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Radial consolidation of pore pressure induced by pile installation

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Numerical analysis considering material and pore pressure effects from partial remoulding during installation.

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Abstract:

The soil surrounding a driven pile is subjected to varying degrees of soil disturbance during the installation, and experiences a complex combination of loading, unloading and reloading in the course of the life cycle of the pile. No current numerical or analytical method is capable of accurately modeling the entirety of the complex deformations and stress changes occurring in real soil during pile installation and subsequent consolidation.

The report contains a plan for numerical simulation of the installation and consolidation of a driven pile using the Cavity Expansion Method and the Modified Cam Clay material model. The analysis is designed to include the change in material properties and additional excess pore pressure caused by large strains and partial remoulding of the soil following pile installation.

The main objective of this master's thesis has been to enhance the modeling of the consolidation process by:

- Superposing the excess pore pressure caused by soil remoulding with the pore pressure resulting from cavity expansion.
- Applying representative sets of material parameters for radial zones with decreasing degrees of soil disturbance.

No calculation results are included in the report.

An overview of effects contributing to pile setup are given, and the observed trends of overconsolidation ratio and plasticity index on soil behaviour is noted throughout the theoretical part.

Keywords:

1. Pile
2. Pore pressure
3. OCR
4. Setup

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Preface

The following work is a master's thesis in geotechnics. It was written during the spring semester of 2016 for the TBA4900 course of the MSc in Civil and Environmental Engineering at NTNU.

This master's thesis is a continuation of the author's previously completed project thesis TBA4510, carried out during the autumn semester of 2015. The project thesis presented a literature study of subjects relating to consolidation of pore pressure, setup and ultimate capacity of piles driven in clay.

The main objectives of this master's thesis has been to continue the literature study, as well as conducting a numerical analysis in PLAXIS 2D, superimposing the initial excess pore pressures from pile driving assumed in the Cavity Expansion Model (CEM) with excess pore pressures generated by the partial or complete remoulding of the soil surrounding the pile.

The topic was conceived in cooperation with my supervisor Professors Gudmund Eiksund and Gustav Grimstad. I thank them for their ingenuity and for imposing reasonable limits on the work proposed.

Due to personal problems during this semester as well as the loss of some computer files mid-semester, the numerical analysis has not progressed as planned. I have attempted to outline the planned analysis.

Trondheim, 2016-06-10

Anne Mestvedt Olaussen

Summary

Installation of driven piles displaces and remoulds the surrounding soil, generating excess pore pressures. In the consolidation phase, these excess pore pressures are gradually equalized, the remoulded soil is reconstituted and the capacity increases. The main contribution to this time-dependent capacity increase comes from the consolidation of excess pore pressure from installation.

The soil experiences a complex combination of loading, unloading and reloading in the course of installation, consolidation and loading. No single current numerical or analytical method is capable of accurately modeling the entirety of the complex deformations and stress changes occurring in real soil.

The main objective of this master's thesis is to perform a numerical simulation of the installation and consolidation of a driven pile using the Cavity Expansion Method and the Modified Cam Clay material. A general approach for a proposed method of numerical simulation which should enhance the modeling of the consolidation process has is proposed. The analysis is designed to include the change in material properties and additional excess pore pressure caused by large strains and partial remoulding of the soil following pile installation.

Secondary objectives were to discuss soil behaviour during the installation and consolidation phases for a driven pile in clay, and to investigate the influence of overconsolidation ratio and plasticity index on the process of pile installation, consolidation and ultimate capacity. The use of the Cavity Expansion Method (CEM) in determining the excess pore pressure profile after pile installation has been discussed, and the Modified Cam Clay material model has been presented. Soil behaviour in the installation and consolidation phases has been described.

An overview of effects contributing to pile setup are given, and the observed trends of overconsolidation ratio and plasticity index on soil behaviour is noted throughout the theoretical part. Effects of remoulding, straining and reconsolidation on material properties has been discussed, but not quantified. Selection of MCC material parameters for the numerical analysis been briefly mentioned.

Sammendrag

Installasjon av en pel forskyver og omrører leira rundt pelen, slik at det skapes overtrykk i porene. I konsolideringsfasen utjevnes gradvis dette poreovertrykket og den omrørte leira rekonstitueres og pelens kapasitet øker. Hovedbidragene til denne tidsavhengige kapasitetsøkningen kommer av utjevningen av poreovertrykket fra installasjonen.

Jorda rundt pelen utsettes for en kompleks kombinasjon av pålastning, avlastning og repålastning i løpet installasjon, konsolidering og lastpåføring. Ingen nåværende numerisk eller analytisk metode er i stand til å korrekt modellere hele dette komplekse deformasjons- og spenningsforløpet.

Hovedmålet med denne masteroppgaven har vært å utføre en numerisk simulering av installasjon og konsolidering av en pel ved bruk av CEM-metoden og materialmodellen MCC. En generell fremgangsmåte for en foreslått metode for forbedring av numerisk analyse av konsolideringsprosessen er gjennomgått. Analysen er ment å inkludere endringer i materialegenskaper samt tillegget i poreovertrykket som skyldes store tøyninger og delvis omrøring av leira under installasjon.

Sekundære mål var å diskutere leiras oppførsel i løpet av installasjon og konsolidering av en pel, og undersøke innvirkningen av overkonsolideringstallet og plastisitetsindeksen på installasjon, konsolidering og endelig kapasitet av peler. Bruk av CEM-metoden til å bestemme poretrykksprofiler etter installasjon er gjennomgått, og materialmodellen Modified Cam Clay er presentert. Innvirkningene av omrøring, tøyning og rekonsolidering på materialegenskaper er blitt diskutert, men ikke satt tall på. Valg av materialparametre til den numeriske analysen har såvidt blitt omtalt.

Contents

List of Figures

Chapter 1

Introduction

1.1 Background

Installation of driven piles displaces and remoulds the surrounding soil, generating excess pore pressures. In the consolidation phase, these excess pore pressures are gradually equalized, the remoulded soil is reconstituted and the capacity increases. The main contribution to this time-dependent capacity increase comes from the consolidation of excess pore pressure from installation. Further development of numerical models for predicting ultimate pile capacity and required setup times could lead to economic and scheduling savings in pile foundation projects, and a more streamlined design process. A better understanding of the complex changes in stress and material properties occuring during the consolidation process is part of this process, as well as how the consolidation influences the capacity in various soil types.

No current numerical or analytical methods is capable of accurately modeling the entirety of the complex deformations and stress changes occurring in real soil during pile installation and subsequent consolidation. Models such as the cavity expansion method (CEM) or the strain path method are simplified methods of modeling the installation process, yielding stress fields and excess pore pressures which result from the installation. The soil surrounding the pile experiences severe disturbance or complete remoulding, and different zones surrounding the pile experiences a complex combination of loading, unloading and reloading during the life cycle of the pile.

