer as well. For these reasons missing the displacements in the top layers is of no effect on the analysis.

When applying volumetric strain in Plaxis, this is applied with Equation 3.1 to the soil. The strain is translated in to a force generated by the soil volume in the chosen directions of the applied strain. The magnitude of the generated force is depending on the E-modulus of the soil. A low E-modulus generates a small force. It could be that the E-modulus is too low to generate a force, which will not results in the desired displacement. An other consequence is that a difference in E-modulus between soil layers will result in different volumetric strains.

$$\{F\} = \{k\}\{u\} \tag{3.1}$$

In Figure 3.15 the effects on the displacement are shown. Instead of the soil layers the pile volume has a linear elastic material with an given E-modulus. With an E-modulus of 100 kPa, the generated force is so small, that the horizontal displacements are close to zero. With increasing E-modulus, the displacements increase. With an E-modulus of 50.000 kPa a limit seems to be reached and it is expected that the applied strains are generated. When using the soil layers as strained materials, the displacements vary significant from the linear elastic soil with a E-modulus of 50 MPa. In the following models the soil is replaced with concrete, with an E-modulus of 30 GPa. On the concrete the volumetric strain is applied to model the expansion phase. The displacements generated with the concrete are the same as displacements generated with the linear elastic soil with an E-modulus of 50.000 kPa.

The first hypothesis is that in theory a volumetric strain of 100% should represent the installation process. The results of 100% volumetric strain in drained and undrained conditions are shown in Figure 3.16a and 3.17a. It clearly shows that the modelled soil displacements in the drained conditions are smaller than the measured displacements. The same differences between the drained and undrained results are visible as were in the prescribed displacement model. The undrained model has larger and similar displacements over

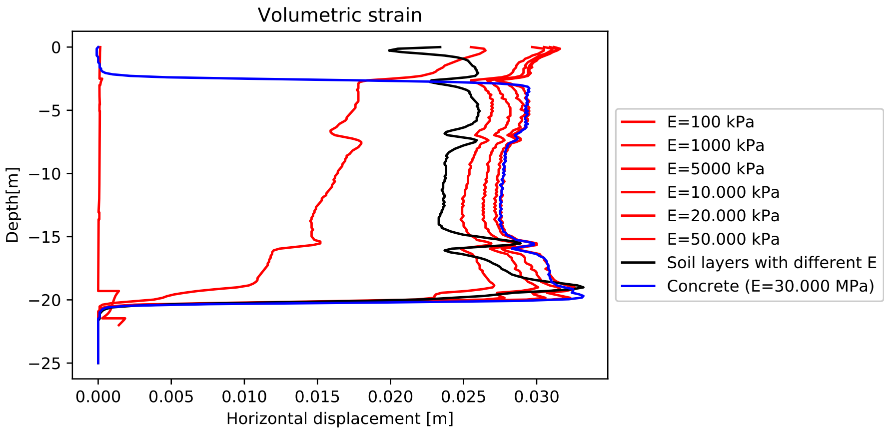
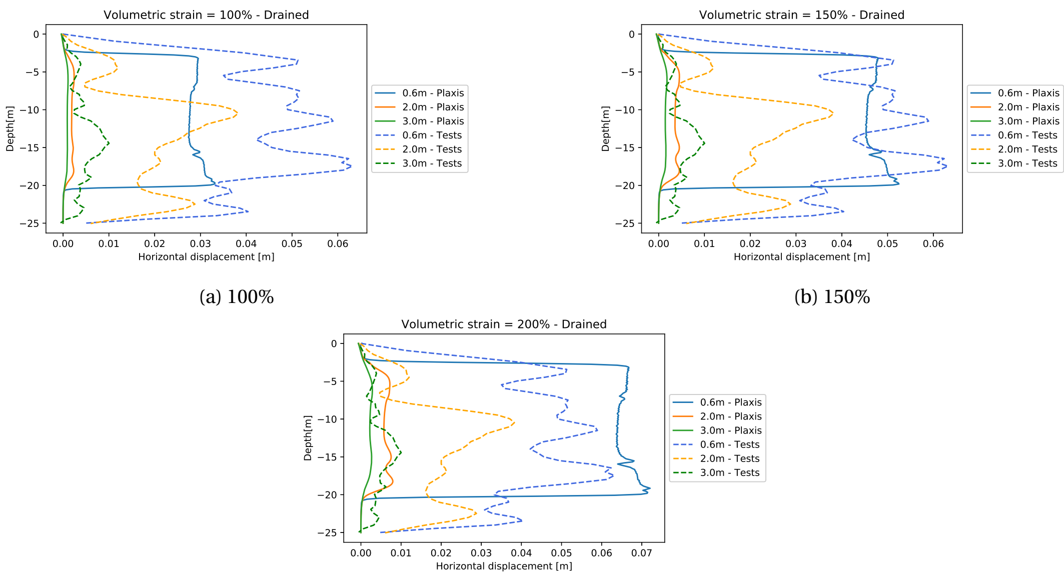


Figure 3.15: Displacements with different stiffness for the strained soil

the different soil layers, due to the constant volume in undrained conditions. Larger volumetric strains (150% and 200%) are thereafter tried to see if they better match with the measured soil displacement. The results are shown in Figure 3.16b, 3.16c, 3.17b and 3.17c. The model with 150% volumetric strain shows the best fit with the measured data. The drained model is a good match at a distance of 0.6m. At the other distances, the measured displacements are larger than the modelled displacements. In the undrained model, the displacements are larger and more homogeneous. The displacements, with exception at a distance of 2m, are over the entire length slightly smaller than measured. The models with 200% volumetric strain fit less well than those with 150% volumetric strain. In both drained and undrained conditions, the modelled displacements at a distance of 0.6m are too large. In drained conditions the differences between the soil displacements at a distance of 2m and 3m with the drained model with 150% volumetric strain are small. Only the undrained model correlates well at a distance of 2m.

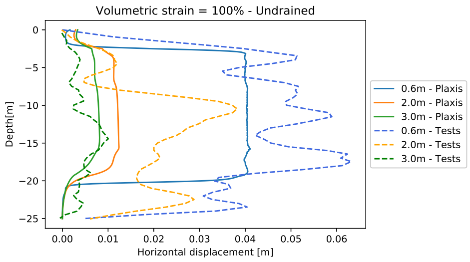


(a) 100%

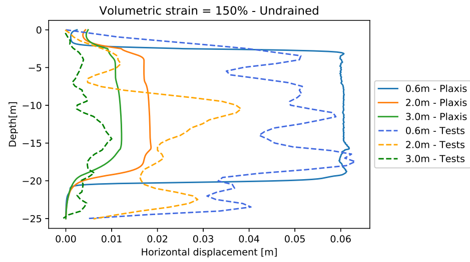
(b) 150%

(c) 200%

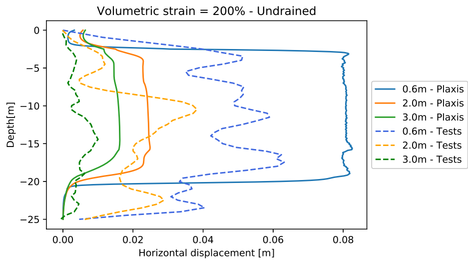
Figure 3.16: Field data compared with different volumetric strains



(a) 100%



(b) 150%



(c) 200%

Figure 3.17: Field data compared with different volumetric strains

3.2.3. Conclusion

In this chapter three different methods to model the installation of a screw pile in Plaxis 2D are compared. The first two models used a prescribed displacement to model the expansion phase. The applied CEM turned out to be quite similar to the method with a prescirbed displacement of the radius of the pile, so the applied CEM has no added value and is discarded. The radius prescribed displacement method is a good model in drained conditions for a first indication of the soil displacements. In this drained model, a clear difference in the displacement between the soil layers is visible. The undrained model overpredicts the displacements. When not looking at the magnitude of the displacement, but at the volume balance of this problem, the new volume of the pile is 4 time larger than the original pile. So there is too much soil being pushed aside, which makes this an unrealistic method.

The final method used volumetric strain for the expansion phase. The theoretical value of 100% volumetric strain generated a soil displacement smaller than the measured displacements. With larger volumetric strains, the best fit was with a volumetric strain of 150%. The drained model could be used as a lower limit, while the undrained model as the upper limit. In a FE analysis the soil layers are completely heterogeneous, this results in the same response along the entire soil layers. While in reality the soil layers are heterogeneous and will have stronger and weaker zones. The difference in soil parameters in a soil layer are missed in the FE analysis. Also the interaction between the weaker and stronger parts in a soil layer is missed. The soil will always find the path of least resistance. When a part of the soil layer has more resistance against the deformations, the displaced soil could move more towards the weaker part of the soil layer. With the drained and undrained model as limits, most values should fall within the modelled displacements of the models.

4

Case study harbour of Rotterdam

For the case in Shanghai a fitting model has been determined. The next step is to verify whether this model is also suitable for soils in The Netherlands. Tests have been done in the harbour of Rotterdam to determine the soil displacements in Dutch subsoil and the effect on adjacent piles. The test results have been used to validate the calculation model for Dutch soil conditions. In this chapter first some theoretical background about the tests is discussed. This will be followed by a description of the field test setup and its test results. Thereafter the 2D axisymmetric and 3D models are evaluated. The chapter will finish with a conclusion and recommendations.

4.1. Background

4.1.1. Inclinometer

Soil displacement can be monitored with an inclinometer. The inclinometer is a probe with two wheels on both sides. In the probe is a measuring tool which measures the angle of the probe relative to earths gravity. A tube, with guidance grooves on the inside, is placed in the soil (Figure 4.1). At every half meter the angle is measured. With the angles of every half meter it is possible to determine the profile of the tube. This profile is a relative measurement unless a point in the profile can be assumed to be fixed. Usually the bottom of the tube is placed in a stable soil layer, so it can be assumed not to move. If this is not possible, the top of the tube should either be fixed to something (e.g. adjacent structures) or the movements of the top of the tube should be monitored. When the soil around the tube is displaced, the tube will move with the soil. If the profile of the tube is determined at several times, it is possible to see the change in profile, which is assumed to be representative for the soil movement.

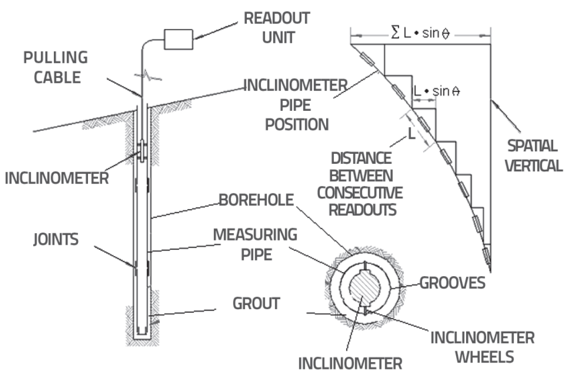


Figure 4.1: Inclinometer measuring principle (Dunnicliff and Green, 1993)

To acquire accurate measurements with the inclinometer it is necessary to measure in 2 directions of the tube. In the tube are 4 grooves through which the probe can go through. The direction in which the largest movements are expected should be chosen as A0-direction of the tube(Figure 4.2). In this direction the upper wheel of the probe is placed for the first measurements. The direction of the second measurement should be in the A180-direction. The probe measures the inclination in both the A-direction and in the B-direction. However, in the A-direction the measurements are more accurate than in the B-direction. In the A-direction is the rod confined between the grooves. In the B-direction is this not the case and could the probe have some freedom in its movements in that direction.

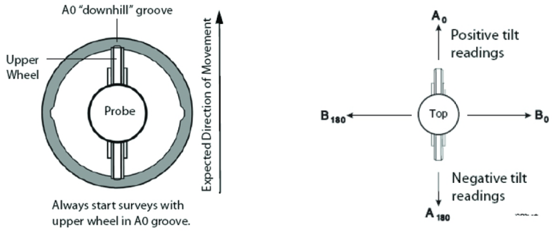


Figure 4.2: Orientation probe inclinometer (Digitilt, 2011)

4.1.2. Optical fibres

Optical fibres can give a continuous measurement of the strain or the temperature. An optical fibre consist of a glass core, cladding and a plastic coating around it. Through the glass core light can travel. This follows Snell's law of reflection. The cladding has a smaller reflection index than the glass core, which will make the light reflect at this surface. When a beam of light hits the surface between the core and cladding at an angle larger than the critical angle, it will reflect. This way the light will remain captures in the core of the fibre, maintaining the initial intensity (Figure 4.3).

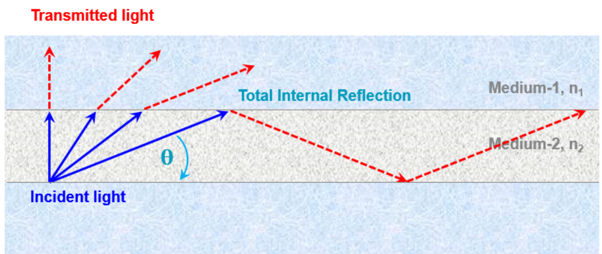


Figure 4.3: Light reflection in a fibre (Omnisens, 2018)

To measure the strain Brillouin Optical Time Domain Reflectometry (BOTDR) is used. When the light travels trough the fibre it is also back-scattered by the glass. This is done by different mechanism and causes backscattering at different frequencies; Rayleight, Brillouin and Raman scatterd light (Figure 4.4). The brillouin backscattering is depended on strain and temperature. When one of these changes, the brillouin backscatter changes frequency. From the change of frequency the change in strain or temperature is determined by a correlation. A change in strain of 1% will give a change in frequency of 500MHz and 1 degree of temperature change of 1MHz. With the speed of light it is possible to determine the position of the measured strains or temperatures. The Raman scatter is only influenced by temperature changes.

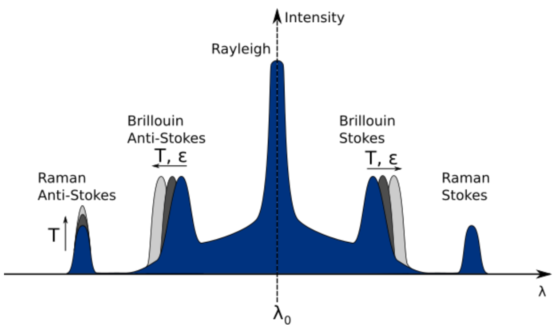


Figure 4.4: Different backscattering in an optical fibre (Responder, 2018)

The light travels with a certain amount of power trough the fibre. The power is measured in decibels. Due to different causes the light losses power. Power losses can occur when either the fibre is not placed correctly or when fibres are not connected correctly. Power loss due to incorrect placed fibre is caused by macro and micro bending (Figure 4.5). Macro bending occurs when the fibre is positioned in a bend with a radius smaller than 10cm. This causes the light to hit the cladding with a angle larger than the critical angle, so the light will not reflect anymore. Micro bending occurs when the fibre experiences different external pressures. This will disrupt the surface between the glass and the cladding. When the light hits the surface at the disrupted area, it could hit it with an angle larger than the critical angle (Omnisens, 2018).

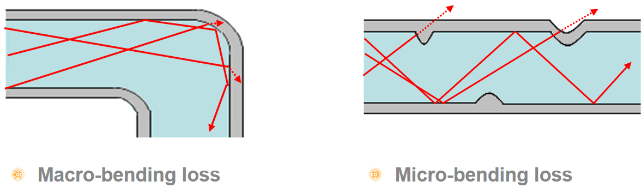


Figure 4.5: Causes of loss in optical fibre (Omnisens, 2018)

The other way of losing power is by making bad connections between fibres. The fibre must be connected to the interrogator. This is the machine that reads the wavelengths and translates it into strain or temperature. For the connection of the fibre to the interrogator a pigtail is used. A pigtail is a fibre with a connector on one end. This connector can be plugged into the interrogator. To connect the fibre to the pigtail a splice is made with the end of the two fibres. A splice is comparable with a weld. This is done in a special machine. Depending on the quality of the splice, the fibre will loose some power in the splice. Power is also lost when the connector has dust on it. With special cleaning tools, this is preventable. However, no matter the quality of the splice or how clean the connector is, some power will always be lost at these points(Figure 4.6).

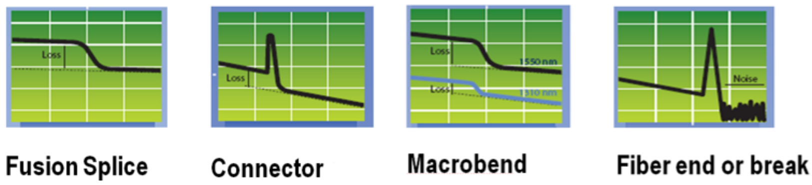


Figure 4.6: power loss at splice and connector (Omnisens, 2018)

When measuring with the optical fibres, there are 2 options; single ended or double ended. With a single ended measurement the fibre is only connected at one end to the interrogator. It will send a pulse in the fibre at one end and the reflected light will also be received at the same fibre end. With a double ended measurement, the fibre is connected with the interrogator at both ends. This gives the opportunity to send the laser in at both ends and get reflected light from both directions. The two pulses collide at the section of the fibre that is being observed. This strongly improves the signal to noise ratio. This gives a more accurate results. Another advantage of installing the fibre in a loop (double ended measurement) is the possibility of changing it in a single ended measurement if the fibre is damaged.

4.2. Measurements harbour of Rotterdam

In the harbour of Rotterdam a new railway viaduct will be built. To see if the considered pile meets all the requirements for the project, test piles were installed. It was possible to use these piles to measure soil displacement and the deformations of the piles. The tests consist of 5 piles installed in a square area with one in the middle (Figure 4.7). The corner piles are installed first and the centre pile last.

The piles used for the test are Fundex piles with a length of 32 meters. A Fundex pile is smooth and has a uniform diameter along the entire length. Only the screw tip has a thread, the rest of the tube does not. The screw tip has a diameter of 850mm and the tube, attached to the screw tip, has a diameter of 610mm. The eventual pile should have a diameter of 711mm.

The reinforcement of the pile consist out of 8 rods connected with a spiral. The reinforcement will be installed over the entire length of the pile. However, the reinforcement cage has been split in 2 parts for transportation and handling reasons. See Appendix B for the design details of the piles and reinforcement.

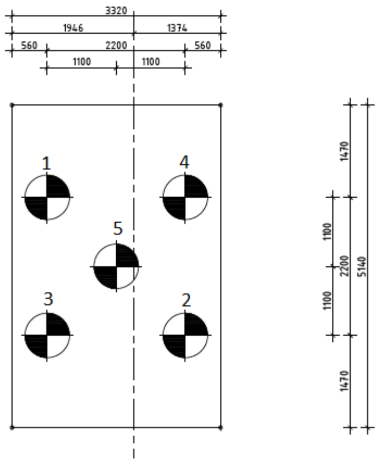


Figure 4.7: Pile plan of tests piles (SaVe, 2018)

To determine the soil profile of the test location several CPT and boreholes were available. A profile of the soil is deduced from these results (Figure 4.8). In Appendix C is the CPT closest to the project location. Below the top layer there is a thin layer of very soft clay. Below this there are several layers of sandy clay. At 20m below NAP the Pleistocene sand layer starts. This sand layer offers the majority of the bearing capacity of the piles.

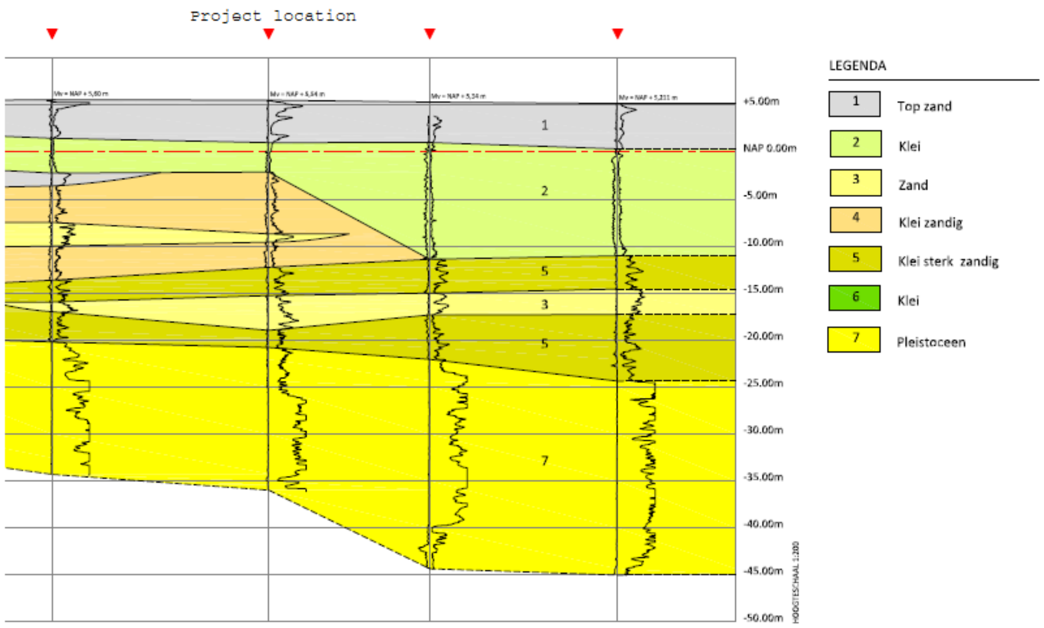


Figure 4.8: Soil profile at test location

4.2.1. Method

The main goal of the tests for this research is measuring the soil displacement and the deformation of the piles. The two types of measurements used are inclinometers and optical fibres. Four inclinometers were installed (Figure 4.9). Two inclinometers are installed in the soil with a depth of 35 meters. They are at a distance of 1.1m and 3.3m from the first pile. Initially this will give insight in the soil displacement at different distances caused by just one pile. When the other piles are installed it will also give insight of the behaviour of the soil when strained by the installation of several piles. Two inclinometers are attached to the reinforcement in the pile. This gives the deformation of the pile due to increased soil pressure from adjacent installed piles.