Material models also impose restrictions on the ability to model soil behaviour realistically. Any numerical model of the process must by its very nature be a simplified version of real soil behaviour, The aim of the numerical analysis is to attempt to enhance the modeling of the consolidation process by applying representative sets of material parameters for each of a number of soil disturbance ranges. The author proposes that this will better represent the change in material properties and the additional excess pore pressure caused by the large strains present in the soil surrounding the pile after installation.

1.1.1 Problem Formulation

Pile load tests show that ultimate shaft friction is strongly dependent on the overconsolidation ratio OCR and the plasticity index I_p of the clay. The reason for the strong influence of OCR and I_p is not fully understood, and a better understanding of these relationships could lead to improvements in material models and calculation methods. The difference in unloading and reloading stiffness during consolidation of the excess pore pressures induced by pile driving is proposed as one explanation of the observed influence on the shaft friction.

1.1.2 Literature Survey

The last century has seen great advances in the field of deep foundation geotechnics, including increasingly sophisticated pile testing methods, increased understanding of soil behaviour and the development of numerical analysis tools and advanced material models.

Two analytical models for the installation process have been proposed - the Cavity Expansion method in 1950, and the Strain Path Method in 1985. These methods have been applied to different material models and problems, and a limited number of close-form solutions exist alongside numerous numerical implementations. The CEM method was proposed and expanded on throughout the late 1960's and 70's by amongst others Randolph & Wroth (1979) and Carter et al. (1979). Solutions for the simple elasto-plastic (EP) material model and for the more advanced Modified Cam clay (MCC) model were quickly proposed, and have been the subject of much research in recent years as well, amongst others by

Recent semi-empirical methods for predicting the ultimate shaft capacity by Karlsrud (2012), include the influence of OCR and I_p. These have been implemented in Norwegian pile design guidelines, but empirically based models have only a limited ability to model all the complex changes in soil behaviour during installation and consolidation. The influence of OCR and I_p

A master's thesis by Bergset (2015) titled "Radial consolidation of pore pressure induced by pile driving" contains a parametric study for OCR and I_p and numerical modeling of the consolidation of four soil with high and low values of OCR and I_p.

1.1.3 What Remains to be Done?

- Currently no single numerical, analytical or empirical method is able to predict ultimate capacity of piles accurately for all possible of fine-grained soil conditions. The physical processes determining the ultimate capacity and controlling the stress-strain changes during installation, consolidation and loading of piles in clay are not fully understood.
- Existing constitutive models can be developed further to enable them to accurately predict the complex behaviour of soil during installation and consolidation. Numerical modeling of the radial consolidation process for clays with a range of OCR and I_p values and comparison with observations from relevant load tests and instrumented piles can be undertaken to establish a greater understanding of the limitations and strength of current models, and to establish standard state-of-the-art procedures for numerical capacity design of piles.

1.2 Objectives

The main objectives of this project are

1. Conduct a numerical analysis of the dissipation of excess pore pressures generated during the installation of a displacement pile. The aim of the numerical analysis is to attempt to enhance the modeling of the consolidation process by applying representative sets of material parameters for each of a number of soil disturbance ranges.
2. Choose a material model and methods for modeling the installation and consolidation phases, and briefly present a theoretical background for these.
3. Investigate the influence of overconsolidation ratio and plasticity index on the process of pile installation, consolidation and ultimate capacity.

1.3 Limitations

Only driven, closed-ended displacement piles in clay are considered. Theory and results concerning piles in sand are considered only in so far as the same principles are applicable to clay piles.

1.4 Approach

- A continuation of the literature study started in the author's project thesis in the autumn of 2015, with emphasis on the CEM method, MCC material model and the consolidation process.
- Investigation of the influence of the installation and consolidation processes on the material parameters and
- Establishing a numerical model in PLAXIS and modeling the consolidation of a pile section and its surrounding soil. The input for the consolidation analysis shall be a superposition of the pore pressure due to volume expansion from inserting the pile, and the pore pressure profile due to different degrees of remoulding of the surrounding clay.

1.5 Structure of the Report

This report is divided into two parts. Chapters 1 through 6 presents the theoretical background for understanding the subject of setup and consolidation of piles. Chapter 7 deals with the influence of overconsolidation ratio and plasticity index on the processes described in previous chapters. Chapter 8 summarizes findings and recommends further work to be undertaken. A brief description of each chapter is given below.

- **Chapter 2** gives a general overview of pile foundations, the setup effect, ultimate pile capacity and the influence of OCR and I_p.
- **Chapter 3** gives an overview of soil behaviour in the installation phase, and presents the Cavity Expansion Method.
- **Chapter 4** describes soil behaviour in the consolidation phase and the process of excess pore pressure dissipation.
- **Chapter 5** presents the Modified Cam Clay (MCC) model.
- **Chapter 6** describes the setup and calculation stages of the numerical analysis.

- **Chapter** 7 summarizes significant findings, and gives an overview of some of the work that remains to be done in this field.

Chapter 2

Pile capacity and setup

Pile foundations are used when a shallow foundation does not have sufficient capacity, excavations are difficult or costly, or if the expected settlements are too large. Piles may transfer the load to a deeper, high-capacity layer using end-bearing or utilize the friction of a large exposing a large shaft area to the surrounding soil. Piles in soft clay are commonly mainly friction piles.

The life-cycle of a pile can be divided into an installation phase, a consolidation phase and a loading phase as shown in Figure 2.1. Randolph (2003) states that any scientific method for predicting the ultimate shaft friction must consider the complex stress and strain changes during all three phases. As opposed to for shallow foundations, installation of a pile produces change the stresses and void ratios of the soil surrounding the pile shaft soil significantly, creating large displacements, soil disturbance and large excess pore pressures. As the excess pore pressures equalize, the soil gradually regains and increases its capacity during consolidation to beyond the strength of the undisturbed soil. Even after the load is applied, the pile can continue to gain capacity due to ageing effects Karlsrud (2012). The soil response in the first two phases is described in more detail in Chapters 3 and 4.

2.1 Pile Types and Installation Methods

Piles can be classified according to their material, geometry or installation technique. Of these, the installation technique is the most influential.

Replacement piles are bored or drilled. Because there is little displacement of soil during installation, replacement piles generate no large pore pressures. This thesis is concerned with

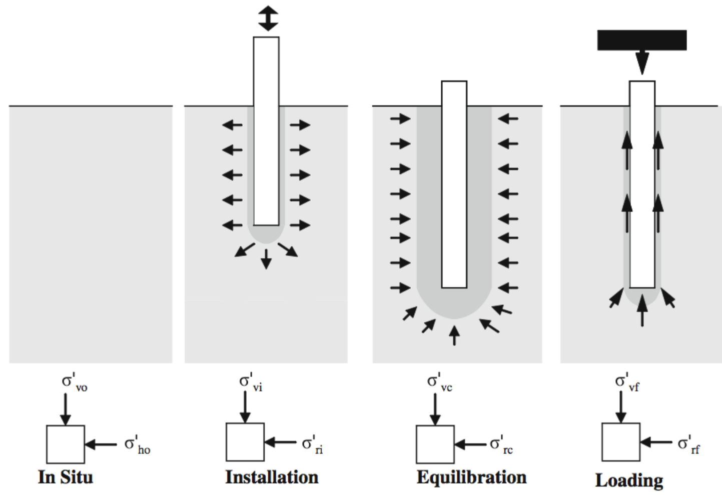

Figure 2.1: Phases: installation, consolidation and loading Doherty & Gavin (2011)

the dissipation of the excess pore pressure generated after installation, so replacement piles are not relevant.

Displacement piles are most commonly installed by driving, but could also be installed by jacking, vibration or screwing. Both side friction and end-bearing contributions to the pile capacity are influenced by the disturbance of the surrounding soil. Friction piles carry the applied load by a combination of tip resistance and shaft friction. The tip-resistance of friction piles is negligible compared to the shaft friction for slender piles and in soft clays (NGF 2012). Therefore, only the mechanisms contributing to ultimate shaft friction will be considered in this thesis.

Eurocode 7: Geotechnical design - Part 1: General rules (2008) sets down principles and guidelines for the design of piles. The capacity of a pile is verified by ensuring that the pile has sufficient resistance for a given load combination. The characteristic bearing capacity of a pile can either be measured or calculated, using test piles or pile load tests, capacity formulas or dynamic testing during installation. Test piles may be non-instrumented or have built-in instrumentation to measure strains, displacements, pore pressure or accelerations. Static load tests provide the most reliable performance data but are relatively time consuming and expensive, while dynamic load tests are quicker. The development of accurate predicitive models are essential for economic, safe and efficient foundation design, especially in offshore foundation design because maintained load tests to confirm capacity can be prohibitively expensive in offshore applications.

2.2 Setup and ultimate capacity

Pile setup refers to the gain in pile resistance over time and it has been observed by many researchers. Both sands and clays exhibit time-dependent behaviour affecting their stress-strain response and shear strength, but the underlying causes of the soil behaviour is different for fine- and course-grained soils. In the following, only clay is considered. The most significant process for clays is capacity gain during the consolidation of excess pore pressures generated by installation and reconstitution of the surrounding remoulded soil, but other phenomena also impact the capacity and deformations of the soil surrounding the pile. Among these are creep, ageing, thixotropy, stress relaxation and viscous effects (Alves et al. 2009). Dissipation of pore pressure and the reconstitution of remoulded soil are described in more detail in Chapter 4.

2.2.1 Secondary effects contributing to setup

Considerable capacity increase is observed even after the excess pore pressure has equalized. Ageing refers to the increase in pile after the end of reconsolidation. Doherty & Gavin (2013) reports that the fully equalized shaft resistance increased by at least 40 % over a 10 year period in a Belfast soft clay, and Karlsrud (2012) reports a similar increase for pile tests at Haga. Plasticity index has a major impact on ageing, such that capacity increase with time is much larger in NC or low-plastic clays than in high OCR or high-plasticity clays. The ageing capacity increase may be due to a geochemical effect affecting bonding between clay particles and/or a further increase in total and effective stresses caused by creep effects. Creep, or secondary compression, is the volume change of a soil caused by the adjustment of the soil particles. According to Bergset (2015), creep increases with increased I_p.

During pile installation, soil adjacent to the pile will be disturbed and remolded, and its strength is reduced. After pile installation is completed, the reduced strength in the surrounding soil will start to regain its strength with time (Abu-Farsakh et al. 2015). This strength increase under constant volume and effective stress is known as thixotropy, a pure geochemical effect influenced by clay structure and mineralogy, water content and concentration of dissolved pore water ions. In a fully thixotropic material, remoulded soil that has less strength than the original soil will recover its full strength with time regardless of whether the soil is subjected to consolidation or not (Abu-Farsakh et al. 2015).

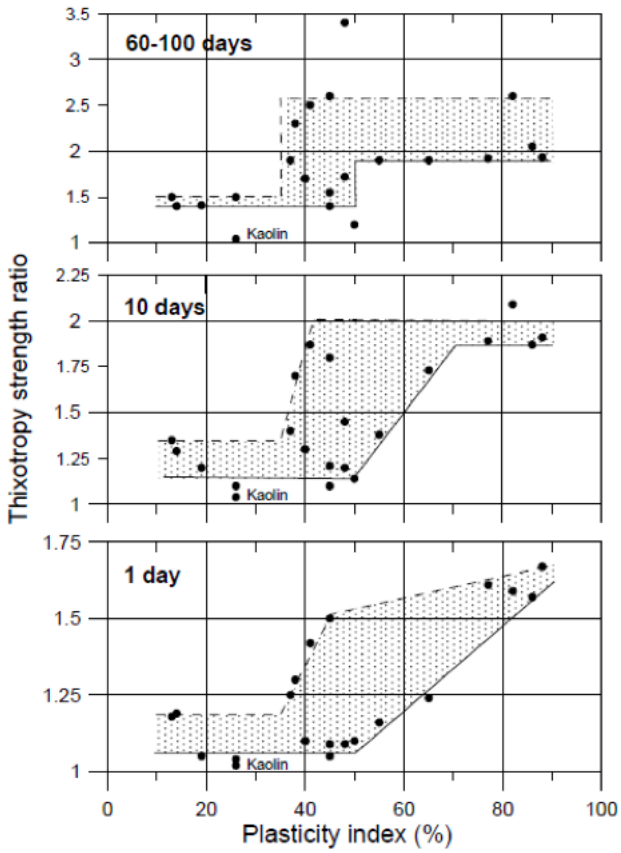

Figure 2.2: Thixotropy strength ratio C_t as function of time and plasticity index (Andersen and Jostad, 2002)

Andresen and Jostad (2002) report an increase of 1,5-2,5 times the remoulded strength after approximately 100 days, and that most of this increase occured within the first days after remoulding. Thoxitropy may occur both before and during the dissipation of pore pressures, but it is difficult to distinguish from capacity gain caused by reconsolidation (Karlsrud 2012). Since the interaction is unknown, only thixotropy effects occuring after reconsolidation is complete should be included. The thixotropy strength ratio C_t relates shear strength at a certain time after pile installation with strength gain due to thixotropy and the remoulded shear strength.