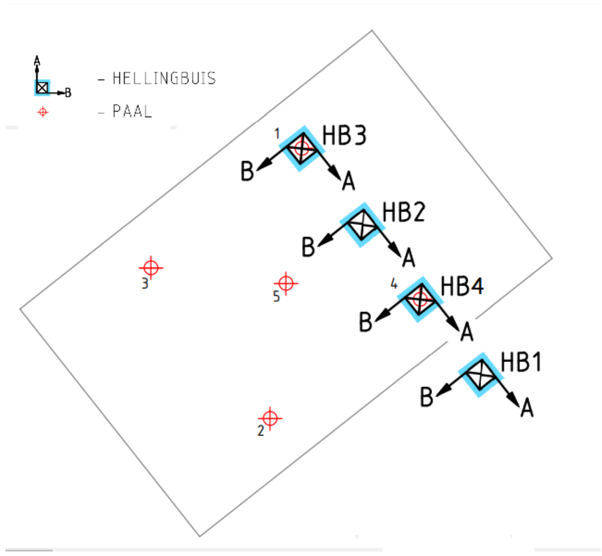


Figure 4.9: Position of the inclinometers

The optical fibres are installed in the four corner piles. Pile 5 does not have any optical fibre, because it is the last pile, so it will not experience deformations from later installed piles. In every pile the fibre is attached to the reinforcement in the pile. To be able to see the strains in all the directions of the pile, the fibre is attached on four sides of the reinforcement(Figure 4.10). To match the measured strain with the right side of the pile it is important to know where the fibre is located in the piles. The fibres are installed with the same configuration in all the piles and labelled, so afterwards it is possible to determine the position of the fibres. The orientation of the cage is done with the help of the earthing rod. From the position of the earthing rod it is possible to determine the position of the fibres.

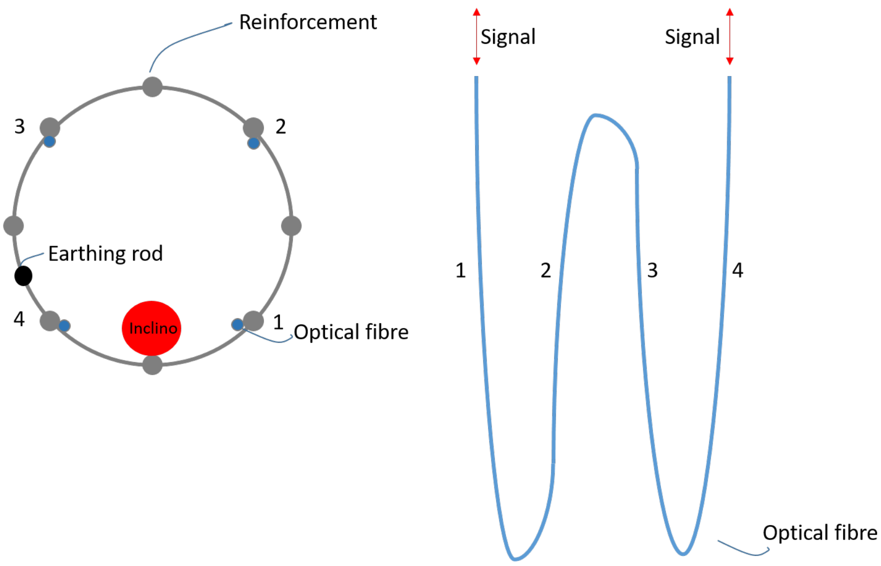


Figure 4.10: Attachment of optical fibre to the reinforcement cage

Because the reinforcement is installed in 2 parts on site, it is unfortunately not possible to run the fibre through both cages. To be able to measure the pile to the full length, the fibre is connected separately on both cages. The ends of the fibre of the bottom cage are extended and tied together to pass the upper cage to the surface.

To be able to connect the optical fibre to the interrogator an extra length is added. This gives length to make a splice between the fibres and pig tails. To distinguish the 2 ends of the fibre, the end of the fibre at 1 was marked with a blue tape wrapped around it and the end of the fibre at 4 with a red tape. The fibres from the bottom cage are recognisable by the fact that the ends are tied together.

The bends at the bottom and top of the reinforcement cages were considered to be vulnerable. To protect these parts against falling concrete, a flexible PVC tube is placed around the fibres (Figure 4.11).

Figure 4.11: PVC tubes around optical fibres in the bends

The measurements start just before a new pile is installed. The measurements continue during the installation, so the development of the strain is monitored. The concrete first heats up and when the hydration is done, will cool down slowly. With the BOTDR it is not possible to tell the difference between strain and temperature changes, so the assumption is made that during the short time of installation the temperature change is so small it can be assumed constant. All the changes measured are all attributed to the strain.

4.2.2. Results

In the field things did not go as planned, which influenced the results. In Appendix D is a logbook of what happend during the test. This explains more in detail how the results came into being. The rest of this chapter is mainly focused on the retrieved data and not on how some of the data was lost.

Inclinometers

All inclinometers were measured every day before a new pile is installed. For the tubes in the ground a baseline measurement is needed before installation of the first pile starts. The inclinometers in the pile have a baseline measurement once the pile is installed. Due to large deformations and possible disconnection of the tube sections in the first pile, not all the inclinometers were measured to their entire depth. In Table 4.1 is an overview of the inclinometers with their start of operation and till what depth they are measured. Inclinometer 2 had after installation of pile 1 a curve that was too sharp for the measuring probe to pass below 5 meters depth. After installation of pile 4 the tube was pushed back, which decreased the sharpness of the curve and it could be measured to the bottom again. After installation of pile 5 the soil displacements were too large again, so that the curve increased again and the tube was only measurable the upper 5 meters. Inclinometer 3 is only measured up to 12m depth. The probe did not want to pass the tube further. When lowering a small load trough the tube, this did not go any further than 12m depth. This indicates that it is not a sharp bend blocking the probe, but the tube being filled with most likely concrete. Probably 2 sections of the tube were detached of each other and concrete could flow into the tube.

Table 4.1: Measured length inclinometers

	Inclino 1	Inclino 2	Inclino 3	Inclino 4
Baseline measurement	35m	35m	-	-
After pile 1	35m	5m	12m	-
After pile 2	35m	5m	12m	-
After pile 3	35m	5m	12m	-
After pile 4	35m	35m	12m	32m
After pile 5	35m	5m	12m	32m

The results of inclinometer 1 are shown in Figure 4.12. These are the displacements in comparison with the baseline measurement. In Appendix E are the displacement differences per day of inclinometer 1. This shows the effect of each individual pile on the inclinometer. In Table 4.2 are the average displacements per soil layer noted.

The inclinometer is at 3.3m distance of pile 1. The inclinometer shows only movement in the A-direction of the inclinometer. This is in the radial direction away from the installed pile. In the clay layer are displacement measured up to 18mm. In the sandy clay layer the displacements are around the 2.5mm. In the sand layer almost no displacement is measured. During installation of pile 2 displacements are mainly measured in the B-direction. The distance between pile 2 and the inclinometer is 3.1m. The soil displacements are the largest in the clay layer again, with a displacement up to 28mm. The sandy clay layer and sand layer have the same displacements as after installation of pile 1; 2.5mm and 0mm. Pile 3 is at a 3.9m distance from the

Table 4.2: Average displacement per soil layer during installation of the piles

	Pile 1		Pile 2		Pile 3	
	A	B	A	B	A	B
Distance	3.3m		3.1m		3.9m	
Clay layer	12,9mm	0,0mm	-0,1mm	-18,6mm	1,5mm	0,1mm
Sandy clay layer	2,0mm	-0,8mm	-0,2mm	-4,4mm	0,7mm	0,2mm
Clay layer	-0,5mm	-0,1mm	-0,8mm	-1,0mm	0,4mm	-0,1mm
	Pile 4		Pile 5			
	A	B	A	B		
Distance	1.1m		2.5m			
Clay layer	26,2mm	-2,0mm	-4,8mm	1,7mm		
Sandy clay layer	12,1mm	-4,3mm	-1,8mm	1,0mm		
Clay layer	5,5mm	-3,3mm	0,3mm	0.1mm		

inclinometer. During the installation of pile 3 barely any displacements are measured in the inclinometer. Pile 4 is the closest to the inclinometer at 1.1m distance. The installation of pile 4 causes displacement in the A and B direction of the inclinometer, with the largest displacements in the A-direction. In the A-direction the displacements in the clay layer are around the 26mm, in the sandy clay layer around the 12mm and in the sand layer displacements of 6mm were measured. The displacements measured in the B-direction are uniform over the depth of the inclinometer between the 2mm and 4mm. This is not a displacement in the radial displacement, as would be expected. Close to the pile is an area were the soil is highly disturbed by the installation of the pile. This probably causes the pile to deviate from the expected path. The displacements due to the installation of pile 5 are not in the expected direction. The inclinometer moved towards the installed pile. During the installation of pile 5 the casing was not retrievable. Different techniques were used to try to extract the tube. This caused large disturbances in the soil. Because of this the measurements are not representable for normal pile installation and not used in further analysis.

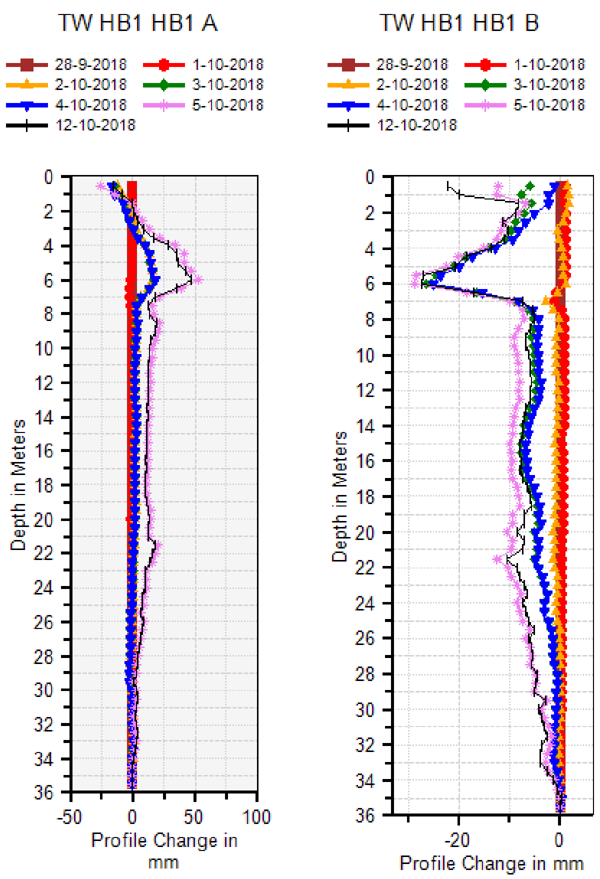


Figure 4.12: Profile changes inclinometer 1

Because inclinometer 2 and 3 were not measurable until the bottom, there was no fixed point for these inclinometers. Without an fixed point it is not possible to compare the different measurements. No conclusion can be drawn from these results, so they are not used for further analysis. Inclinometer 4 shows the same direction of movement as inclinometer 1 after installation of pile 5 (Figure 4.13). As mentioned, this is in an unexpected direction, most likely due to the activities around pile 5. These measurements has been done to determine the effect of the installation on adjacent piles. This is not retrievable from this data, but it can be compared with the measurements from the inclinometer after installation of pile 5 (Figure F.4). Inclinometer 4 is at a distance of 1.56m from pile 5 and has larger displacements than inclinometer 1, which is at a distance of 2.5m from pile 5. That the displacements in inclinometer 4 are larger is logical, because it is closer to pile 5. When looking at the pattern of both displacements, they are similar to each other. In the A-direction the average difference is between the displacements is 6.2mm and for the B-direction 4.3mm. This suggests that the pile, in which inclinometer 4 is installed, moves similar to inclinometer 1 and thus similar with the soil displacements.

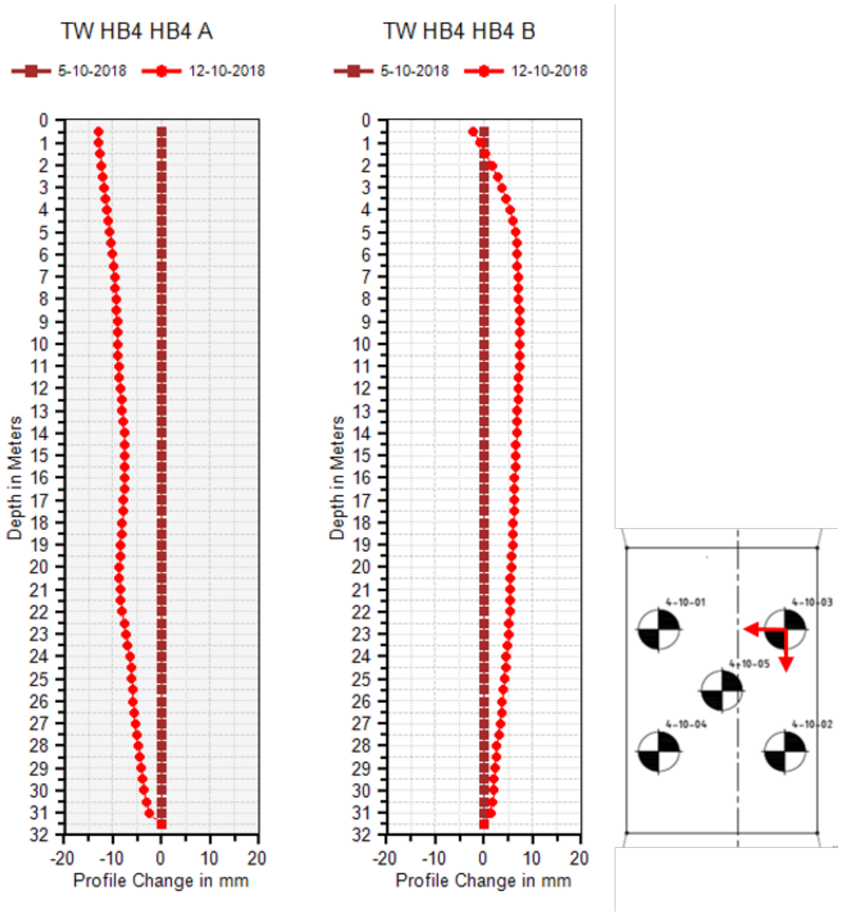
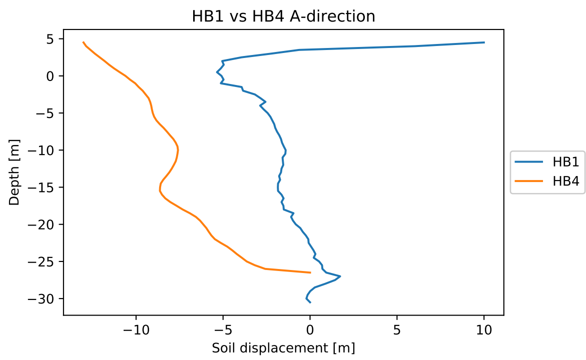
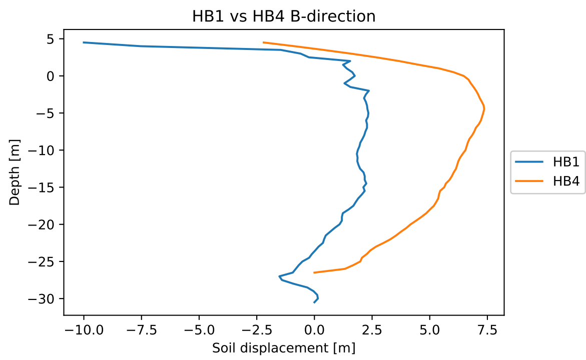


Figure 4.13: Profile change inclinometer 4



(a) A-direction



(b) B-direction

Figure 4.14: Inclinometer 1 vs Inclinometer 4

Optical fibre

The optical fibres proved to be very fragile. During the concrete casting process some fibres were severely damaged and could not be used. Fortunately some fibres did survive the installation and could be used for further analysis. In Table 4.3 is noted which fibres were measurable. It turned out that all of the fibres had a break somewhere along the fibre, but some were still measurable with the single ended mode.

Table 4.3: Measured length optical fibres

	Top cage	Bottom cage
Pile 1	Single ended: 30m	Broken
Pile 2	Broken	Broken
Pile 3	Broken	Broken
Pile 4	Single ended: 60m	single ended: 90m
Length of total fibre	60m	100m

Optical fibre in pile 1

The optical fibre on the top cage of pile 1 had a measurable length of 30m. This suggests that the break of the fibre is at the top bend between side 2 and 3. With the single ended mode is was still possible to get data from the fibre. The strains in pile 1 are measured during the installation of pile 2, 3 and 4. The fibre was not measured during the installation of pile 5, because the extension cable to the interrogator was needed for the the fibres in pile 4. The result of the optical fibre in pile 1 during the installation of pile 1 are shown in Figure 4.15. The measurement have an interval of 1,5 minutes. Unfortunately some data got lost, so the shown data is from after the installation of pile 2. The strain changes between the different measurements are all between roughly 50 $\mu m/m$ and -50 $\mu m/m$ (Figure 4.16). The error margin with single ended mode is 50 $\mu m/m$, so this is more likely due to a measurement error than due to a change in strain. There are also no strain changes expected, because pile 2 is already installed and no changes occur in the soil. The measurements do however show that the fibre is still usable despite the destructive environment during pile installation.

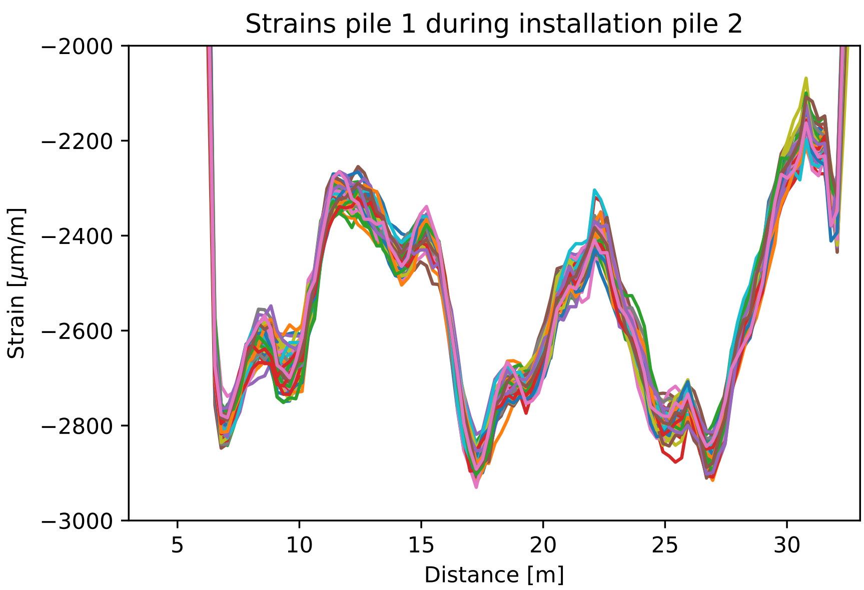


Figure 4.15: Strains pile 1 during installation pile 2

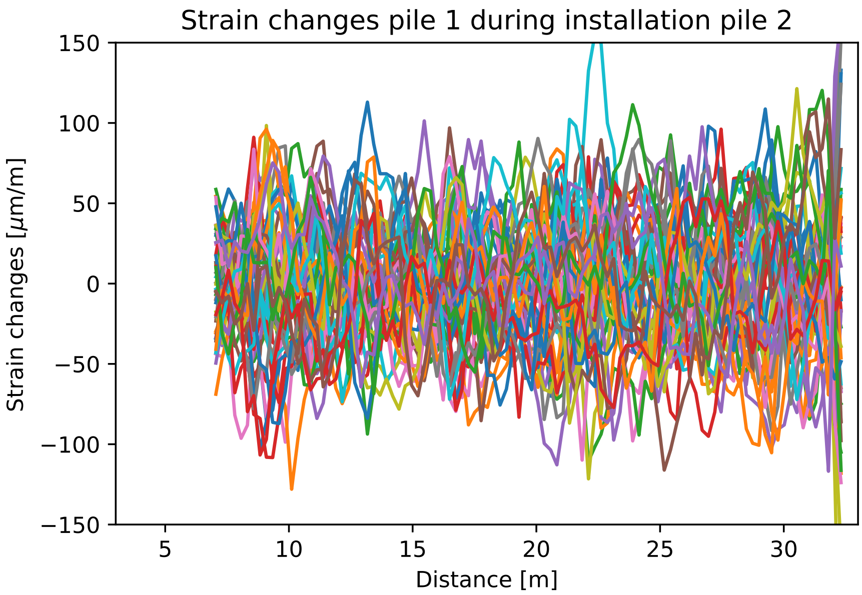


Figure 4.16: Strain changes pile 1 during installation pile 2

The measurements of the fibres in pile 1 continued during the installation of pile 3 and 4 (Figure 4.17 and 4.19). The measurement are done every 1.5 minute and started before the installation started until after the installation. In the measurements during the installation of pile 3 there is a lot of noise in the data. Somewhere in the fibre the signal got lost, this could be due to several reasons as mentioned in subsection § 4.1.2. In this case, dust in the connector is the assumptive reason of the noise. The data should look similar to the measured strains during installation of pile 2. When looking at the values between the -2000 $\mu m/m$ and the -3000 $\mu m/m$, a similar pattern becomes visible (Figure 4.18). However, even with most of the noise being filtered out of the results, there is still too much noise in the data to use it for further analysis. The measurement of the strains during the installation of pile 4 is also unusable. The noise is even more extensive than the noise during installation of pile 3. Is seems like there is a break in the fibre at 115m (Figure 4.19).