2.3 Ultimate shaft friction

Determining the ultimate capacity of pile foundation is a complex matter. The soil is subjected to large deformations, and complex stress-strain changes occur during installation and consolidation.

The early methods for determining ultimate pile capacity used empirical correlations with pile test data to relate the shaft friction to undrained shear strength (α-methods), to in-situ vertical effective stress (β-methods) or to both (γ-methods). Most of these have showed limited prediction accuracy in other soil conditions than those used in their calibration, because they are based on results from a limited number pile types or soil conditions. In addition, such simplified methods could not model the complex stress-strain changes necessary to describe pile installation and consolidation (Doherty & Gavin 2013). Because of their simplicity, total stress design approaches remain popular, but factors such as stress history or length effects governing the empirical parameters are often contradictory (Doherty & Gavin 2011).

Numerical models used in combination with results from instrumented piles and load tests have over the past 30 years lead to the development of more theoretically well-founded analytical methods that attempt to account for pile installation effects on the stress field and stress-strain and strength characteristics of the clay surrounding a pile. CEM and SPM methods show promise, but require further developments to correctly model all aspects of pile behaviour (Randolph 2003). Karlsrud (2012) reports that present design methods in common use are still of a semi-empirical nature.

2.3.1 Empirical and Semi-Empirical Methods

Based on his collection and analysis of a large number of instrumented pile tests in various soil conditions, Karlsrud (2012) proposes two new semi-empirical methods for calculating ultimate shaft capacity, an α-method and a β-method. These are incorporated into the 2012 Norwegian Pile Guidelines (NGF 2012). Karlsrud (2012) found no clear evidence for the influence of pile diameter, length or stiffness on the local ultimate shaft friction, but these length effects are included in several other proposed models (Doherty & Gavin 2013).

Karlsrud (2012) argues that the shearing along the pile shaft during axial pile loading closely resembles the Direct Simple Shear (DSS) mode of failure, and has therefore chosen to use the direct shear strength s_{ud} as the reference strength in the two methods.

α-Method

The ultimate shaft friction is given by

$$\tau_{us} = \alpha \cdot s_{ud} \tag{2.1}$$

The value of the α-factor determined by normalized undrained shear strength s_{ud}/σ'_{v0} and

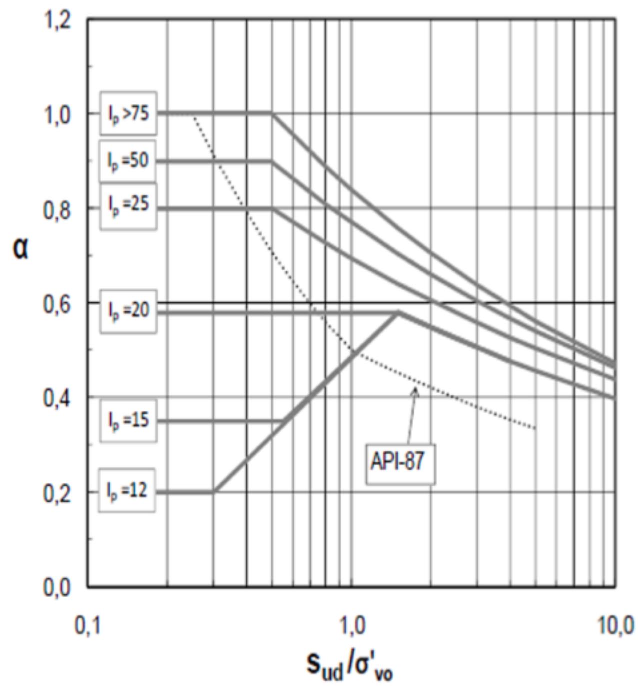

Figure 2.3: Determining α-factor for Karlsrud's α method (Karlsrud 2012)

plasticity index PI, from Figure 2.3. PI is shown to have a significant effect on the shaft friction. As expected, low-plastic clays have significantly lower shaft friction than high-plastic clays.

β-Method

The ultimate shaft friction in the β-method is a product of vertical in-situ effective stress and a β-factor determined from the overconsolidation ratio and the plasticity index, Figure 2.4.

The ultimate shaft friction is given by

$$\tau_{us} = \beta \cdot \sigma'_{v0} \tag{2.2}$$

The predicted ultimate shaft friction increases for increasing overconsolidation, and for increasing values of the plasticity index.

2.4 Influence of OCR and PI

Pile load tests show that ultimate shaft friction is strongly dependent on the overconsolidation ratio OCR and the plasticity index I_p of the clay. The reason for the strong influence of OCR

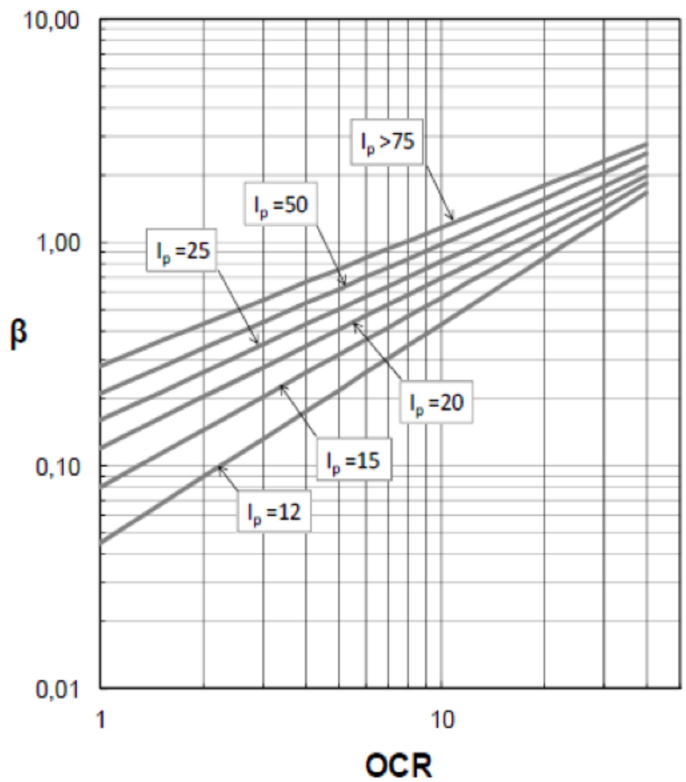

Figure 2.4: Determining β-factor for Karlsrud's β method (Karlsrud 2012)

and PI is complex, and is not fully understood. Possible explanations include differences in unloading and reloading stiffnesses.