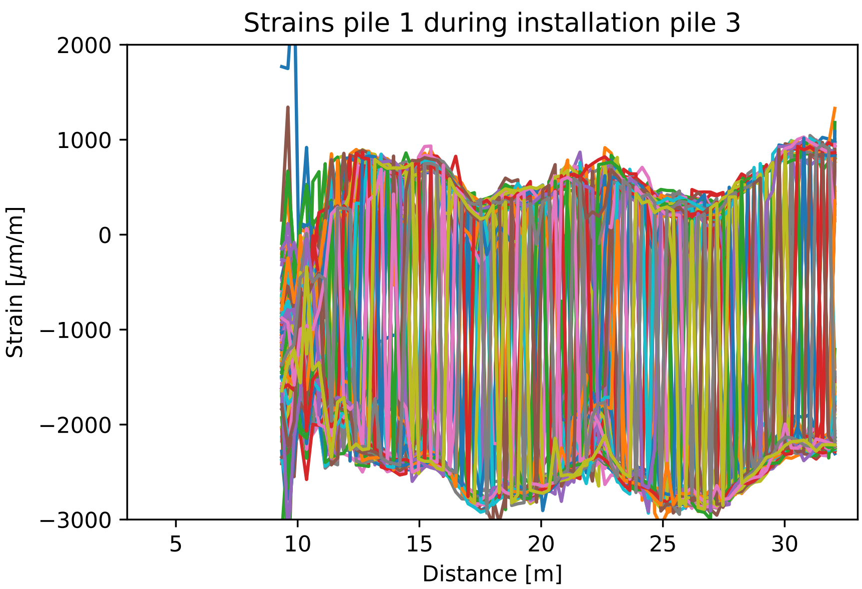


Figure 4.17: Strains pile 1 during installation pile 3

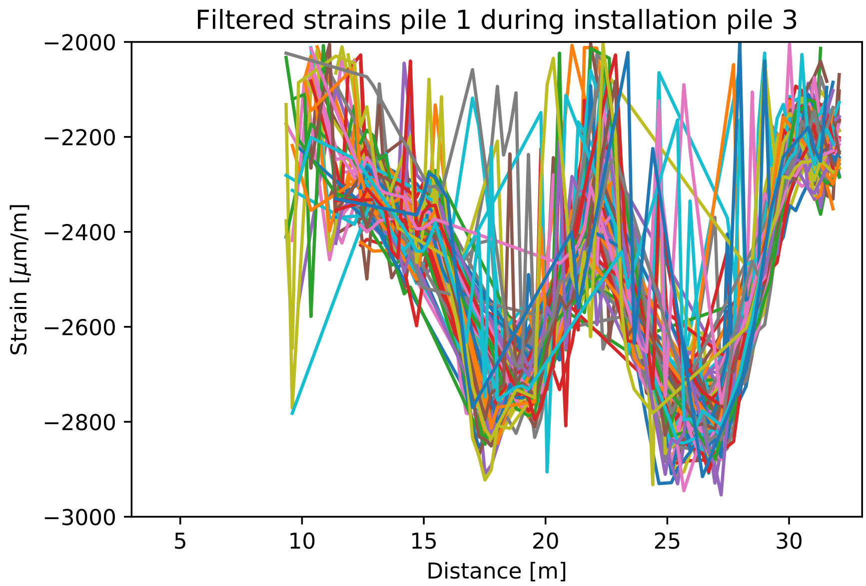


Figure 4.18: Filtered strains pile 1 during installation pile 3

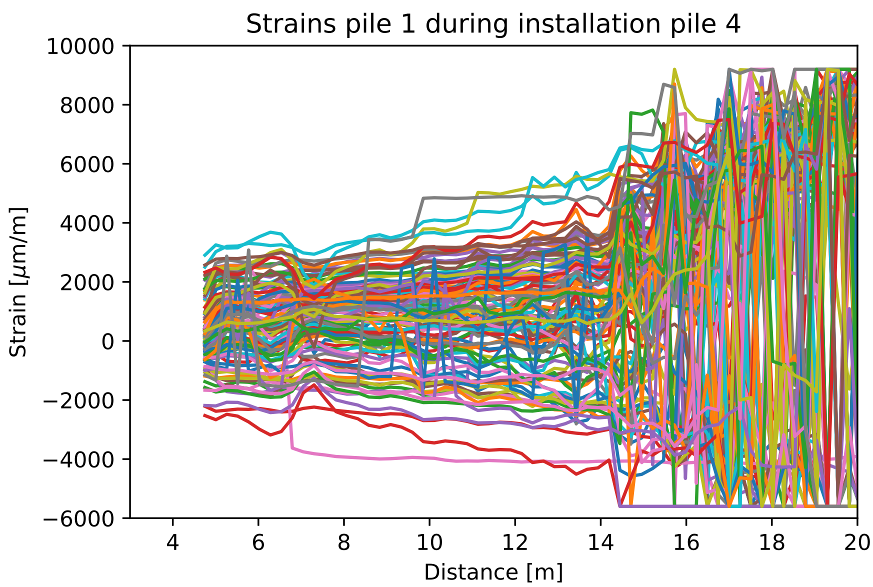


Figure 4.19: Strains pile 1 during installation pile 4

Optical fibre in pile 4

After the installation of pile 4 the optical fibres of pile 4 are connected with pig tails, so the fibres can be connected with the interrogator. The optical fibres from the top cage were too short to make a splice on site. The fibres from the bottom cage were possible to connect and to measure. The fibre is broken, but with single ended modus it is possible to measure almost the entire fibre. During the night the connection between the pig tail and fibre was damaged. Due to time constrains, it was not possible to repair the fibres before the pile was installed. The fibres were measured again after pile 5 was installed. This offers the opportunity to compare the strain before and after installation, but the development of the strain during installation has not been recorded. As discussed the installation of pile 5 did not follow the plan. The casing of the pile could not be retrieved. This has a large influence on the results and should be taken into account when analysing the results.

The results from the measurements are shown in Figure 4.20. It shows a clear symmetry in the loops the fibre makes in the reinforcement cage. Both 2 loops at the bottom of the cage (between parts 1-2 and 3-4) have less compression than the loop at the top of the cage (between part 2-3). The change in strain due to the installation of pile 5 is shown in Figure 4.21. Almost the entire length of the fibre was more compressed after the installation of pile 5. The average strain changes are around 200 μm/m. This could be caused by three things; Deformation of the pile, temperature difference due to the hydration of the concrete, shrinkage of the concrete. If the pile was deformed, this would have resulted in a different pattern in the strain changes. Bending is the most common deformation type. This would result in tension on one side of the pile and in compression of the other side. In this case the fibre only shows uniform behaviour, so this cannot be explained by pile bending. This suggests that, if the pile is influenced by the newly installed pile, the pile is loaded evenly along the entire shaft. The shrinkage of the concrete during the hydration of the concrete could explain the uniform compression of the fibre. Also the temperature changed would give a uniform change in the fibre. Most likely is a combination of these factors the cause of the strain changes.

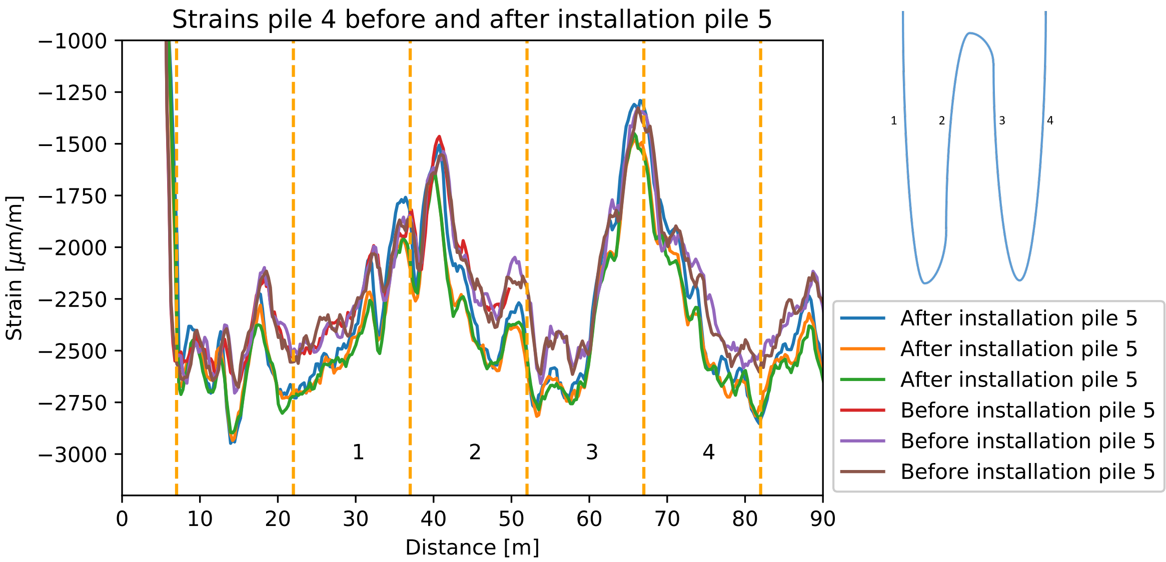


Figure 4.20: Strains pile 4 bottom cage before and after installation pile 5

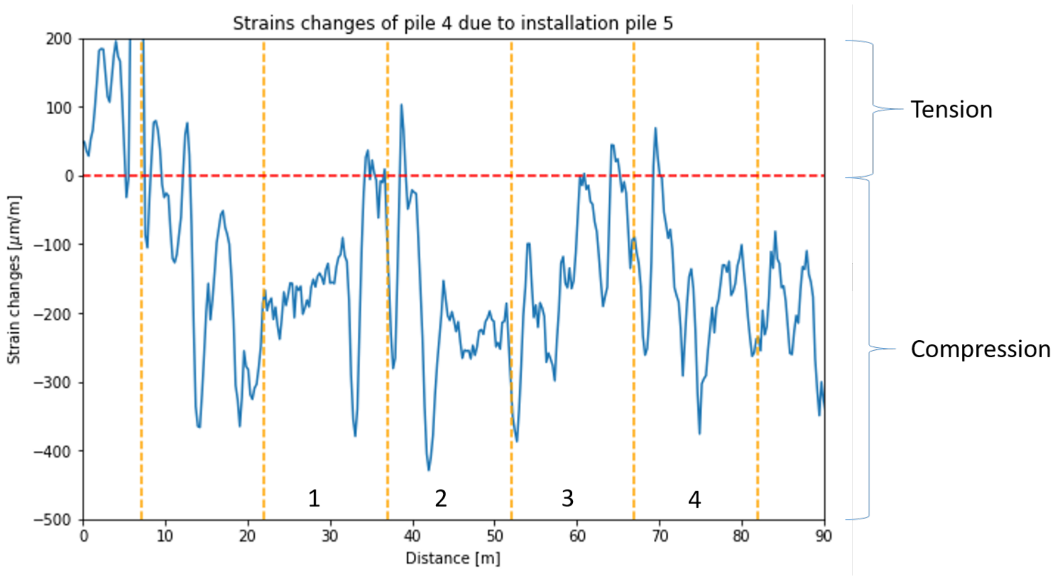


Figure 4.21: Strains changes pile 4 bottom cage due to installation pile 5

After the installation of pile 5 and after the concrete had hardened is was possible to reconnect the fibres in the top cage of pile 4 to the interrogator. This cannot be used for assessing the strain changes, but will show if the fibre is damaged during the installation process. In Figure 4.22 the results are shown. The fibre could be measured with single ended mode over almost the entire length. These measurement do not show the same pattern as the bottom cage. In this fibre the loop at the top of the cage experiences a lower amount of compression instead of the highest amount of compression. This shows that the installation of the fibre in this way does not give a standard pattern in the strain measurements. No further conclusions can be drawn from these measurements.

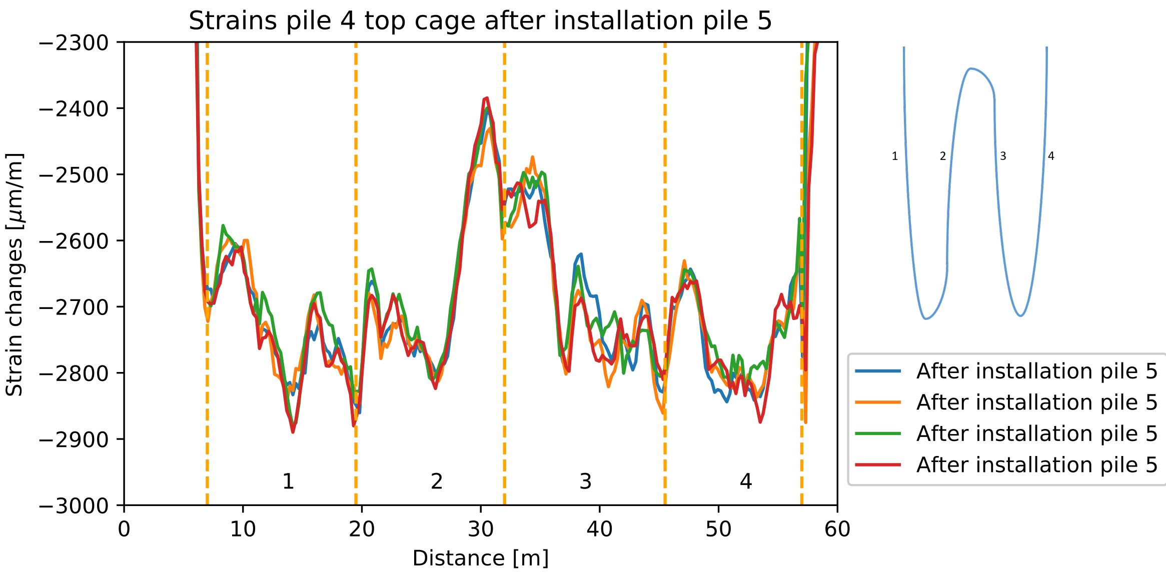


Figure 4.22: Strains pile 4 top cage after installation pile 5

4.2.3. Discussion

Inclinometer

The inclinometers in the ground are meant to analyse the soil behaviour due to the pile installations. Because inclinometer 2 could only be accessed the upper 5m, only inclinometer 1 is used for this analysis. Inclinometer 1 is at different distance of every pile. Because of this it is possible to see the soil response at different distances from the pile. In Appendix E are the profile changes per installed pile. The soil profile is divided in 4 main soil layers(Table 4.4).

Table 4.4: Soil layers

Layer	Begin	End	Soil type
1	0	-3.5	Top layer
2	-3.5	-7	Clay
3	-7	-25	Sandy clay
4	-25	-36	Sand

For these soil layers the average soil displacement is calculated for every distance from the installed pile (Figure 4.23). The top layer has an unexpected pattern in the displacements, because the measured displacements do not decrease with an increasing distance from the pile. This layer is very sensible for activities on the surface. The soil in this layer will also move in vertical direction, but if the piling equipment is present on the surface this movement is prevented. Because of the sensitivity of this layer, it is not further analysed. For the other layers the trend shows a clear decrease in displacement with an increasing distances from the pile. The soil displacements are depending on the diameter of the pile. When normalising Figure 4.23 with the radius of the pile, it will be also usable for different diameter piles. The normalized graph is shown in Figure 4.24. In this graph also the trendline for each soil layer and the theoretical CEM value is shown. The CEM theory assumes that the volume of the soil is a constant, so the soil is incompressible. The result from these tests show that this does happen in reality. For each layer a correction factor on the CEM value is calculated. For clay the values are around 0.54 times the CEM values. However, the clay layer has large deviations from this line. This could be because it is still to close too the surface, so it still sensitive to activities on the surface. Another option could be the heterogeneity of the layer. The drainage of the groundwater plays an important part in the soil displacement. If this differs in the clay, the displacement is also different. The measured displacement by Meng et al. (2015) showed relaxation of the soil during the dissipation of the excess pore pressures. To rule this out, measurement should be done after consolidation. With clay layers it might be safe to assume that it is incompressible and to use the CEM value. For the sandy clay layer the value lie around the 0.15 times the CEM value and the sand layer around 0.06 times. These values show less variation from the calculated trend. For these layers the CEM values are to conservative.

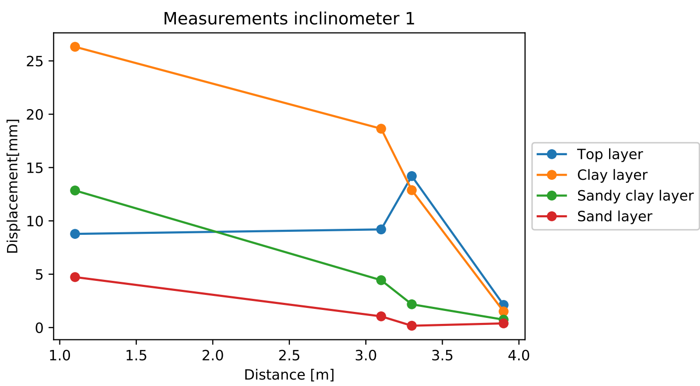


Figure 4.23: Soil displacement per layer at different distances from the pile

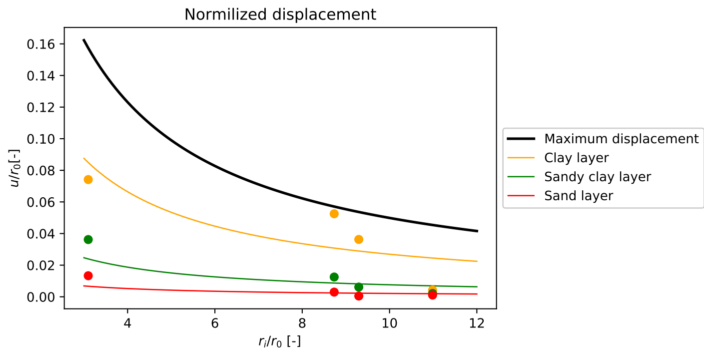
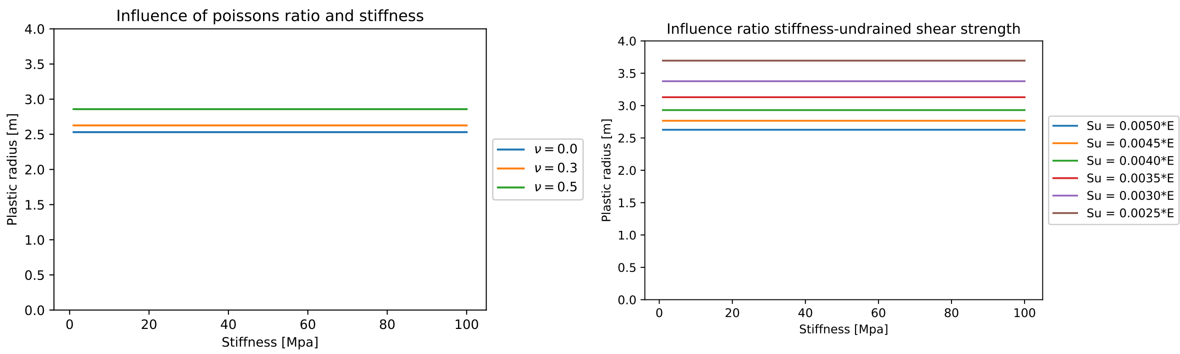


Figure 4.24: Normalized horizontal displacement

The CEM value which is used as comparison for the measured displacement does not take any soil parameters in to account, it only uses geometry to determine the largest displacement possible. The CEM cannot determine the displacement with the limited data from the tests. With the data that is available it is possible to calculate the radius of the plastic zone (Yu, 2000), which gives an indication of the spread of the displacement in the area around the pile (see subsection § 2.2.1). With Equation 2.1 the influence of different soil parameters on the magnitude of the plastic radius is determined (Figure 4.25). The soil parameters that can be varied are the Poisson's ratio, stiffness and the undrained shear strength. First the undrained shear strength is kept constant. This shows that the Poisson's ratio has a influence on the plastic radius, but the stiffness does not. When keeping the Poisson's ratio constant the stiffness still does not have any influence, but the ratio between the stiffness and the undrained shear strength does. The soil parameters for the location in Rotterdam are determined with correlations, so the parameters are not accurate enough see if the equation would give a good prediction. Still, the impact of the parameter variation is limited and the influence of the soil stiffness is limited.



(a) Poisson's ratio - Stiffness
S_u=0.005*E

(b) Undrained shear strength - Stiffness
$\nu = 0.3$

Figure 4.25: Influence of soil parameters on the plastic radius

Optical fibre

The data from the optical fibres are not usable for further analysis. However, from the test some valuable conclusions are possible. It was assumed that during the installation of the pile no significant temperature change would occur, so all the measured changes in the signal could be attributed to the strain. This is still a plausible assumption, but only valid if the measurement are done during or immediately after installation of the pile. The building site environment is an extra challenge for the optical fibres. The fibres should be preferably handled in a clean environment. The dust and concrete everywhere making good splices very difficult. Also the connectors were difficult to keep clean because of this.

The optical fibres were more fragile than expected, partly due to a different concrete casting procedure. Along the shaft of the pile the fibre remained, for as far it can be seen from the data, undamaged. Most problems occurred at the top of the pile. The parts that are not tied closely to the rebar cage are the most vulnerable. At the top, extra length for the connection with the pig tails is needed, this part cannot be tightly attached to the cage and is dragged along with the falling concrete. The activities at the building site during the installation of a subsequent pile are also a risk for the fibres that exit the pile. A small bump can break the splice between the pig tail and the fibre. If this happens during the installation of the pile. it is not possible to repair the splice and not further measurements can be done.

4.2.4. Recommendations

The main goal of the test piles was to see if the casing could be retrieved. The test for soil and piles displacements could use these piles as well, but would not disturb the main test. For possible future test it is desirable to not combine it with other tests. Everything can be designed and executed most favourable for the intended tests. For example not using a reinforcement cage in 2 parts, but a cage existing of just one part, would have been favourable for the optical fibres. Besides this there are recommendations for the improvement of the method and also practical recommendations for execution of the tests on the construction site.

Method

During these tests there were no measurements done of the pore pressures. Pore pressure measurements could give an interesting insight in the development of excess pore pressure and the consolidation. Pore water pressures might explain the large variations in the measured displacements of the clay layer.

To make the optical fibre measurement more reliable, it would be useful to know the temperature of the concrete. With the Brillouin scatter the observed changes could be due to strain or temperature changes. It is also possible to do measurements using the Raman scatter. The changes measured with the Raman scatter are only depending on temperature change. At the moment this requires an extra fibre installed in the pile. It will be possible in the future to do Brillouin scatter and Raman scatter measurements through the same fibre.

Execution

During the tests, the optical fibres and inclinometers had a high rate of failure. With some adjustments these can be prevented in future tests.

For the installation of the inclinometer tubes it is important to know where all the drilling equipment will be placed. With this it is possible to position the inclinometers where they will not be damaged.

The most difficult part of the optical fibres is to keep the overlength, required for connecting with the inrerrogator, undamaged. The overlentgh should be stored safely during the installation of the pile. The metal plate to protect the fibres from the falling concrete, works good (see Appendix D), but the coil of fibre was still damaged. The concrete did not pull the fibre down, but still sanded the fibres while it fell past the coil. It was also difficult to keep the coil in a radius larger than 10 cm. If a steel bar is welded on the reinforcement cage it could guide the overlength. With this the fibre does not need to be coiled and will be easily accessible after installation. It is the same idea as with the plastic tubes, but firmly welded on the cage. The steel bar can be removed after the overlength is detached, so it is not in the way for the installation of next piles. The fibre is still sanded this way, but this can be solved with a stronger fibre.

If the fibre survived the installation, the overlength with the splices is the most vulnerable part of the fibre. In this test the fibre was just draped around the reinforcement cage, but this way it is easily damaged by people, wind or tools. A box should be designed in which the fibre is safely stored and can still be connected to the interrogator.

4.3. 2D axisymmetric model

In this section the finite element model for the case in the Harbour of Rotterdam is discussed. The recommendations from the Shanghai model for the best fit are used for this model to see if this fits the Rotterdam measurements as well.

4.3.1. Method

Phases

The Rotterdam model only has 2 phases (Figure 4.26). Because the pore pressure and consolidation turned out to be difficult to judge, it is of no use to model this. For the expansion phase the method is use that was determined the best for the Shanghai model. This is the method with 150% volumetric strain for the expansion phase. For the Rotterdam case the only measurement without influence of other piles is done after the installation of pile 1. This pile is at a distance of 3.3m from the inclinometer which measures the displacement due to the installation of the pile. To keep the analysis as similar as possible to the Shanghai case, these measurements are used to fit the modelled displacements. The modelled displacements are again determined with a plate placed at a distance of 3.3m from the centre of the pile.

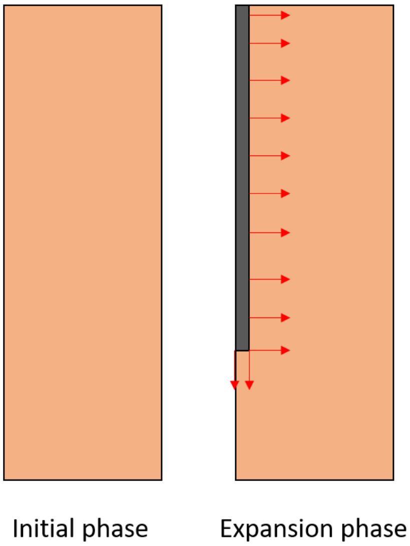


Figure 4.26: Phases Rotterdam model

Soil parameters

With the help of a CPT the soil parameters of the soil layers at the test site are determined. Determining soil parameters from a CPT is a rough estimation of the actual parameters. The Mohr Coulomb soil model has parameters that are the easiest to determine. This is done with the help of table 2b of the Eurocode 7 (7, 2018). Because of this it is decided to use the MC model. For the Hardening Soil Small Strain model there also are correlations to determine the soil parameters, but these give just a larger error margin with the measured data. In Table A.4 the Mohr-Coulomb soil parameters for the soil layers are noted. For the Shanghai case was looked at the difference between the MC model and the HSS model. From this can be expected that the MC model will give a small overestimation of the soil displacements in drained condition. In undrained conditions the soil displacements are similar for both soil models. This should be taken into account when looking at the results from the FE analysis.

Table 4.5: Soil parameters Harbour of Rotterdam

Soil layer	γ_{unsat} [kPa]	γ_{sat} [kPa]	C [[kPa]	ϕ [°]	ψ [°]	E [MPa]	ν [-]
Top layer	17	18	0	22.5	0	3	0.25
Clay	14	14	0	17.5	0	1	0.4
Sandy clay	20	20	0	25	0	15	0.2
Sand	21	21	0	35	0	75	0.3

Dimensions and structures The dimensions of this model are 25m width and 40m depth. This is chosen so the boundaries do not influence the displacements. The pile has a diameter of 0.7m and a length of 35m. This is according to the design of the pile. There is only one plate used in this model at a distance of 3.3m. This has a length of 35m, the same as the inclinometers used in the field tests.

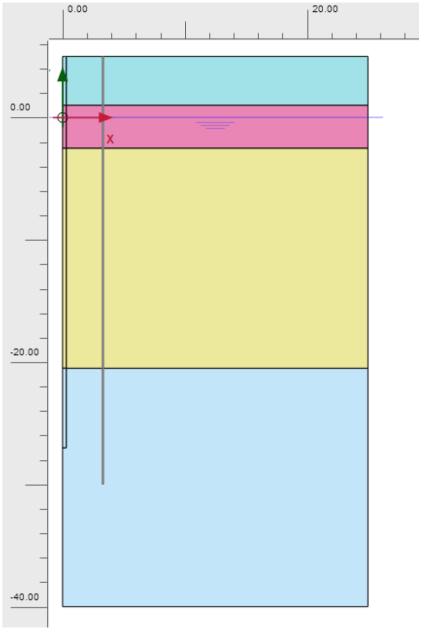


Figure 4.27: Dimensions Rotterdam model

Mesh
For this model is chosen for the 15-node elements. The 15-node element is a 4^{th} order interpolation for displacements. The size of the elements are medium. The model has 1298 elements and 10637 nodes.

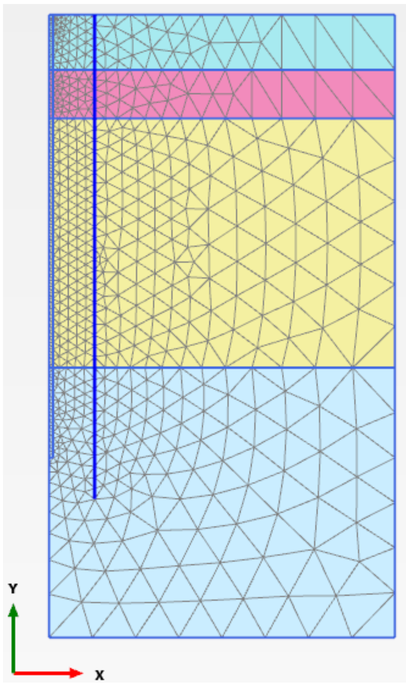


Figure 4.28: Mesh Rotterdam model

Boundary conditions

All the boundaries use the default settings. This means that the displacements in the x-direction in the vertical boundaries are fixed. The displacements in the y-direction are free. For the horizontal boundaries there is a difference between the top and bottom boundary. At the top boundary (surface level) all the displacements are free. For the bottom boundary all the displacements are fixed. The waterflow is open in all direction except in the bottom horizontal boundary (Plaxis, 2018).

4.3.2. Results

The first model is with a volumetric strain of 150% to see if the model of Shanghai is also applicable here. The results of this in undrained conditions is shown in Figure 4.29. Only the undrained conditions were modelled for this volumetric strain, because is was already clear that the modelled displacements are too large. The same is the case when applying 100% volumetric strain (Figure 4.30).

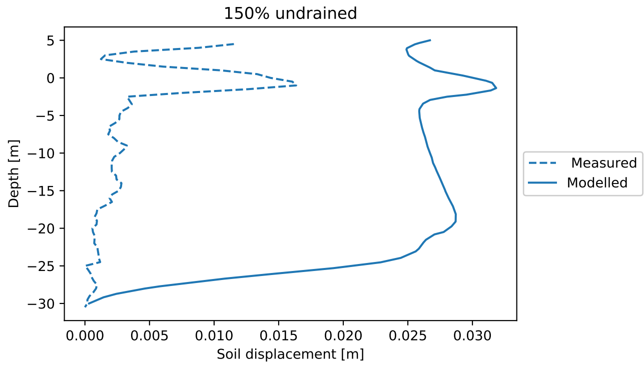


Figure 4.29: Field data Rotterdam compared with 150% volumetric strain undrained

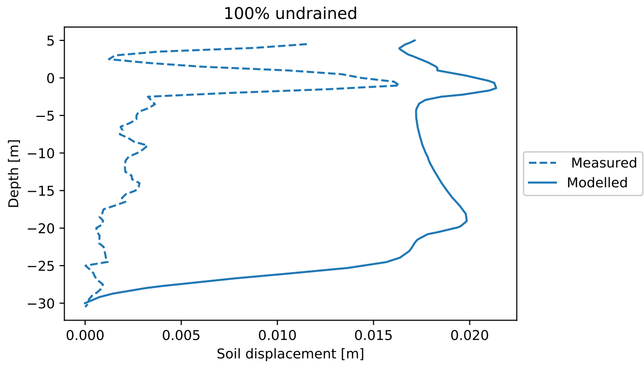
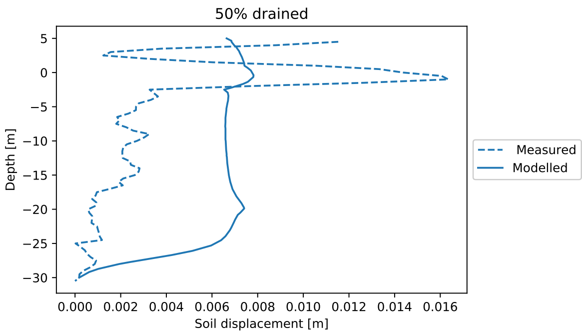
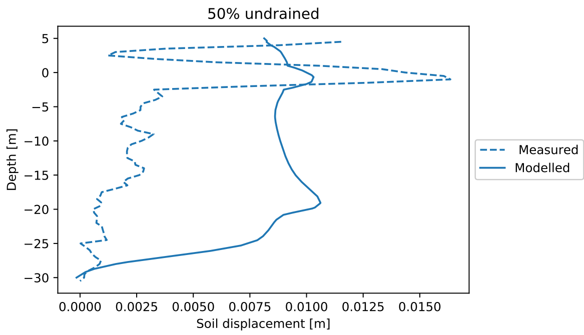


Figure 4.30: Field data Rotterdam compared with 100% volumetric strain undrained

To see which volumetric strain will give a match with the measured data, the magnitude of the volumetric strain is further decreased. For 50% volumetric strain the undrained is closer to the measured displacements(Figure 4.31b). So for this volumetric strain it is also run with drained conditions(Figure 4.31a). For the Shanghai case a lower and upper limit was suggested with the drained and undrained model. In this case could the 50% give a first estimation of the magnitude of the displacements, but the measured displacements are not clearly lying withing the drained and undrained model. An even smaller volumetric strain of 25% is tried to see if this would give better results (Figure 4.32). The drained model does fit as lower limit. However, the undrained model does not include most of the higher values.

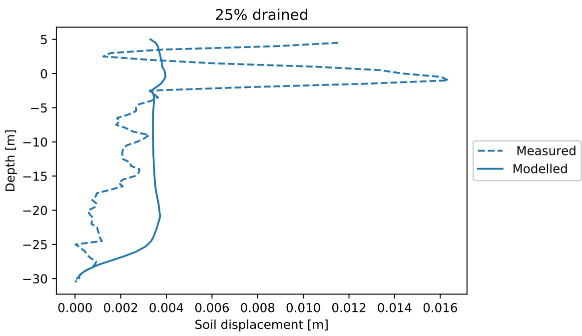


(a) Drained

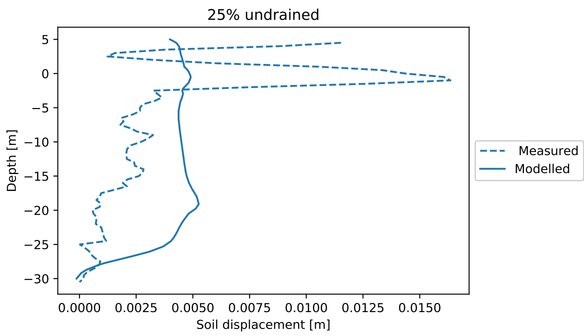


(b) Undrained

Figure 4.31: Field data compared with 50% volumetric strain



(a) Drained



(b) Undrained

Figure 4.32: Field data compared with 25% volumetric strain

4.3.3. Sensitivity analysis

The difference in soil displacements between soil layers is not represented correctly. This could be due to incorrect soil parameters in the model. To determine which soil parameters have a large influence on the soil displacement a sensitivity analysis is done. For this analysis a model is used with only one soil layer. The soil parameters from this soil layer are varied one at the time. First the MC model from Rotterdam is investigated. The graph with the results from the analysis are shown in Appendix F. The changed parameters are the stiffness, cohesion and friction angle. All these parameters do not have any influence on the modelled displacement. This counts for both undrained as drained conditions.

Also a sensitivity analysis has been executed for the HSS model. In this analysis only the influence of the stiffness is investigated. The HSS soil model uses 3 different stiffness input parameters; $E50_{ref}$, $Eoed_{ref}$ and Eur_{ref}. These are all correlated to each other, so in the sensitivity analysis the ratio between them will be kept constant. Also the shear modulus (G_0) will change with the stiffness, because this is correlated as well. The results of the analysis are shown in Appendix F. It shows that a change in stiffness does not change the modelled displacements in drained conditions. In undrained conditions a small difference is visible in the modelled displacements when the stiffness is varied. A larger stiffness gives larger displacements in the area nearby the pile. The volumetric strains that eventually are applied differ slightly from each other, which could explain the small differences in modelled displacement.

4.3.4. Discussion

For the case of Shanghai it is determined that the method with 150% volumetric strain gives the best fit with the measured data. The same method is used for the Rotterdam case. This method, however, does not give a good match with the measured displacements. The modelled displacements are larger than the measured displacements. Different values for the volumetric strain are tried to see what would give a better fit. With a volumetric strain of 25% the modelled displacement is on average a fit with the measured displacement. The modelled displacements do not show a big difference between the soil layers, which are larger in the measured displacements. This makes it difficult to find a model with the appropriate volumetric strain to fit each soil layer. For the clay layer is 100% volumetric strain a a good fit, but for the other layers is a volumetric strain of 25% better.

The model of Shanghai is not usable for Rotterdam and for Rotterdam different amounts of volumetric strains are needed. The sensitivity analysis showed the difference cannot be because of incorrect determined soil parameters. A small part can be allocated to the use of the Mohr-Coulomb soil model, which gives higher displacements than the Hardening Soil Small Strain model. The other difference between the two cases is the different type of screw pile. However, it is not clear what causes the difference between the models, so it is not possible to determine a model that would predict the soil displacement for each situation. More insights of what influences the magnitude of the soil displacements are needed.

4.4. 3D model

In this section a 3D FE analysis is done of the pile group used for the field measurements in the harbour of Rotterdam. This will give an insight on the response of adjacent piles to the soil displacements. First the soil displacements due to one pile is compared with the 2D model to see whether the soil displacements are the same in both models. After this more piles are installed to see the effect on the previously installed piles.

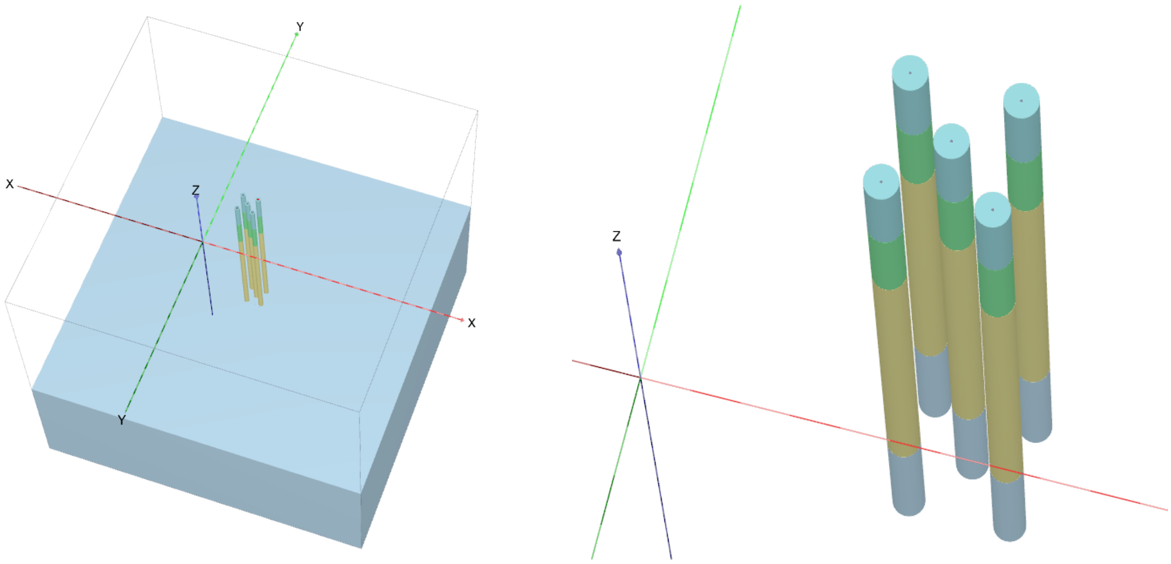
4.4.1. Method

Phases

In the 3D model the 5 piles from the field tests are modelled. For each pile one phase is needed. This gives the 3D model 6 phases in total. The installation of the piles are modelled with a 100% volumetric strain applied to the pile volume. The strain material is concrete to ensure the volumetric strain. The applied strain is again only applied in the horizontal direction. In the 3D model this is the X and Y direction and both have half of the volumetric strain assigned.

Dimensions and structures

The dimensions of the 3D models are chosen similar to the 2D axisymmetric model. The depth is 40m and the ground surface is 50m x 50m. The piles have a diameter of 700 mm and a length of 32m. The distance between the piles are the same as during the tests (see Figure 4.7). Pile 5 is in the middle ground surface, so the model is symmetrical. No beams are used in the 3D model to read the soil displacements. These were only used as an easy way to determine the soil displacement. They are of no influence on the results. However, beams are used to determine the pile displacements. They are placed in the middle of the concrete piles.



(a) Overall view model

(b) Close up piles

Figure 4.33: 3D model Rotterdam

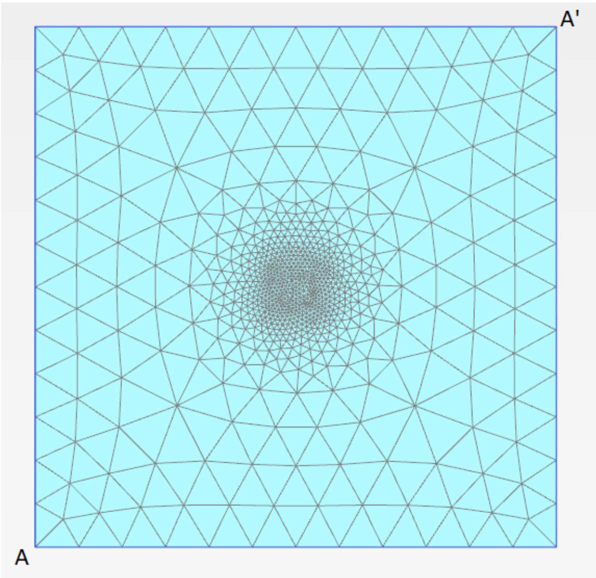
Soil and structure parameters

The soil layers and their parameters are the same as in the 2D model (Table A.4). While evaluating, the results should also be taken into account that the MC soil model is used. This means that there is no stress depended stiffness in the soil and that larger displacements are to be expected than when using the HSS soil model. Also the influence of increasing stiffness of the soil due to the previous installed piles will not be visible in the results. However, the MC model has shorter computing time, which is convenient during the development of the model.

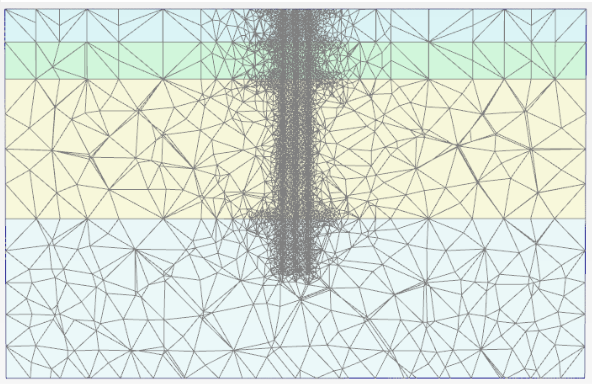
To determine the displacement of the piles a beam is placed in the middle of the pile. The beams have a stiffness and area of 0.001 times the value of the actual pile. This will keep it from giving a significant contribution to the strength of the pile, but it is easy to see the displacements of the pile. It is also possible, if needed, to read the bending moments in the pile from these beams. The modelled moments should be corrected to get the the moments of the full scale pile.