2.4.1 Observed effects

- Bergset (2015) describes that the plasticized radius increases with increasing OCR, and decreases with increasing I_p.
- Greater excess pore pressure is generated for a larger plastic zone.
- Shorter consolidation times are predicted for high OCR clays, because of pore pressure field does not extend as far from the pile. Shorter consolidation times are also found for lower plasticity clays because of higher permeability assumed in the model (Bergset 2015). Field observations confirm rapid development of setup for OC clays and slower for NC clays (Karlsrud 2012).
- Negative excess pore pressures are developed during shearing for clays with OCR > 2, because high OCR clays tend to dilate on shearing. The excess pore pressure at the pile shaft

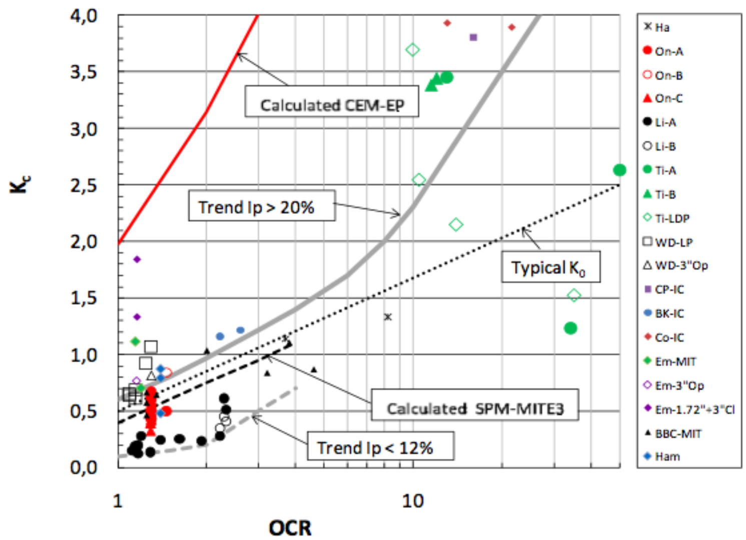

Figure 2.5: Radial effective stresses (Karlsrud 2012)

is somewhat underpredicted for the cases with OCR of 1, and overpredicted for the cases with OCR of 8 (Bergset 2015).

- Figure 2.5 shows results from instrumented piles, CEM elasto-plastic model prediction and SPM-MITE3 model prediction as assembled by Karlsrud (2012). It is clear that piles installed in low-plasticity clays develop very low horizontal effective stresses, while the effective stresses increase with increasing values of OCR for high plasticity clays.

Karlsrud (2012) has observed circumferential arching and vertical silo effects in low plasticity deposits. A large ratio between the compression index in the inner zone and the swelling index in the surrounding clay, may enable arching and silo effects during the reconsolidation phase. The difference in unloading and reloading stiffness could also explain these effects, which in part account for the very low horizontal effective stresses observed in low-plasticity clays.

The remoulded zone around the pile shaft after installation has low relative density and low radial effective stresses. The arching effect is present in a transition zone of high relative density between the remoulded zone and the undisturbed soil, Figure2.6.

2.4.2 Unloading and Reloading

Soil is displaced and compressed outwards from the pile during installation. During the subsequent reconsolidation process, soil particles will move inwards towards the pile and most of

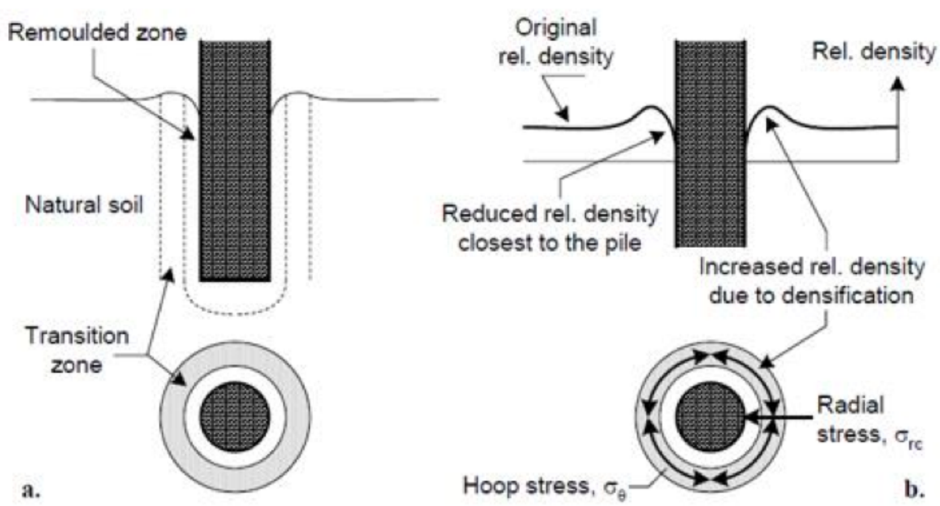

Figure 2.6: a. Zones created during pile installation; b. Relative density in the soil and arching effects around the pile shaft, after Bergset (2015)

the soil will be unloading in shear. However, the inner zone close to the pile experiences further increase in shear strain because the pile prevents the inward movement of the soil. The inner zone experiences primary loading during consolidation, while the outer zone experiences unloading/reloading type stress changes (Bergset 2015).

The inner primary loading zone consists of severely remoulded clay close to the pile, and a zone of disturbed clay with large but measurable shear strains. The volumetric compressibility of the primary loading zone is different from that of the undisturbed zone further out experiencing unloading (Karlsrud 2012). Radial effective stresses at the pile surface decrease with increasing difference in stiffness or volumetric compressibility between the two zones (Bergset 2015). The extent of the inner zone decreases with increasing I_p and OCR.

Figure 2.7 shows the non-linearity and stress dependence of unloading and reloading stiffnesses. The following observations can be made from the figure (Karlsrud 2012):

- The tangent of the unloading modulus is reduced with decreasing effective stress.
- The tangent of the reloading modulus depends on the stress level at which unloading stopped.
- The reloading modulus is initially stiffer than the unloading modulus.
- The reloading modulus is constant until 25 % below the preconsolidation pressure, and then decreases linearly toward the NC modulus line.

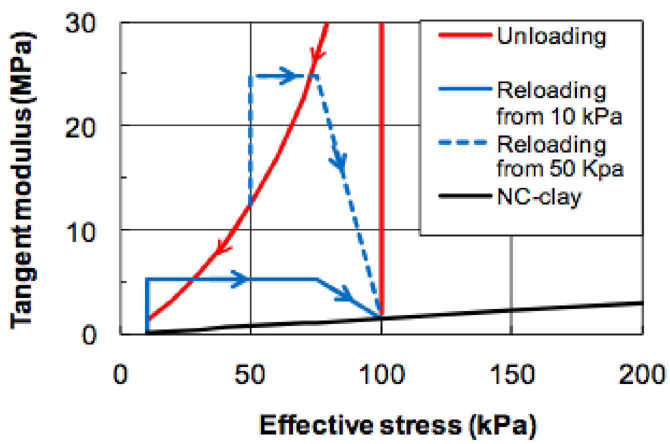

Figure 2.7: Unloading and reloading stiffness (Karlsrud 2012)

Chapter 3

Installation Phase

The effects of the installation of a displacement pile include:

- Excess pore pressures
- Soil disturbanc and reduced soil strength

The effects of pile installation are discussed below, and then the CEM approach for modeling the results of stress and strain changes during the installation phase is described.