Mesh

The mesh of the 3D model exists out of 10-noded tetrahedral elements. The size of the elements are medium. This gives a model with 112032 elements and 152712 nodes.



(a) Top view



(b) Cross section A-A'

Figure 4.34: Mesh 3D model Rotterdam

Boundary conditions

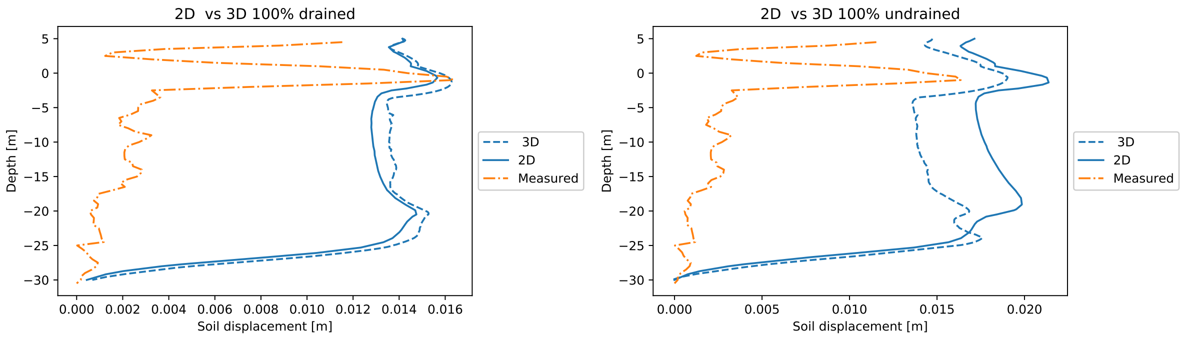
For the 3D model the default setting are used for the boundary conditions. For the displacements at the vertical boundaries this means that the displacements in the normal direction of the boundary are fixed. The movements in the other directions are free. At the top horizontal boundary all displacements are free. In the bottom horizontal boundary all the displacements are fixed. For the flow conditions all the boundaries are open, with exception of the bottom horizontal boundary. This boundary is closed for waterflow.

4.4.2. Results

Soil displacement

In Figure 4.35 the results are shown for an applied volumetric strain of 100% in drained and undrained conditions. It is clear that the displacements of 3D model does not match perfectly with the displacements of the 2D model, for both conditions (Figure 4.35). However, in the drained situation the displacements of both models are similar to each other. Comparing the models with the measured displacements, only the clay layer is a good fit. In the sand layer the displacements are larger than in the sandy clay layer in contrast to the measured displacements. The measured displacements show smaller displacements in the sand layer than in the sandy clay layer.

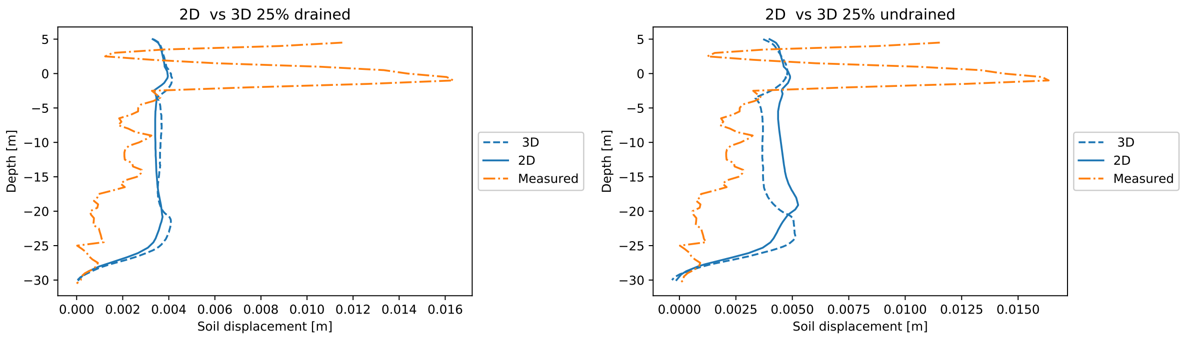
In undrained conditions more differences are visible between the 2 models. The 3D model has smaller displacements than the 2D model. The pattern in both models are similar, but 5mm difference.



(a) Drained (b) Undrained

Figure 4.35: Soil displacements 2D axisymmetric and 3D
ϵ_v=100%

With the 2D model is determined that 25% volumetric strain is a better fit with the deeper soil layers, so this is also used for comparing the 2D and 3D model (Figure 4.36). In drained conditions the 2D and 3D models are almost similar. In undrained conditions the displacements in the 3D model again are smaller than in the 2D model.



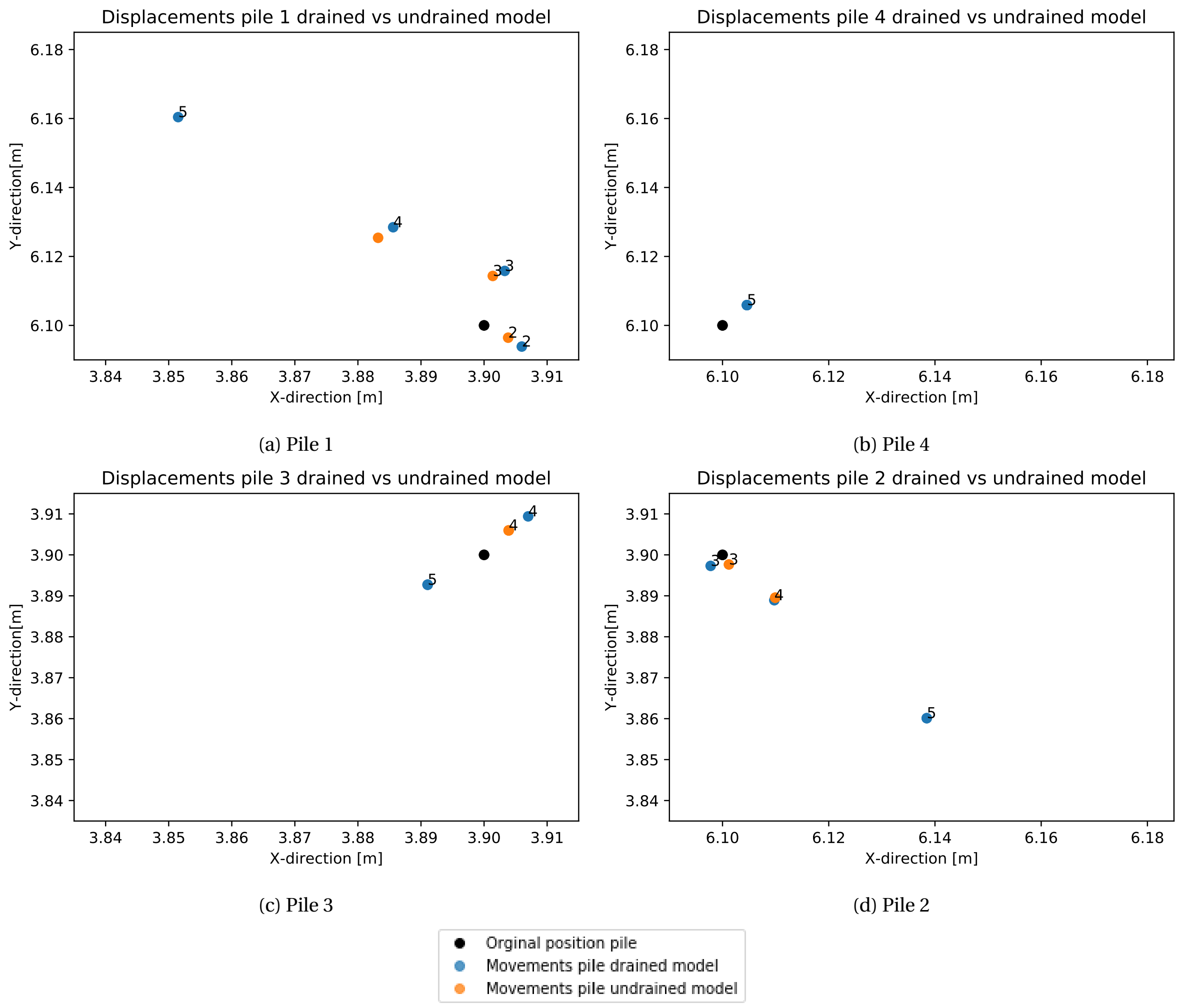
(a) Drained (b) Undrained

Figure 4.36: Soil displacements 2D axisymmetric and 3D
ϵ_v=25%

When comparing the 2D and 3D models it can be concluded that for drained conditions the displacements are similar. For undrained conditions there is a difference between the models. This should be taken into account when changing from a 2D models to a 3D model.

Pile displacement

The pile displacements per pile are shown in Figure 4.37. For each pile is determined the displacements caused by the installation of the later installed piles. The numbers indicate the location of the pile after installation of that number pile. Pile one has movements due to the installation of pile 2, 3, 4 and 5. In the undrained model an error occured for the installation of pile 5, so the pile movements due to the installation of pile 5 are not shown for this model. The results show a random direction of movement of the piles. As expected should the direction of the pile be away from the installed pile. The movement of piles 1 and 2 after installation of respectively pile 2 and 3 are towards the installed pile. In both case the installed pile is on the diagonal of those piles. The other displacements move more in the expected direction. In these graphs the average displacements of the piles are used. It does not say anything about the displacement differences along the pile.



(a) Pile 1

(b) Pile 4

(c) Pile 3

(d) Pile 2

Figure 4.37: Horizontal pile displacement undrained and drained model

4.4.3. Discussion

The soil displacements in the 3D model are not similar to the soil displacement in the 2D axisymmetric model. The displacements in the drained model come close to those in the 2D model. However, the undrained model has smaller displacements than the 2D model. The pile displacements did not give a satisfying result. The piles did not move in the expected directions. For the in diagonal from each other positioned piles is the following observed to explain the unexpected direction. The soil around the first installed pile is densified by the installation of that pile. When installing the subsequent pile, the displaced soil avoids the denser parts of the model. This result in a dominant soil displacement away from the previous installed pile. This soil movements also drags along the soil between the 2 piles, what result in the pile being displaced towards the newly installed pile. Another observation is strange patterns around the installed piles for different parameters. This probably is due to large deformations in the mesh, because of the large applied displacements. The model is not further developed and should be improved to make it usable.

5 Discussion

In this chapter all the results from the two case studies are discussed and how they relate to each other. First the observation from the the measurements from the field tests are compared and conclusions drawn from this. With the conclusion the FE analysis of both cases are studied to see if this also is visible in the results of the FE models.

5.1. Measurements

In section § 3.1 was observed that the displacements are not uniform along the depth and suggested that the soil parameters are of influence on the magnitude of the displacements. The stiffness of the soil layer is expected to have influence on the displacements. To determine the influence of the stiffness for each soil layer the average displacement is determined and plotted against the distance from the pile (Figure 5.1). From these displacements the following things can be observed:

- A lower stiffness gives higher displacements.
- A lower stiffness gives a higher decay.
- From 3 meters (6.8*d_{pile}) displacements are smaller than 10mm.
- Difference in soil displacement of max 35 mm between layers

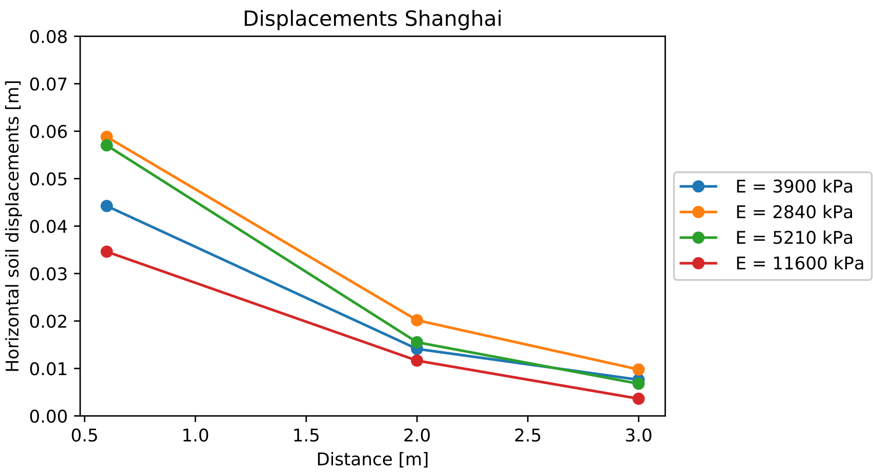


Figure 5.1: Measured soil displacements Shanghai

For the measurements in Rotterdam is the same graph made (Figure 5.2). From this graph can the following things be observerd:

- A lower stiffness gives higher displacements.
- A lower stiffness gives a higher decay.
- From 4 meters (5.6*d_{pile}) barely any displacements occur anymore.
- Difference in soil displacement of max 20 mm between layers

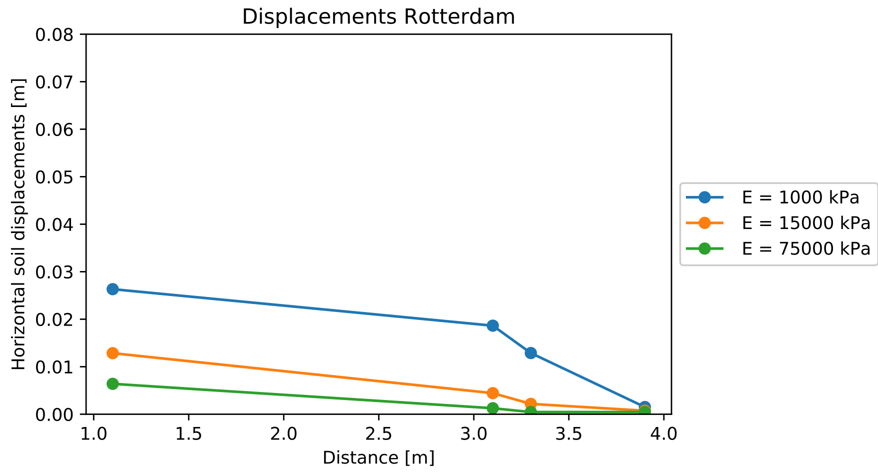


Figure 5.2: Measured soil displacements Rotterdam

Both cases show the same observations. However, the displacements of Shanghai are larger than the displacements in Rotterdam, while the Shanghai pile has a smaller diameter. Both are screwed displacement piles, but installed with different techniques. This could give a different initial displacement than expected. The volume of the casted concrete for the Rotterdam piles suggests that the pile has the desired diameter. This would also fit with the observation that all the measured values are below the CEM-values (see Figure 4.24), which should give in theory the maximum possible displacement. The displacements of Shanghai almost all exceed the CEM-value (see Figure 3.4). When adjusting the diameter of the Shanghai pile to let all the measurements fall within the CEM-values, the diameter should be at least 0.58m.

5.2. Numerical models

As was determined with the sensitivity analysis in subsection § 4.3.3, has the stiffness no influence on the modelled displacements. So the difference in the modelled displacements is due to other factors. What this factors could be is not further investigated, but with the conclusions from the sensitivity analysis is this not allocated to the difference in stiffness.

For the Shanghai case in subsection § 3.2.2 it is decided that the model with 150% volumetric strain as expansion method is the best fit with the measured displacement. For this model is determined what the decay of the modelled displacements is and whether the observations are similar to the measured displacement. From Figure 5.3 can be observed that:

- All soil layers start with the same initial displacement.
- From 3.5m (8*d_{pile}) distance all displacements are under 10mm.
- Between the soil layers there is less difference than with the soil layers in the measured displacements.
- Difference in soil displacement of max 8 mm between layers

The volumetric strain is adapted to fit the measured displacements in Shanghai. With the right volumetric strain the decay is according to the measured displacement. However, the difference between the soil layers is less clear in the modelled displacements.

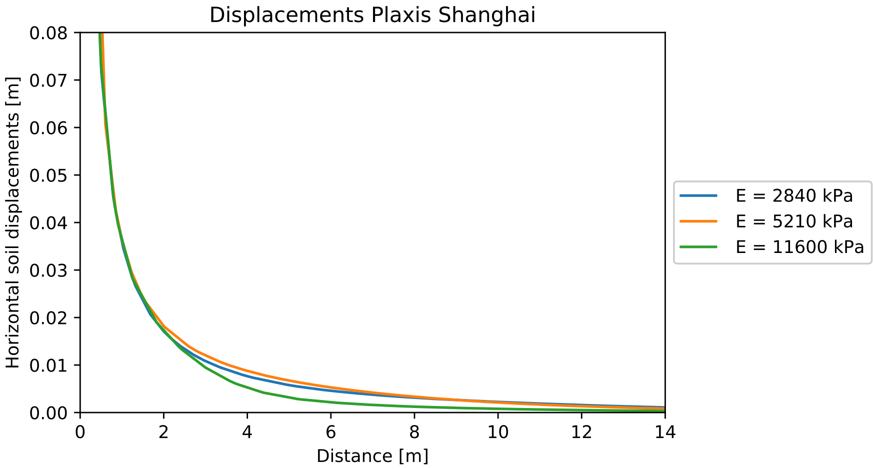


Figure 5.3: Modelled soil displacements Shanghai

For the Rotterdam case in subsection § 4.3.2 it was shown that 150% volumetric strain results in displacements are larger than measured. However, this is still used here for the comparison (Figure 5.2). From the decay in displacement from the model can be said that:

- All soil layers start with the same initial displacement.
- From 6.5m distance all displacements less than 10mm. This is a larger distance than for the measured displacement.
- Between the soil layers there is less difference than with the soil layers in the measured displacements.
- Difference in soil displacement of max 8 mm between layers

Because not the optimal volumetric strain was used, the displacements are a lot larger than the measured displacements. Also the decay of the displacements take a lot longer than with the measured displacements. This shows that choosing the correct initial displacement is crucial for a fitting model.

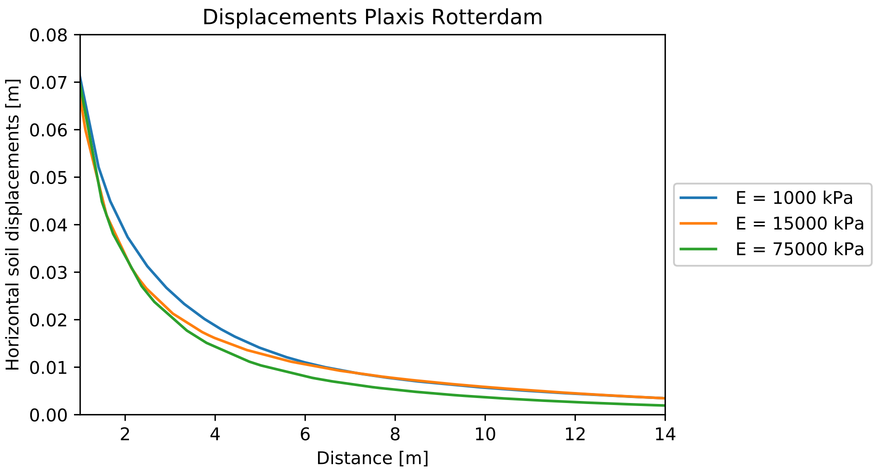


Figure 5.4: Modelled soil displacement Rotterdam

Conclusion

The measured displacements show a clear influence of the soil layers on the soil displacement. However, the finite element models show that a change in stiffness has no effect on the soil displacement, so the difference between the model and measurements should be due to something else. The applied volumetric strain does influence the magnitude of the soil displacements. The applied volumetric strain is correlated to the diameter of the pile. If combining this with the difference in displacements in the soil layers, it is suggested that the diameter of the pile varies per soil layers. When the pile is installed, a displacement is forced of at least the tube radius. After this the concrete is cast into the tube and the tube is extracted. At this point the concrete pressure will interact with the pressure of the soil. Soil layers with a low stiffness will have less resistance against the concrete, this will result in a larger diameter pile. Soils with a higher stiffness will have a larger resistance against the concrete and could decrease the diameter of the pile. The diameter of the pile is a result on the interaction of the pressure of the concrete and that of the soil. The eventual diameter of the pile will determine the effective soil displacement in that layer. Because the diameter can vary per layer, so can the soil displacement (Figure 5.5).

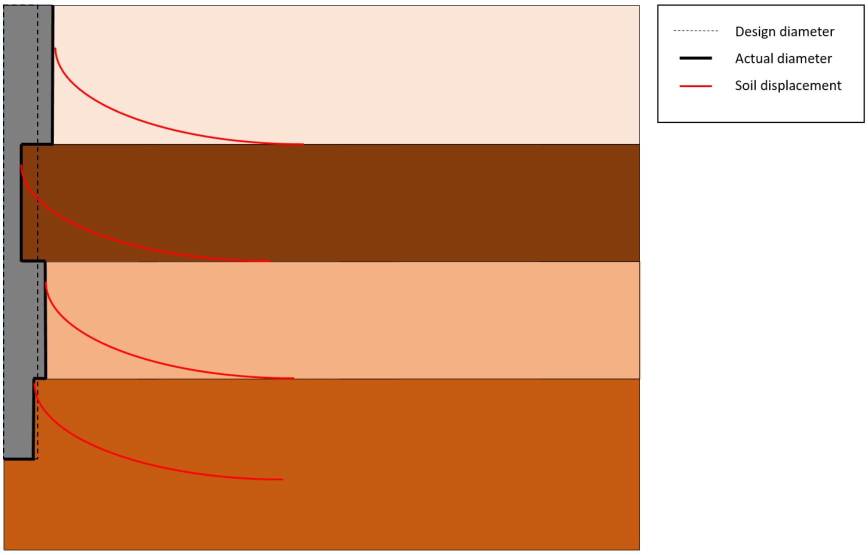


Figure 5.5: Impression of interaction concrete and soil and its effect on the soil displacement

In summary can be said from these observations of the measurements and the models that the soil displacement is depended on:

- The initial displacement at the edge of the pile.
- Stiffness of the soil
- Distance from the installed pile

In the finite element model only the volume balance of the displaced soil si taken into account, not the pressure balance between the soil and the concrete. So the model applies the same displacement on each layer, which results in an uniform pile diameter and uniform soil displacements. The pressure of the concrete could be added to the model to simulate this phenomena more accurately.

When taking another look at the pile displacements measured during the construction of 'Het Collectiegebouw' a few things can be said about that situation. Soil displacements for the largest part are decreased after 6 times the diameter distance from the pile. At Het Collectiegebouw piles are used with a diameter of 670mm, this gives an expected displacement until 4m distance from the pile. The smallest distance between piles was 1.3m, so within the range of the expected soil displacement. Another negative factor for the soil displacement is the large layer of clay and peat at that location. This layer has a very low stiffness and will give high displacements. Taken these factors into account it could have been expected that large soil displacement would occur due to the installation of the piles.

When large displacements are expected measures should be taken to reduce the displacements. The stiffness of the soil is not changeable, but the initial displacement and the distance are. Changing the initial displacement is the easiest solution, by either pre-drilling the piles or changing installation technique.

6

Conclusion & Recommendations

6.1. Conclusion

The aim of this thesis is to determine whether the installation effects of screw piles could be predicted by use of a FE analysis. The installation of screw piles causes volumetric expansion and as a consequence the surrounding soil compresses. Already installed piles are influenced by this soil movement which affects it's position or could cause deformations. This has been observed in the project Museum Boijmans in Rotterdam. Pile displacements could cause a decrease of the capacity of the pile or could cause problems with the connection between the piles and the super structure. All kind of measures could be taken to mitigate the undesirable effects of the installation of the piles, however predicting those deformations would give information to optimise the installation.

It has been chosen to use Plaxis FE models to investigate whether ground displacements due to installation effects can be predicted. The model results have been compared with datasets from measurements from China and Rotterdam.

6.1.1. Case study harbour of Shanghai

In the harbour of Shanghai tests were executed by Meng et al. (2015) to measure the soil displacements due to the installation of a screw pile. The measurements showed a clear displacement of the soil. The displacements decreased with increasing distance to the pile, as was expected from the literature. However, the values of the displacements did not match the Cavity Expansion Method (CEM). These measurements have been used to create a 2D FEM to predict the soil displacement in a better way than the CEM does. Two different methods were tried to model the expansion phase of the model. In this phase the volume of soil is pushed to the side. This is done by prescribed displacements at the edge of the pile volume or by applying a volumetric strain on the pile volume. In the model the soil displacement is measured at the same distance as has been done in the field tests in Shanghai. With the prescribed displacement method the displacements are larger than the measured data. The best match was obtained with the model in which 150% volumetric strain was used as expansion phase. The model in drained conditions could be used as lower limit and the undrained model as upper limit.

6.1.2. Case study harbour of Rotterdam

Measurements

From these results the question was if this model is also suitable for projects in The Netherlands. Because no data was available for verifying this model with Dutch subsoil, tests were executed in the harbour of Rotterdam to measure the soil displacements around the pile. The measurements again correspond with the literature that the displacement decreases with increasing distance from the pile. Also the expected direction of movements are met. Subsequently for these measurements the difference between the soil layers has been studied. Three main soil layers are identified from CPT data and compared with the displacements according to the CEM. This showed that there is a big difference between the soil layers. The clay layer has the largest displacement. For this layer the CEM value is not a bad suggestion. When looking in the sand layer the displacements are a lot smaller than the suggested displacement with the CEM. The values of the CEM are a large overprediction and can be corrected with a factor of 0.06. The in between lying sandy clay layer has displacements smaller than the clay layer, so also smaller than the CEM values. The correction factor for the sandy clay layer is 0.15.

Modelling

With the soil layers of the harbour of Rotterdam a new 2D axisymmetric model is made, to see if the method for Shanghai is also applicable here. The results show the same pattern in soil displacement as in the Shanghai model, but the size of the displacements does not match the measured displacements. When using 150% volumetric strain the modelled displacement were too large. Smaller volumetric strains were thereafter modelled to find a better fit with the measured displacements. This resulted in an optimal volumetric strain of 25%.

Another part of this research was to see whether the effect on adjacent structures could be modelled. In this research this are the adjacent piles in the pile group. For this a 3D model was made to look into the effects of the soil displacements. Because the measurements for the pile displacement did not give usable data in this field tests, this model can only be judged qualitatively. The expectation is that the pile will move away from the pile that is installed. The expansion phase is modelled with an volumetric strain of 100%. After each installed pile the displacement of the already installed piles is noted. The modelled pile displacements of the piles do not follow the expected direction.

6.1.3. General conclusion

With the results of the field tests and the numerical models it concluded that the soil displacement is depends on:

- The initial displacement at the edge of the pile.
- Stiffness of the soil
- Distance from the installed pile

The stiffness of the soil layers determine the actual pile diameter in that layer, which determines the initial displacements in that layer. The farter away from the pile, the smaller the soil displacements will be.

When a numerical model is made for a new project the initial displacement of the different soil layers should be determined. With this the model gives an accurate prediction of the soil displacement. However, the current 3D model is not correct in predicting the displacement of adjacent piles.

6.2. Recommendations

For better insights on the installation effects of screw piles on adjacent further research is needed. In subsection § 4.2.4 are more specific recommendations given for when the field tests are repeated. For future research the following recommendations are suggested.

- The effect of different soil layers on the diameter of the pile should be investigated. This will determine if this indeed is the reason for the difference in soil displacement in the soil layers. This could be done with measurements closer to the pile or by excavating the pile. Temperature measurements with optical fibre could also say something about the concrete coverage of the reinforcement cage.
- The 2D model should be expanded with a phase in which the pressure of the casted concrete is included. With this phase should the interaction between the concrete pressure and soil pressure be modelled and its influence on the soil displacement.
- The current 3D finite element model cannot be used to predict the pile displacements. This should be developed further to get accurate predictions. A finite element analysis is not the best method for large displacements. Other methods could be investigated to create a better model.
- Because no data is available of pile displacements and deformations to verify the 3D model, this should be obtained from new tests. Optical fibres are still a good options to measure this, but tests should be specially done for this purpose. The results from these tests should be used to improve the 3D model for pile displacements.
- Ones the pile displacements is modelled correct, it should also be determined which magnitude of pile displacements or deformations are acceptable. This will determine when measures are needed to reduce the soil displacements and its effect on adjacent structures.

Bibliography

Eurocode 7. *Geotechnisch ontwerp - Deel 1: Algemene regels*. 2018.

M.M. Baligh. Strain path method. *Journal of geotechnical engineering*, 111:1108–1136, 1985.

P. Basu and M. Prezzi. Design and applications of drilled displacement piles, 2009.

W. Broere and A. F. van Tol. Modelling the bearing capacity of displacement piles in sand. *Proceedings of the Institution of Civil Engineers-Geotechnical Engineering*, 159(3):195–206, 2006.

J.P. Carter, M.F. Randolph, and C.P. Wroth. Stress and pore pressure changes in clay during and after the expansion of a cylindrical cavity. *International journal for umerical and analytical methods in geomechanics*, 3:305–322, 1979.

J.P. Carter, J.R. Booker, and S.K. Yeung. Cavity expansion in cohesive frictional soils. *Géotechnique*, 36:349–358, 1986.

L.T. Chen and H.G. Poulos. Piles subjected to lateral soil movements. *Journal of geotechnical and geo-environmental engineering*, pages 802–811, 1997.

M.K. Chong. Soil movements due to displacement pile driving. In *Proceedings of the international conference on case histories in geotechnical engineering*, 2013.

Digitilt. Digitilt inclinometer probe manual, 2011.

J. Dijkstra, W. Broere, and F. Tol. Numerical investigation into stress and strain development around a displacement pile in sand. *Numerical Methods in Geotechnical Engineering*, 01 2006.

John Dunnicliff and Gordon E Green. *Geotechnical instrumentation for monitoring field performance*. John Wiley & Sons, 1993.

H. K. Engin, R. B. J. Brinkgreve, and A. F. Van Tol. Simplified numerical modelling of pile penetration - the press-replace technique. *Internation journal for numerical and analytical methods in geomechanics*, 39 (15):1713 – 1734, 2015.

D Joseph Hagerty and Ralph B Peck. Heave and lateral movements due to pile driving. *Journal of Soil Mechanics & Foundations Div*, 1971.

A. Marinucci and M. Chiarabeli. The benefits of displacements piles, 2016. URL `https://soilmecna.com/2016/11/18/the-benefits-of-displacement-piles/`.

A. Marinucci and M. Chiarabelli. The history of displacement piles, 2016. URL `https://soilmecna.com/2016/11/18/the-history-of-displacement-piles/`.

K. R. Massarsch. Soil movements caused by pile driving in clay. Technical report, 1976.

K. Rainer Massarsch and Carl Wersäll. Cumulative lateral soil displacement due to pile driving in soft clay, 02 2013.

Z. Meng, J. Chen, L. Zhang, J. Wang, and J. Yao. Field tests to investigate the installation effects of drilled displacement piles with screw-shaped shaft in clay. *Journal of geotechnical and geo-environmental engineering*, 141, 2015.

MVRDV, 2019. URL `https://www.mvrdv.nl/projects/10/depot-boijmans-van-beuningen`.

Q. C. C. I. Ni, C. C. Hird, and I. Guymer. Physical modelling of pile penetration in clay using transparent soil and particle image velocimetry. *Geotechnique*, 60(2):121, 2010.

Omnisens. *Training on Omnisens vision dual*, 2018.

J. P. Oostveen and J. A. G. Kuppers. Pile driving of soul-displacing piles through soft soils. *Proceedings of the 11th internation conference on soil mechanics and foundation engineering*, 1985.

Andrew C Palmer. Undrained plane-strain expansion of a cylindrical cavity in clay: a simple interpretation of the pressuremeter test. Technical report, BROWN UNIV PROVIDENCE RI DIV OF ENGINEERING, 1971.

J. L. Pan, A. T. C. Goh, K. S. Wong, and C. I. Teh. Ultimate soil pressures for piles subjected to lateral soil movements. *Journal of Geotechnical and Geoenvironmental Engineering*, 128(6):530–535, 2002.

Plaxis. *Reference manual 2018*, 2018.

M.F. Randolph and C.P. Wroth. An analytical solution for the consolidation around a driven pile. *International journal for umerical and analytical methods in geomechanics*, 3:217–229, 1979.

M.F. Randolph, J.S. Steenfelt, and C.P. Wroth. The effect of pile type on design parameters for driven piles. In *Proceedings of the 7th european conference on soil mechanics and foundations engineering*, pages 107–114, 1979.

Responder. The joy of optical fibres, 2018. URL `https://www.erc-responder.eu/2018/11/16/the-joy-of-fibre-optics/`.

C. Sagaseta and A.J. Whittle. Prediction of ground movements due to pile driving in clay. *Journal of geotechnical and geo-environmental engineering*, 127:55–66, 2001.

C. Sagaseta, A. J. Whittle, and M. Santagata. Deformation analysis of shallow penetration in clay. *Internation journal for numerical and analytical methods in geomechanics*, 21(10):687 – 719, 1997.

SaVe. Betonnen spoorviaduct theemsweg palenplan proefpaalneming. Technical report, 2018.

W.D. Wang, Q. Li, Z.H. Xu, and J. Zhang. Determination of parameters for hardening soil small strain model of shanghai clay and its application in deep excavations. In *Proceedings of the 19th International Conference on Soil Mechanics and Geotechnical Engineering*, 2017.

H. S. Yu. *Cavity expansion methods in geomatics*. Kluwer academic publishers, 2000. ISBN 0-412-79990-1.

Appendices

A
Plaxis input parameters

Table A.1 HHS parameters Shanghai soil layers

Soil layer		1 Filled soil	2 Clay	3 Silty clay	4 Very soft silty clay	5 Clay	6 Hard clay	7 Sandy silt
γ	$[kN/m^3]$	19.5	18,6	17,5	16,6	17,9	18,3	20
e_{init}	[-]	0,75	0,89	1,13	1,43	1,05	0,93	0,63
$E50_{ref}$	[kPa]		6,32E+03	3,90E+03	2,84E+03	5,21E+03	1,16E+04	1,94E+04
$Eoed_{ref}$	[kPa]		4,21E+03	2,60E+03	1,89E+03	3,47E+03	7,71E+03	1,29E+04
Eur_{ref}	[kPa]		4,17E+04	2,57E+04	1,87E+04	3,44E+04	7,63E+04	1,28E+05
power	[-]		0,8	0,8	0,8	0,8	0,8	0,5
C	[kPa]		20	10	12	9	14	0
ϕ	[deg]		18	20,5	15,6	16,2	20,5	33
ψ	[deg]		0	0	0	0	0	3
$\gamma_{0.7}$	[-]		2,70E-04	2,70E-04	2,70E-04	2,70E-04	2,70E-04	2,70E-04
$G0_{ref}$	[kPa]		9,48E+04	7,91E+04	5,22E+04	1,12E+05	1,94E+05	2,10E+05
k_x	[m/day]		1,21E-05	1,56E-04	2,07E-05	1,21E-04	3,72E-05	6,48E-02
k_y	[m/day]		2,59E-05	5,96E-04	5,36E-05	2,16E-04	7,17E-05	4,23E-02

Table A.2 Properties plate

Material type	Elastic
EA1	1.000 kN/m
EA2	1.000 kN/m
EI	0.01E-6 kNm^2/m
d	0.364E-3 m
ν	0.000
Rayleigh α	0.000
Rayleigh β	0.000

Table A.3 Concrete parameters

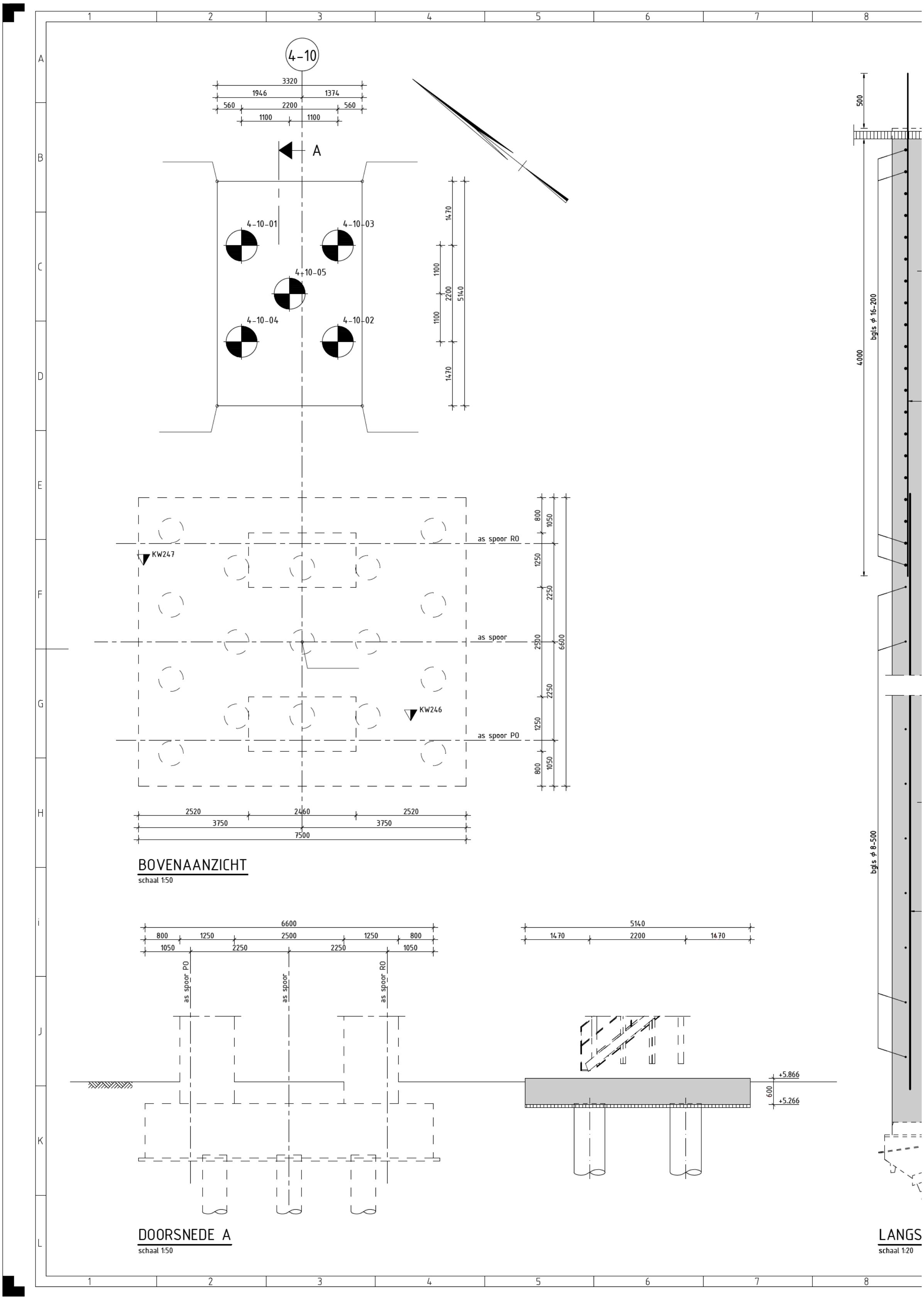
Material set		
Material model	Linear Elastic	
Drainage type	Non-porous	
General properties		
γ_{unsat}	24,00	kN/m^3
Stiffness		
E	30,00E6	kN/m^3
ν	0.1	

Table A.4 Soil parameters Harbour of Rotterdam

Soil layer	γ_{unsat} [kPa]	γ_{sat} [kPa]	C [[kPa]	ϕ [°]	ψ [°]	E [MPa]	ν [-]
Top layer	17	18	0	22.5	0	3	0.25
Clay	14	14	0	17.5	0	1	0.4
Sandy clay	20	20	0	25	0	15	0.2
Sand	21	21	0	35	0	75	0.3

B

Pile design Rotterdam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as spoor
as spoor P0
KW247
KW246
BOVENAANZICHT
schaal 1:50
DOORSNEDE A
schaal 1:50
bgls ϕ 16-200
bgls ϕ 8-500
+5.866
+5.266
LANGS
schaal 1:20

PAALRENVOOI grondverdringende schroefpaal met groutinjectie en verloren punt (teruggetrokken casing)								
MERK	Type	Afmetingen (mm)	Theoretische Lengte (mm)	PaalPuntNiveau (m t.o.v. N.A.P.)	Helling	Stramien	Paalnummers	Aantal
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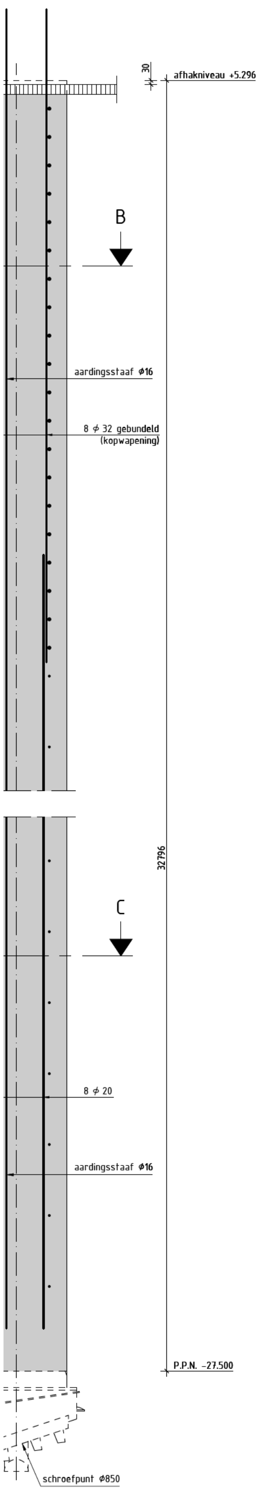
							TOTAAL AANTAL	5



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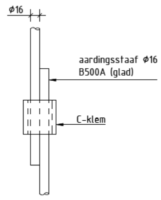

DETAIL AARDINGSSTAAF

schaal 1:5

principedetail t.b.v. doorkoppeling

NEN-EN-1992-1-1		laslengte			
Toe te passen cement	CEM III/B42.5 LH HS		n_1 = 0,7	n_1 = 1,0	n_1 = 0,7 - 'slechte' omstandigheden: h >250, d =250; h >600, d =300
Sterkteklasse	C30/37	⌀ 12	645 mm	455 mm	
Milieuklasse	XC2	⌀ 16	860 mm	605 mm	
Dekking (tenzij anders vermeld)	90mm Afwijkende dekking : c= ..mm; △C dev = 10mm	⌀ 20	1075 mm	755 mm	n_1 = 1,0 - 'goede' omstandigheden: h ≤250
		⌀ 25	1345 mm	940 mm	
Aanduiding vloer- en wand-wapening	Wapening: B500B; Notatie: ⌀	⌀ 32	1790 mm	1255 mm	

OPMERKINGEN TBV WAPENING:
- Alle overlappingslassen zo veel mogelijk verspringend aanbrengen
- In de hoeken van de beugels altijd staven aanbrengen, staven in de hoeken van de beugels gaat boven de h.o.h.-afstand van de staven.
- lassen aan wapening, (ook hechtlassen) is alleen toegestaan indien voorgeschreven door constructeur
- Hulpstaven zijn toegestaan onder bepaalde voorwaarden, aan te geven door de constructeur

OPMERKINGEN:

- peilmaten in m t.o.v. N.A.P., coordinaten in m, overige maten in mm
- hoeken in het 360-graden stelsel
- maatvoeringsniveau op b.k. werkvloer (=o.k. fundatie)

VERKLARING:

- gewapend beton C30/37
- werkvloer
- toekomstig
- nr. sondering

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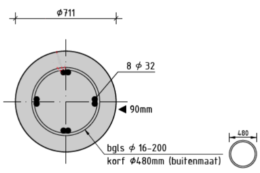

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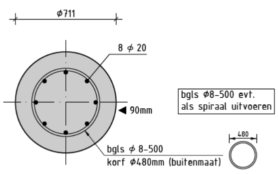

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0.2	2018-09-06	Versie t.b.v. beoordeling door SaVe	DCa	SSc	MKr
0.1	2018-09-06	Interne controle	DCa	SSc	MKr

PROJECT :

REALISATIE ONDERBOUW SPOOR THEEMSWEGTRACÉ

Ministerie van Infrastructuur en Milieu

Port of Rotterdam

Medegefinancierd door de Europese Unie
De financieringsfaciliteit voor Europese verbindingen

HOOFDAANNEMER : SaVe Samen Verbinden

CONSULTANT : wagemaker

Postbus 412
5240 AK Rosmalen
Burg. Burgerslaan 44/30
5245 NH Rosmalen
Tel. 073 521 64 00
www.wagemaker.nl

DEELTRACE: **DEELTRACE 4**

OPGESTELD : DCa | GOEDGEKEURD : SSc | VRIJGEGEVEN : MKr

ORG	PROCES	SUBPR	WP	DOC.TYPE	NUMMER	INDEX
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SaVe VOF

Oplevering

Deze tekening vervangt:

Betonnen spoorviaduct Theemsweg
Palenplan proefpaalneming
Fundatie trappenhuis stramien 4.10

Rtzgk - Mvt

21.200 24.650

Formaat A1 | Schaal 1:50 | Geocode 950 | Versie 1.0

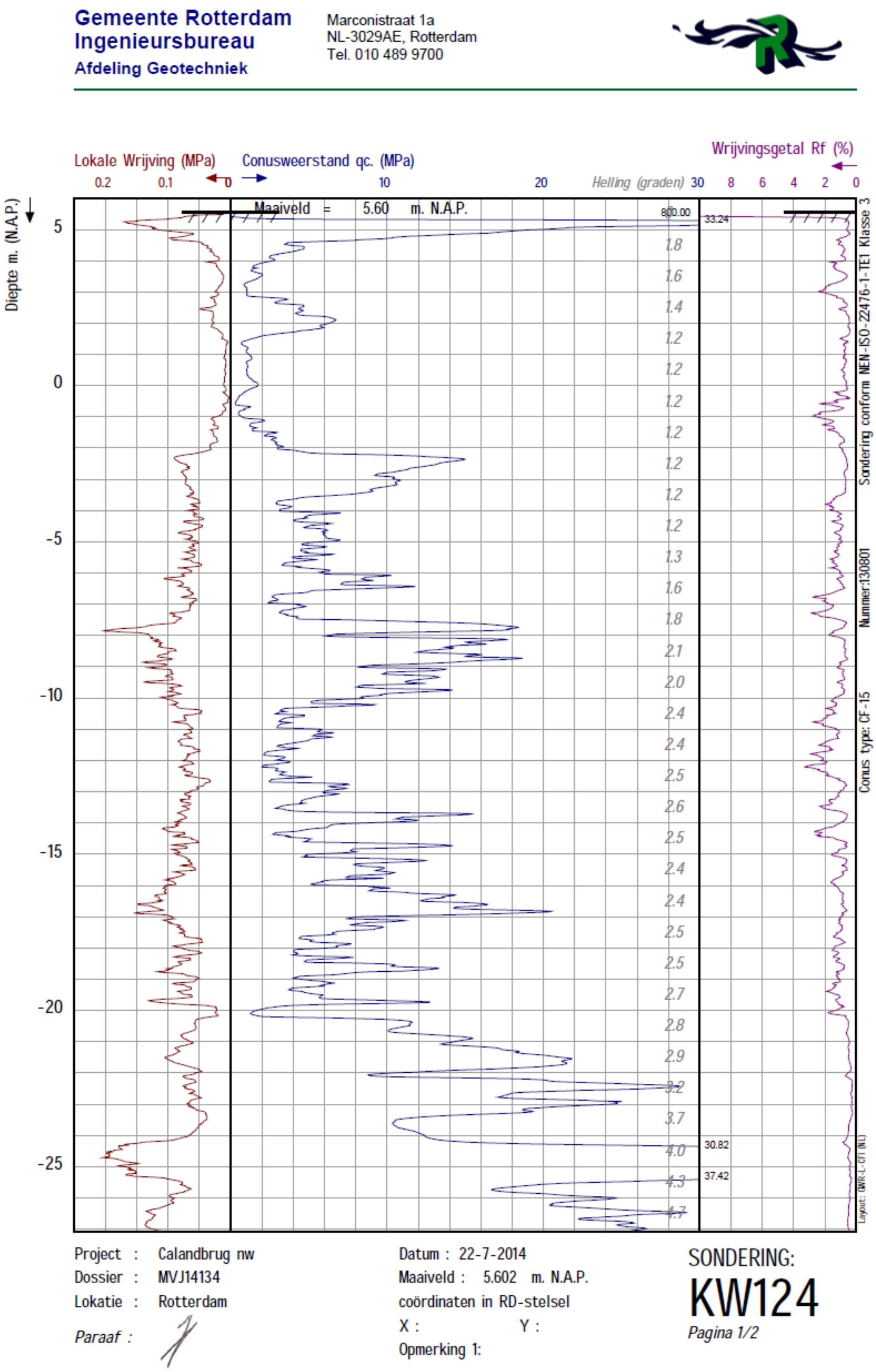
Uitvoerings Ontwerp

Details

ProRail

C

CPT Theemsweg


Gemeente Rotterdam
Ingenieursbureau
Afdeling Geotechniek
Marconistraat 1a
NL-3029AE, Rotterdam
Tel. 010 489 9700
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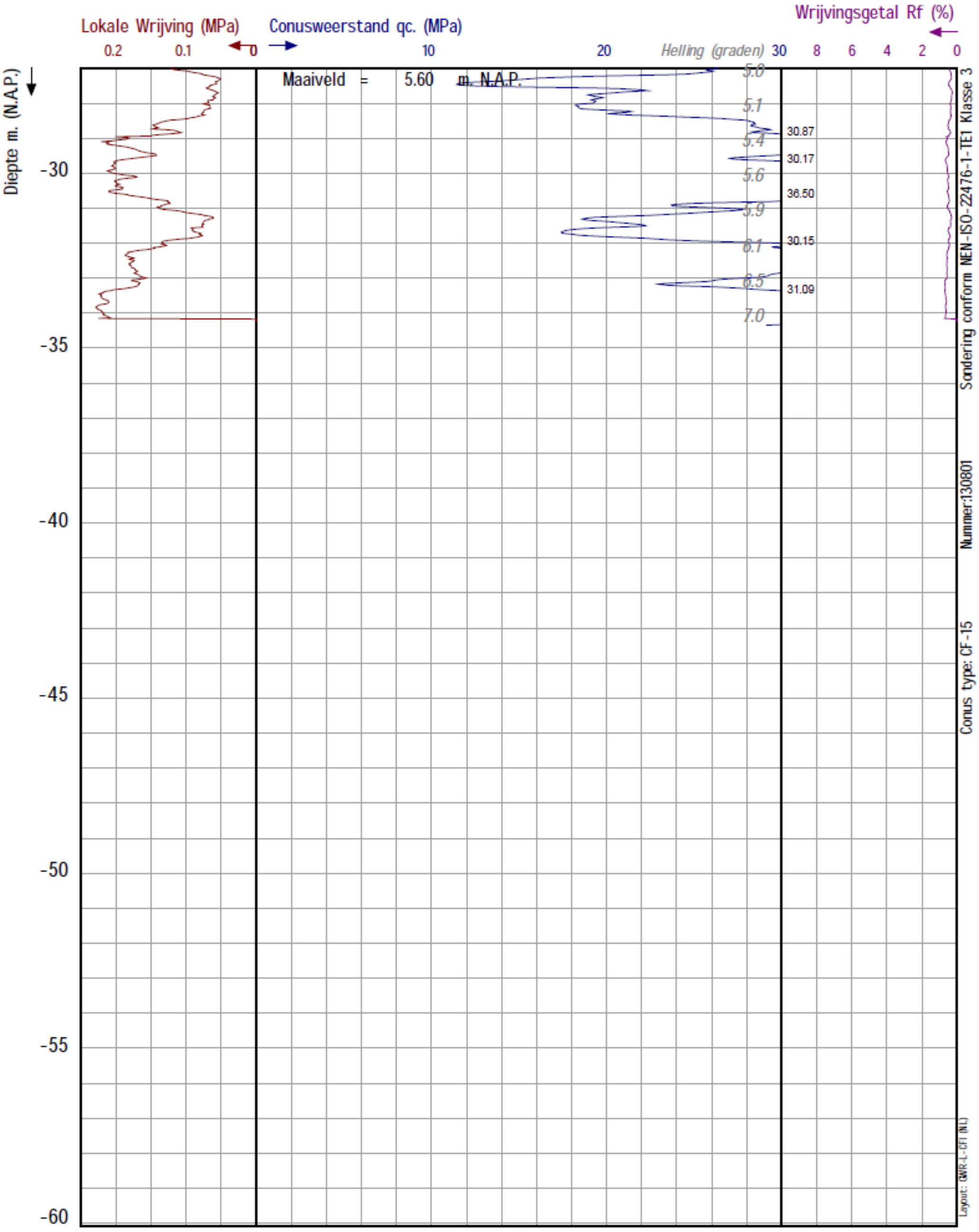
Conusweerstand qc. (MPa)
Wrijvingsgetal Rf (%)
Helling (graden)
Diepte m. (N.A.P.)
Maaiveld = 5.60 m. N.A.P.
Sondering conform NEN-ISO-22476-1-TE1 Klasse 3
Nummer:130801
Conus type: CF-15
Project : Calandbrug nw
Dossier : MVJ14134
Lokatie : Rotterdam
Paraaf :
Datum : 22-7-2014
Maaiveld : 5.602 m. N.A.P.
coördinaten in RD-stelsel
X : Y :
Opmerking 1:
SONDERING:
KW124
Pagina 1/2

Gemeente Rotterdam
Ingenieursbureau
Afdeling Geotechniek

Marconistraat 1a
NL-3029AE, Rotterdam
Tel. 010 489 9700



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Dossier : MVJ14134
Lokatie : Rotterdam
Paraaf :

Datum : 22-7-2014
Maaiveld : 5.602 m. N.A.P.
coördinaten in RD-stelsel
X : Y :
Opmerking 1:

SONDERING:
KW124
Pagina 2/2

D

Logboek field tests

Day 0
The inclinometers are installed 4 (HB2) and 3 (HB1) days before start of the pile installation. Usually a week is needed to stabilise the tubes, but there was not enough time for this. So to establish the baseline, the tubes are measured only 3 days in advance. The tubes will be measured again on the first day of installation before the pile is installed to see if the measurements line up with the previous baseline measurements.

Day 1
The first pile has an inclinometer and optical fibres attached to the reinforcement cages. The overlength of the fibres are attached to the inclinometer, so they are easy to reach once the concrete is poured. The installation process went smoothly until the tube was pulled. While the tube was pulled up, the reinforcement cage got pulled along. This resulted in the reinforcement cage sticking out 2 meters above ground level(Figure D.1. This seemed like no problem for the measuring equipment, however when measuring the inclinometer, the measuring rod could not go further trough the tube at 12 meters depth. This could be due to a bend in the tube, but smaller objects could not pas either, so the expectations was that the tube is partly filled with concrete. This is only explained if the top and bottom cage are separated during the extraction. This resulted in the inclinometer tube being pulled apart as well and concrete entering the tube. So the rest of the week this inclinometer only measured up to 12 meters depth.

Figure D.1: Image of the reinforcement cage being pulled up

The optical fibres were easily located, because they were still attached to the inclinometer. When connected to the interrogator, it turned out that the fibre of the bottom cage was broken. Both ends were tried single ended, but this didn't result in an significant length of measurable fibre. This could be the result of the cages being pulled apart. So this fibre was written of. The fibre from the top cage also had a break. However, single ended it was possible to read 50m of the fibre. The inclinometers located in the ground also experienced some problems during the first day. The drilling machine used for the pile installation choose a poor spot for its support; on top of inclinometer 2. This resulted in the tube being crushed en pushed down by around 20cm (Figure D.2). When the tube was measured, the rod could not continue after 5 meters. Most likely due to the combination of soil forces and weight of the drilling machine, the tube was bend too much. A smaller object could pass beyond the 5 meters all the way to the bottom of the tube. Inclinometer 1 was not subjected to any weight of the machine and had no other problems. The entire tube could be measured.

Figure D.2: Image of the damages inclinometer tube

Day 2

In pile 2 no inclinometer was placed, only fibres. So the overlength of the fibres should be attached to something else. For this a PVC tube was used. The PVC tube is attached to the top of the cage and the fibre was pulled trough the PVC tube(Figure D.3). The fibres from the bottom cage were coiled and attached to the the outside of the PVC tube.

This pile did not have any problems during installation. The reinforcement cages stayed in place during the extraction of the tube. Unfortunately the optical fibre did not survive the pouring of the concrete. The fibre of the bottom cage was never recovered. It was pulled down with the concrete being poured from the top. The PVC tube with the fibres of the top cage was recovered. However, the tube was damaged, chips were broken of the top of the tube. Because of this the damaged tube had sharp edges which had cute through the fibres. In the hope that below the damaged part the fibre was still viable, the PVC tube was pulled of the fibres. With a little force the PVC tube was pulled out, but it also took a long part of the fibre along, which was apparently also broken further down. This all resulted in no usable fibres in pile 2. The inclinometers in the ground were measured again afterwards. Inclinometer 1 was still measurable till 35m depth. Inclinometer 2 was measurable till 5m depth.

Day 3

Pile 3 is similar to pile 2. No inclinometer attached, only optical fibres. To make sure that the PVC tube will not cut the fibres again, the fibres were attached on the outside of the PVC tube. Installation of the pile went according to plan again. Unfortunately the fibres were not retrieved. The top of the pile was freed from the concrete, to see if the fibres were a bit further down. This did not resulted in finding the fibres. The fibres from both cages were therefore unusable. All the inclinometers still had the same measurable depth as the days before.

Day 4

Figure D.3: Image of the extension made with PVC tube

In the original plan the 4th pile had only optical fibres along the cages. However, because most of the optical fibres were broken and the inclinometer tube in pile 1 as well, it was decided to place another inclinometer tube in pile 4. Pile 2 and pile 3 did not experience any problems with the cage being pulled along, so the expectation was that the inclinometer tube would survive the installation this time. To increase the chances of an undamaged tube, the tube was filled with water. This will cancel out part of the concrete pressure on the tube. For the optical fibre a new approach was needed as well. The fibres needed to be protected against the falling concrete. Here for a small umbrella was made from steel attached to the reinforcement cage. Underneath the umbrella are the fibres coiled and tie wrapped to the cage (Figure D.4).

Both new approaches were successful. The inclinometer tube was measurable until the depth op 32m, which is the length of the piles. The umbrella was still in its place after the pile was installed. The fibres were very easily retrievable because of this and less damaged. To see if the fibres were not broken they needed to be spliced with a pig tail and connected to the interrogator. The fibre from the bottom cage was broken, but it was possible to get 200 meters of signal when using single ended measurements. The first 100m is the extension cable, but the other 100m is the fibre inside the pile. This is almost the entire length of the fibre in the pile (With the cage having a length of 17m and the fibre passing the cage 6 times. This gives a total length of 102m). The overlength of the top cage was not long enough to connect with the pig tail. Because the concrete was still wet, the equipment to make the splice could not be placed close enough to the pile.

With inclinometer 2 there was a nice surprise after the installation of pile 4. The measuring rod was able to pass through the tube beyond 5m depth. Apparently the tube got pushed back far enough the let the rod through again. Inclinometer 1 is still measurable its entire depth. **Day 5**

On pile 5 no measuring equipment was attached. The optical fibres of pile 4 needed to be connected, so it could measure the displacement during the installation of pile 5. When connecting the fibres to the interrogator to start the measurements, the fibre from the bottom cage did not have 200m of signal anymore. Most likely the splice between the pig tail and the fibre was damaged during the night. Attempts were made to recover the damage, but due to limited time this did not succeed. It was decided that the splice would be repaired after the installation of pile 5. With the measurement before and after it is still possible to see the total strain in the pile. Only the development of the strains during installation of the adjacent pile is missed.

The installation of pile 5 was the most critical of all installations. The soil was already stressed a lot due

Figure D.4: Image of the umbrella as protection for the fibres

to the installation of the previous piles, this could make the installation of pile 5 more difficult. During the screwing of the tip and tube it was already clear that it went less smooth than the previous piles. The last meters went very difficult. The previous piles took around 40 minutes to reach the required depth, for pile 5 70 minutes were needed. When the required depth was finally reached, the next challenge was the extraction of the tube. The first meter went quit easy, but after this it got more difficult and after 3 meters it got stuck. Several attempts were made to get the tube out with the drill truck itself, but this was not successful. A hydraulic press was already on the premises as a back up to help with the extraction. Before the hydraulic press got towed in, it was possible to measure inclinometer 2 and 3. Inclinometer 2 was bended too much again, so the measuring rod could not pass the 5m depth again. Inclinometer 3 is still measurable until 12m depth. The other inclinometers were blocked by drilling equipment. The hydraulic press did not get the tube out, so over several days different attempts were done to get the tube out.

Day 12

After a week of different attempt the tube was still stuck in the ground. It had been decided to leave it in the ground and cut off the top. On day 12 there was the first opportunity to measure the inclinometers and fibres. Because the optical fibre equipment was not available this day, this was postponed to day 15. Inclinometer 1, 3 and 4 had the same measurable depth as the previous days. Inclinometer 2 was not found anymore. Apparently it was in the way during the extraction attempts, so the casing was removed. The tube that stayed behind without the casing was covered with sand and not found.

Day 15

The fibres of pile 4 were damaged by all the work done in the last week. The splices between the fibres from the bottom cage and the pig tails were done again. The spices were fixed and it was possible again to read the fibre, single ended, for 200m. This can be compared with the measurements from before pile 5 was installed. Now that the concrete of the pile was hardened, it was also possible to connect the fibres of the top cage. The data is not useful anymore for the measuring of strain, because there is no baseline. However, it is still interesting to see if the fibres were damaged or not. It was not possible to read the fibre double ended, but single ended mode could measure 60m.