3.1 Soil disturbance

During installation the pile is driven into the ground, and surrounding soil is pushed outwards by the pile. A plastic zone is developed around the pile where the mobilized shear stress exceeds the undrained shear strength of the soil. Fu & Xiaoyu (2011) states that the soil within a radial distance of 2 pile diameters is completely remodeled. The surrounding soil is strained in a varying degree, but beyond a radial distance of 10 diameters soil deformation is negligible (Abu-Farsakh et al. 2015). The exact radial distance range of each region is disputed, but an often used approximation of the limits of regions is that A equals 0.125D 0.2D, region B represents 3D 10D, and region C is assumed to be 10D away from the pile shaft (Fu & Xiaoyu 2011).

Figure 3.1 shows the soil disturbance in terms of the distance from the pile wall. Zone A nearest the pile is completely remoulded, and Zone B is disturbed in some degree. Zone C is undisturbed by the pile installation.

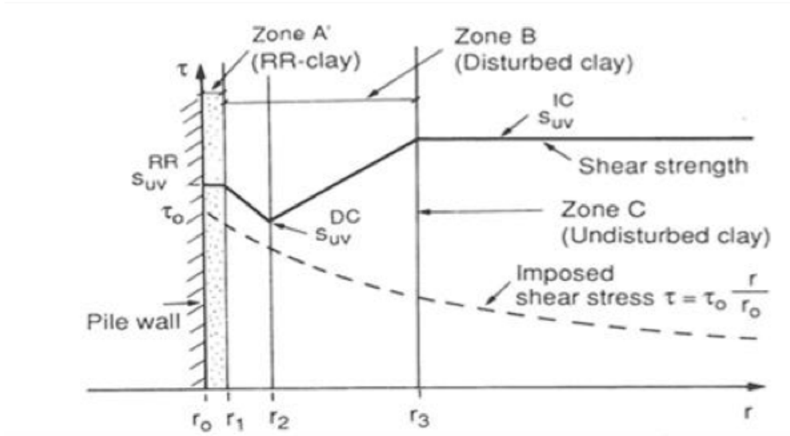

Figure 3.1: Zones of impact of pile installation, Karlsrud (2012)

3.2 Excess pore pressure build-up

The excess pore pressure built up during installation is primarily due to increase in total stress as the soil is displaced outwards by the pile. In addition, changes in mean effective stress due to shearing and partial remoulding of the surrounding soil contribute to the excess pore pressure. Shearing will produce positive excess pore pressures for lightly overconsolidated clays, and negative pore pressures for dilatant, heavily overconsolidated clays (Randolph 2003).

The generated pore pressures close to the pile are often larger than the effective overburden pressure. Along with the remoulding of the zone near the pile this facilitates the installation of the pile to the required depth Randolph & Wroth (1979). The magnitude of the pore pressures induced by driving decrease rapidly with distance from the pile wall, and becomes negligible at a distance of 5-10 pile diameters Dong Guo (2000).

3.3 Cavity Expansion Method (CEM)

Using a radial consolidation theory, Randolph & Wroth (1979) show that the rate of increase of pile capacity in soft clay is related to the rate of pore pressure dissipation. Therefore, one can predict the change in capacity by modeling the change in pore pressures (Dong Guo 2000). The initial stress and excess pore pressure distributions may be simulated using the Cavity Expansion Method (CEM) or Strain Path Method (SPM). Karlsrud (2012) states that the SPM model tends to yield lower effective stresses and pore pressures at the pile shaft than the CEM model. The CEM and SPM methods also give notably different stresses and excess pore pressures as

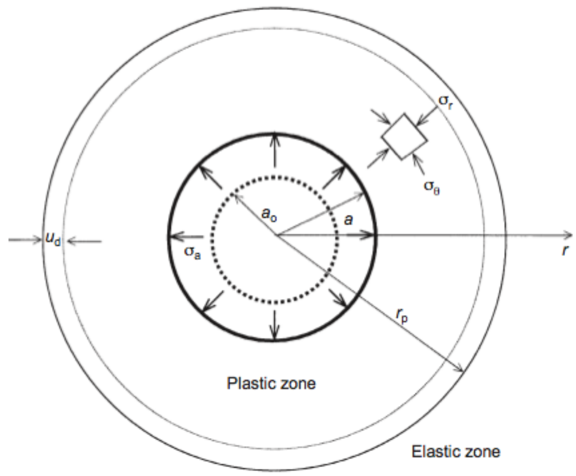

Figure 3.2: Plastic and elastic zones surrounding the pile, Cao et al. (2001)

function of radial distance from the pile wall.

3.3.1 Modeling the installation phase

The cavity expansion method was presented by Carter et al. (1979). The method is used to predict the response of the surrounding soil when a pre-existing cavity is enlarged, and can also be extended for use in modeling the creation of a cavity, as for a displacement pile. It can be used to model the disturbance of soil due to pile driving as well as modeling the distribution of excess pore pressure.

The CEM method is a one-dimensional approach, and therefore ignores vertical deformations, shearing around the pile tip, and the influence of the ground surface. It cannot properly model the complex strain histories of elements close to the shaft of displacement piles. It therefore provides poor estimates of shaft stress, but provides reasonable predictions of radial displacement (Doherty & Gavin 2011). Deformation of the soil is upward when the pile penetration depth is lower than 10D, but when the pile penetration depth is greater than 10D, the deformation of the soil surrounding the pile is in the radial direction (Fu & Xiaoyu 2011). In this thesis, surface heave, stress distributions around the pile tip, and the ultimate shaft friction capacity is of lesser importance than the radial pore pressure changes and radial displacements. The CEM method modeling the pile installation as the expansion of a cylindrical cavity is appropriate, and the CEM method is easily incorporated into the PLAXIS 2D staged construction workflow.