E
Inclinometer results

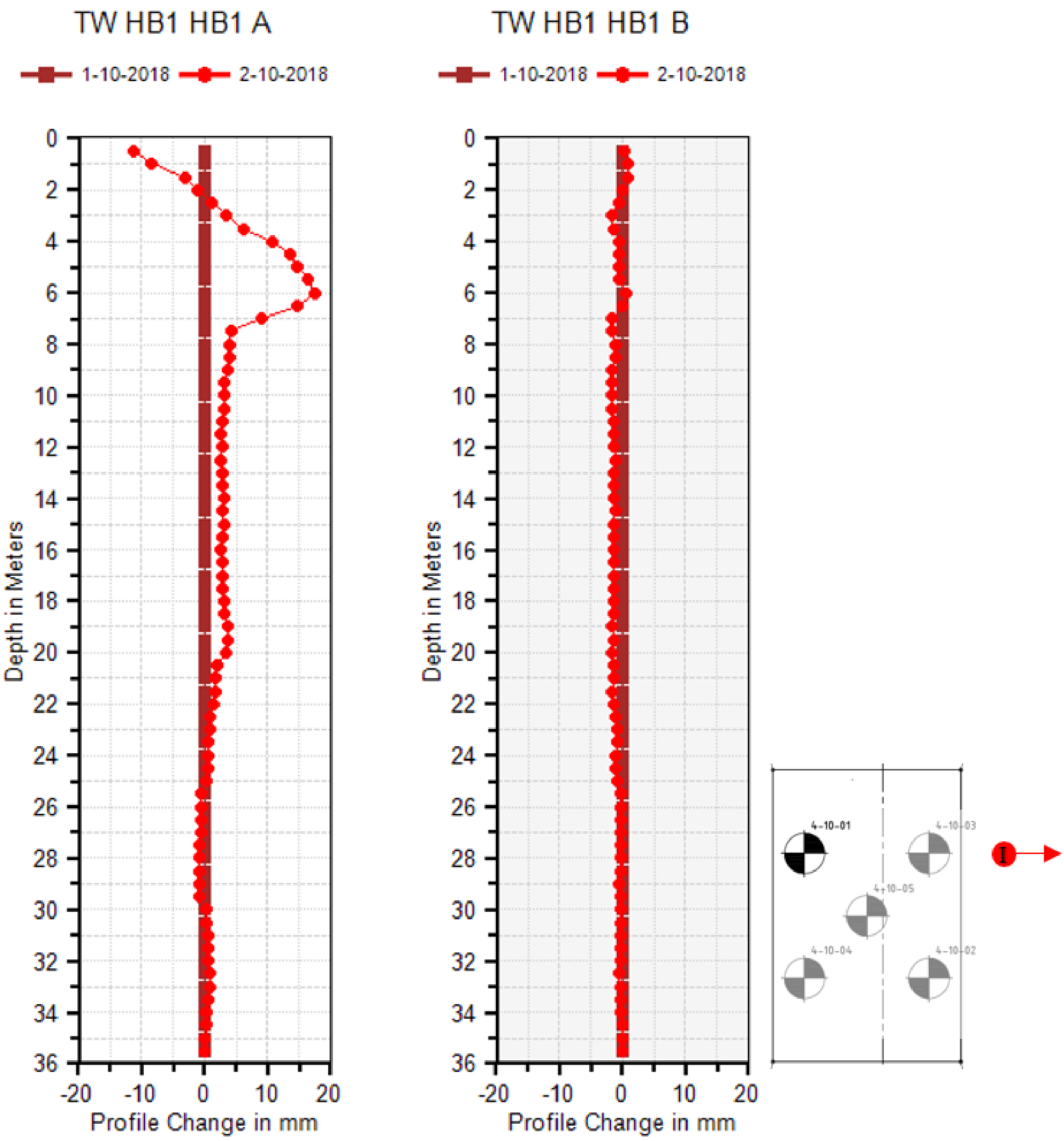


Figure E.1: Profile change after installation pile 1

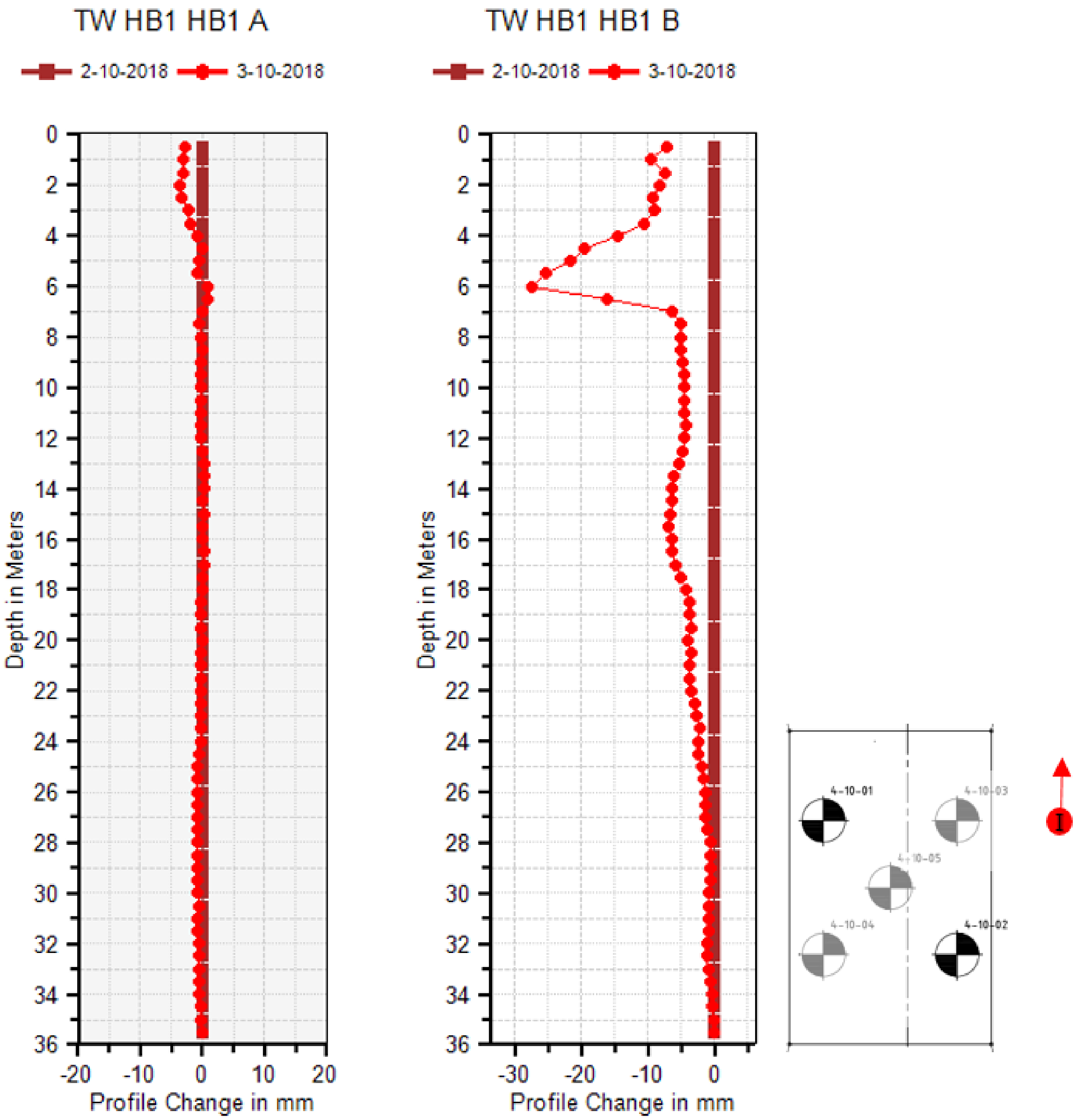


Figure E.2: Profile change after installation pile 2

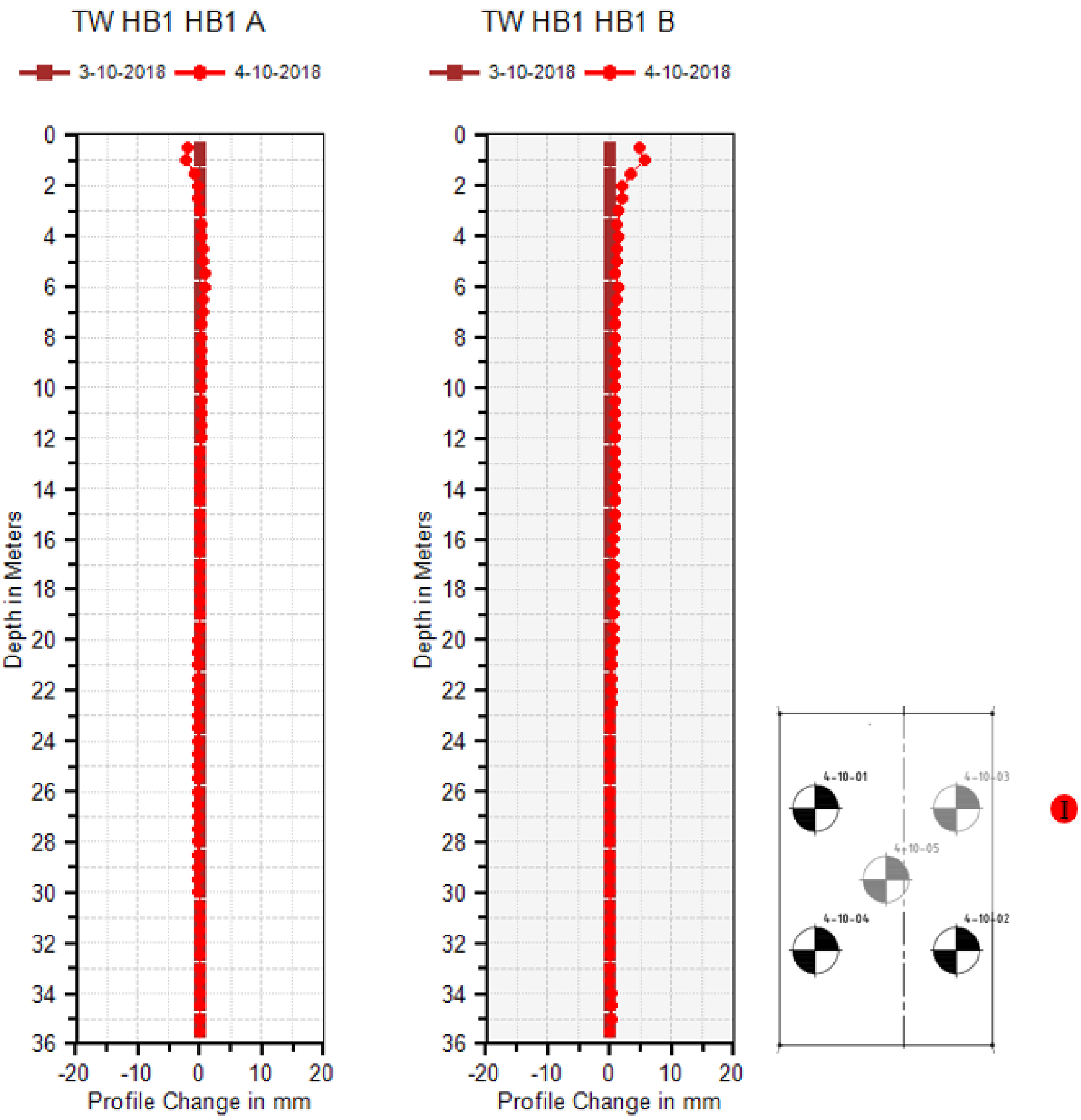


Figure E.3: Profile change after installation pile 3

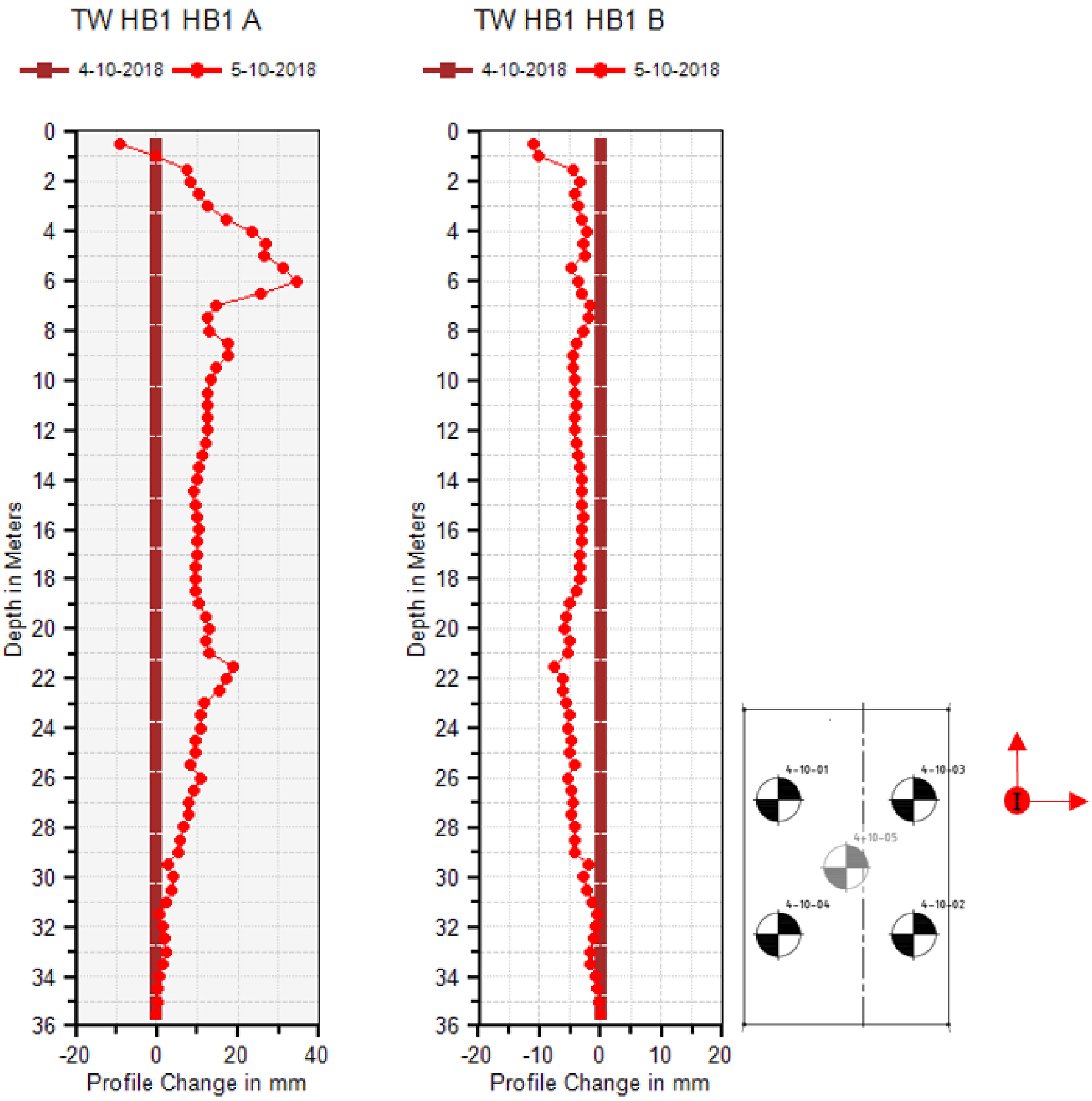


Figure E.4: Profile change after installation pile 4

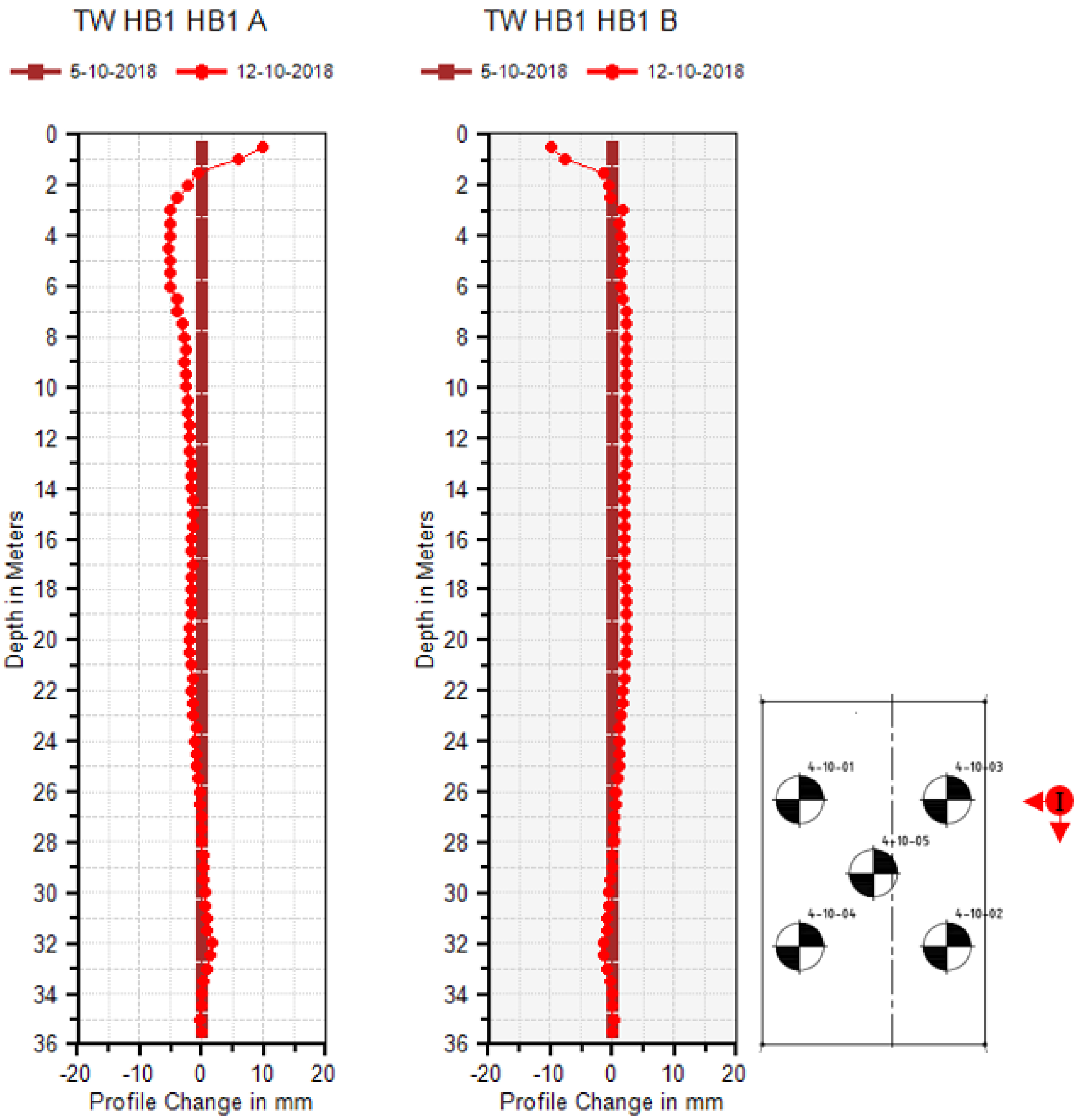
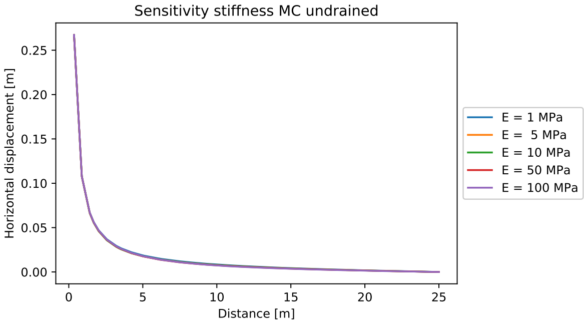


Figure E.5: Profile change after installation pile 5

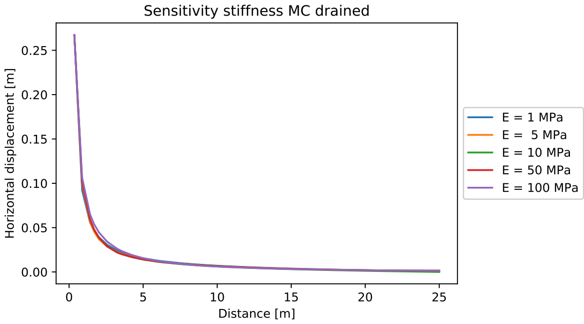
F

Sensitivity analysis

F.1. Mohr-Coulomb

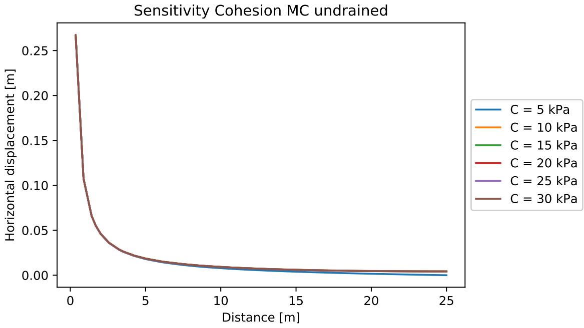


(a) Undrained

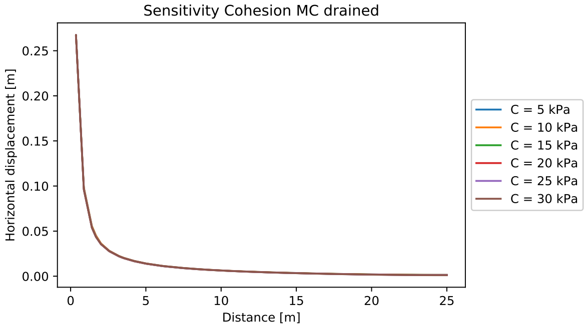


(b) Drained

Figure F.1: Sensitivity stiffness

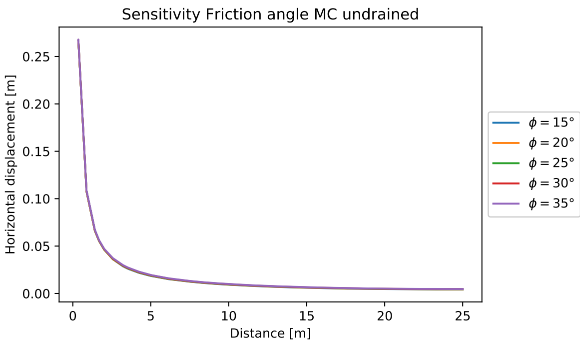


(a) A-direction

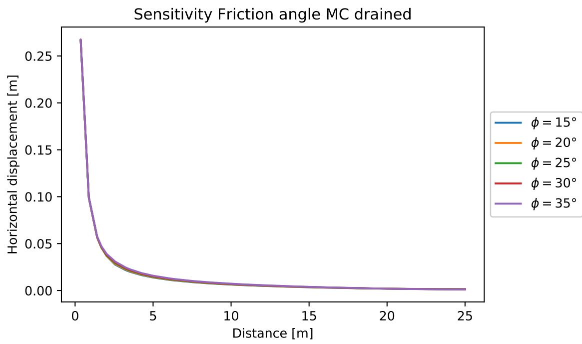


(b) Drained

Figure F.2: Sensitivity Cohesion



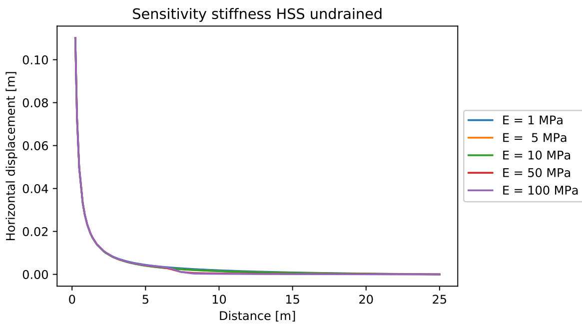
(a) Undrained



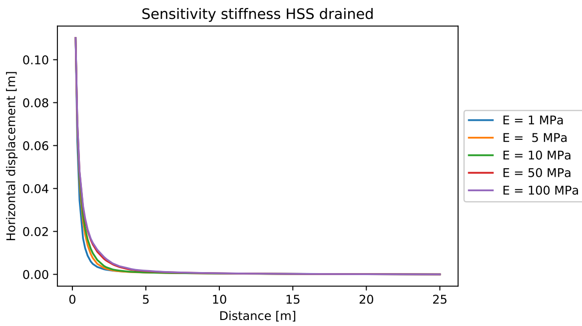
(b) Drained

Figure F.3: Sensitivity friction angle

F.2. Hardening Soil Small Strain



(a) Undrained



(b) Drained

Figure F.4: Sensitivity stiffness