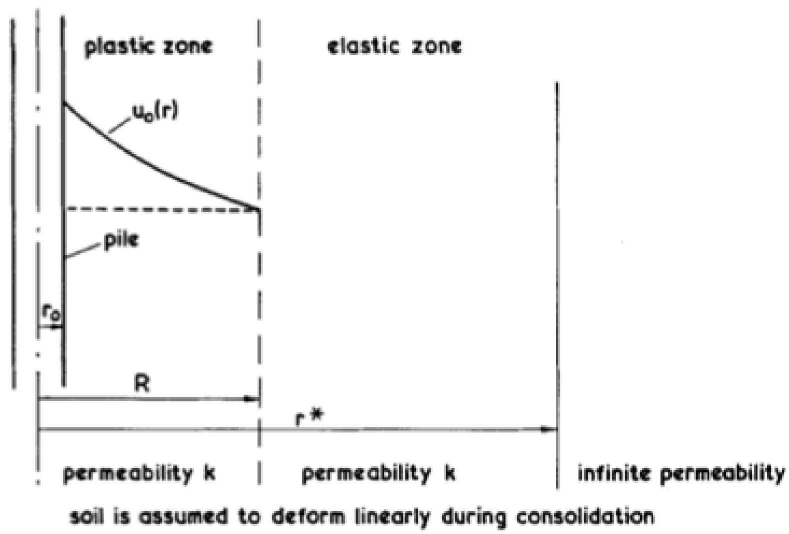

Figure 3.3: Plastic and elastic zones, and initial pore pressure distribution (Randolph & Wroth 1979)

In the CEM method, pile installation is simulated by expanding a cylindrical cavity in a soil mass (Carter et al. 1979). During undrained installation, the radial displacement δr of a soil particle at a distance r from the centre of a closed-ended pile with a radius R (Figure 3.2) is given by

$$\frac{\delta r}{R} = \sqrt{1 + (\frac{r}{R})^2} - \frac{r}{R} \tag{3.1}$$

Figure 3.3 shows the elastic and plastic zones, as well as the initial pore pressure distribution for the EP soil.

3.3.2 Elasto-plastic solution

Randolph & Wroth (1979) determined the initial excess pore pressure distribution caused by the expansion of a cylindrical cavity in an ideal elastic, perfectly plastic material Icharacterized by shear modulus G and shear strength s_u, briefly shown below.

The equations of radial and vertical equilibrium for an one-dimensional undrained cavity expansion are

$$\frac{\delta \sigma_r}{\delta r} + \frac{\sigma_r - \sigma_\theta}{r} = 0 \tag{3.2}$$

$$\frac{\delta \sigma_z}{\delta z} = \gamma \tag{3.3}$$

The radial and tangential stress changes inside the plastic zone for a cavity expanded from zero radius to radius r_0 are

$$\Delta \sigma_r = s_u (1 + ln \frac{G}{s_u} - 2 ln \frac{r}{r_0}) \tag{3.4}$$

$$\Delta \sigma_\theta = s_u (-1 + ln \frac{G}{s_u} - 2 ln \frac{r}{r_0}) \tag{3.5}$$

The width of the plastic zone around the wall is given by

$$r_p = r_0 \sqrt{\frac{G}{s_u}} \tag{3.6}$$

Mean effective stress is assumed to remain constant under undrained conditions, and thus the excess pore pressure will equal the change in mean total stress. This gives the following distribution of excess pore pressure immediately after driving inside and outside the plastic zone, respectively.

$$\Delta u_0 = 2 s_u \ln \frac{r_p}{r} \tag{3.7}$$

$$\Delta u_0 = 0 \tag{3.8}$$

This assumption of an elasto-plastic soil model may yield reasonable predictions for the excess pore pressures provided that suitable values for the secant shear modulus are chosen Randolph & Wroth (1979). It is observed that the initial excess pore pressure at the pile shaft is proportional to the undrained shear strength c_u, and also depends on the ratio G/c_u.

Karlsrud 2012 has calculated maximum excess pore pressures based on the CEM-EP approach for different values of the overconsolidation ratio, Figure 3.4. It is apparent that the value of the pore pressure at the pile shaft decreases for increasing values of OCR.

3.3.3 Modified Cam Clay Solution

Carter et al. (1979) utilised both an elasto-plastic model and a work-hardening MCC model, and

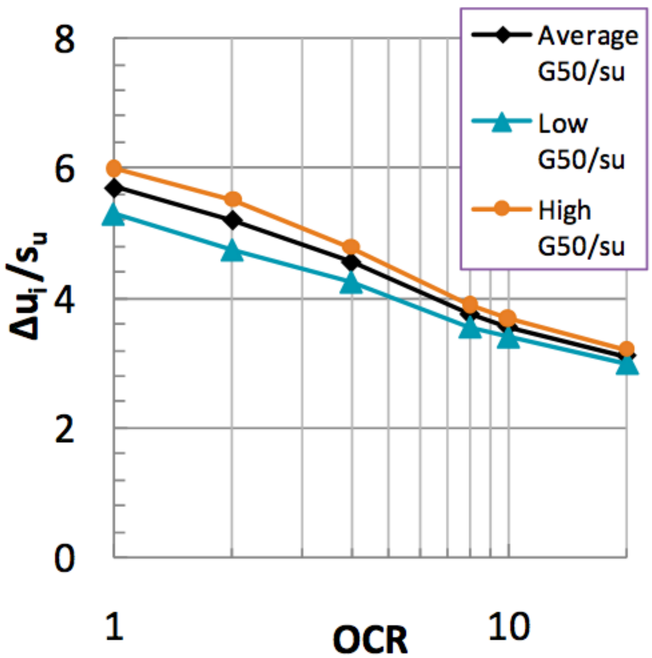

Figure 3.4: Normalized maximum excess pore pressure based on CEM-EP model (Karlsrud 2012)

concluded that the dissipation of pore pressures with time is relatively unaffected by the choice of soil model. A good estimate can be obtained by assuming a linear elastic soil with a sensible choice of parameters. However, the predicted stress changes are much more dependent on the type of soil model than the pore pressures.

Using an elasto-plastic soil model for the cavity expansion problem gives a closed-form solution, however it has two serious shortcomings (Randolph et al. 1979). Firstly, that the additional pore pressure generated by pure shear stresses is not included. Shear induced excess pore pressures are expected to be positive in low and moderate OC clays, while in highly overconsolidated clays shearing produces negative pore pressures because of dilation. Secondly, that the elasto-plastic soil model does not correctly relate the strength of the soil to the current effective stress state and stress history of the soil.

Using the Modified Cam clay (MCC) soil avoids these particular shortcomings (Randolph et al. 1979). Closed-form solutions are difficult to establish for advanced soil models, but closed form solutions have been established by making assumptions about the shear stress variation around the cavity. An alternative to analytical or semi-analytical solutions is using the MCC model in a finite element analysis to determine the resulting stress and pore pressure.

Cao et al. (2001) and Abousleiman & Chen (2012) have applied the MCC model to the expansion of a cylindrical cavity, using a small strain analysis in the elastic zone and a large strain analysis in the plastic zone.

The approximate solution presented by Cao et al. (2001) is

$$\frac{\Delta u_a}{p_0^{'}} = (\frac{R}{2})^{\Lambda} ln[\frac{G\sqrt{m+2}}{Mp_0{'}(R/2)^{\Lambda}}] + p_0{'}[1 - \frac{R}{2})^{\Lambda}] \tag{3.9}$$

where Λ is the volumetric strain ratio Λ = (1 - κ / λ) = 0,75 for triaxial compression. R is the isotropic overconsolidation ratio, u_a is the pore pressure at the cavity wall, M=1 for a cylindrical cavity, G is the shear modulus and $p_0^{'}$ is the initial mean effective stress.

According to Cao et al. (2001), the simplified solution provides good estimates of the radial stress and the circumferential stress around the cavity and the pore pressure near the cavity wall. The normalized excess pore pressure at the cavity wall is shown in Figure 3.5 for different isotropic overconsolidation ratios R.

Using this solution, Cao et al. (2001) observes that the size of the plastic zone in a stiff soil is much larger than that in a soft soil. For the lightly to moderately overconsolidated soil (R<5) the size of the plastic zone decreases rapidly as R increases. An increase in overconsolidation ratio results in increased Δu at the cavity wall, decrease in Δu at the plastic boundary and reduction in the plastic radius r_p. This increases the pore pressure gradient, and therefore leads to a quicker dissipation rate.

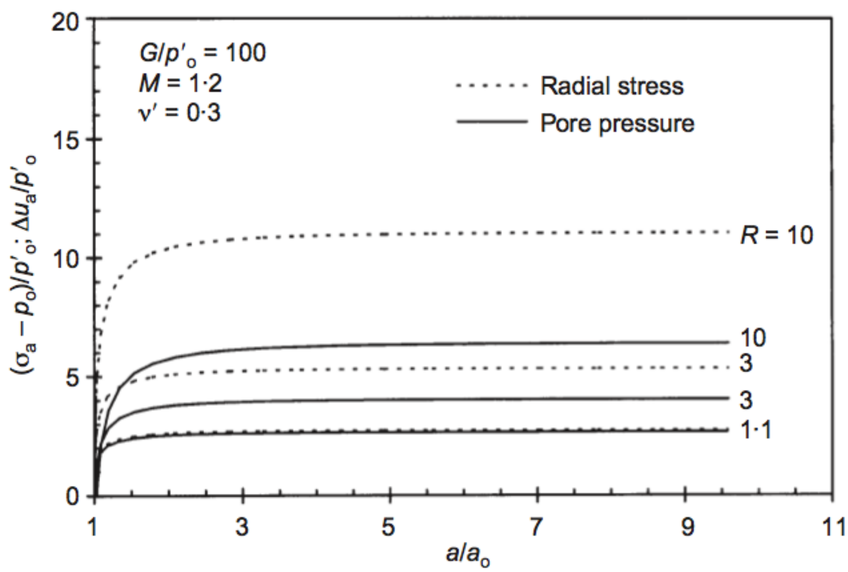

Figure 3.5: Normalized excess pore pressure at the cavity wall and normalized cavity pressure with cavity (Karlsrud 2012)

Chapter 4

Consolidation Phase

Chapter 3 describes how excess pore pressures are built up during the rapid installation of a driven pile. In the consolidation phase, the disturbed soil will reconsolidate as the excess pore pressure dissipates. The pore pressure gradient drives a radial flow from the pile towards the far field (Ottolini et al. 2015), and the soil contracts. As the pore pressure dissipates, the mean effective stress acting on the pile increase, leading to an increase in bearing capacity (Dong Guo 2000).

Fine-grained soils often have very low permeability. The consolidation process may last for a very long time, but a significant increase in capacity may be observed only a short time after installation (Randolph 2003).

4.1 Consolidation Theory

According to (Karlsrud 2012), the radial consolidation process following pile installation depends on three main factors:

1. The radial extent of the excess pore pressure field
2. The shape of the field
3. The coefficient of radial consolidation, c_h, and its variation with distance from the pile wall and with effective stress changes during the consolidation process.

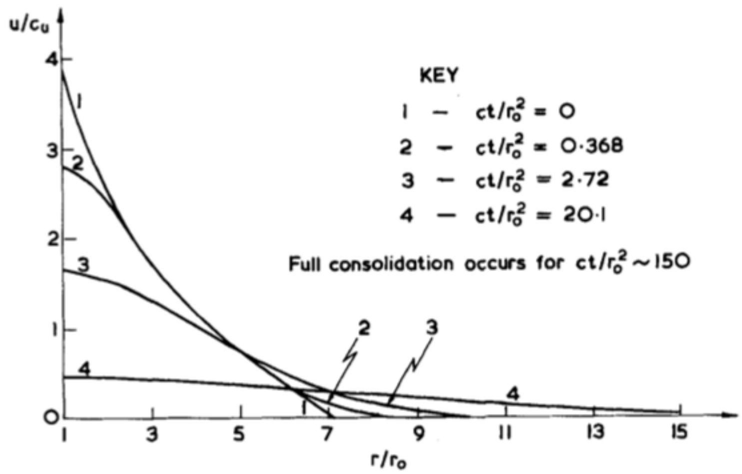

Figure 4.1: Variation of radial distribution of excess pore pressure with non-dimensional time for G/cu=50 clay (Randolph & Wroth 1979)

4.1.1 Elastic solution for logarithmic variation of u_0

According to Randolph et al. (1979), field measurements show that the major pore pressure gradients during consolidation are radial. It is therefore reasonable to assume that consolidation takes place primarily by flow radially outwards from the pile. During consolidation, soil particles that were displaced during the installation of the pile move radially inwards towards the pile, and therefore undergo an unloading in shear. The soil skeleton is assumed to deform elastically during the consolidation phase, and under conditions of plane strain and axial symmetry.

Assuming elastic soil, the governing equation presented by Randolph & Wroth (1979) for the radial consolidation problem is identical to Terzaghi's one-dimensional consolidation equation:

$$\frac{\delta u}{\delta t} = c_h\left(\frac{\delta^2 u}{\delta r^2} + \frac{1}{r}\frac{\delta u}{\delta r}\right) \tag{4.1}$$

where c_h is the radial coefficient of consolidation which is constant for elastic soils. c_h is a function of permeability k, coefficient of volume compressibility m_v, elastic shear modulus G and Poisson's ratio ν.

Randolph & Wroth (1979) report that the model gives realistic values for the decay of pore pressure near the pile with time after driving, but that a more advanced soil model should be used in order to predict stress changes accurately. The solution is applicable to any initial radial distribution of pore pressure.

Figure 4.1 shows the radial pore pressure distribution for a soft clay at four moments during

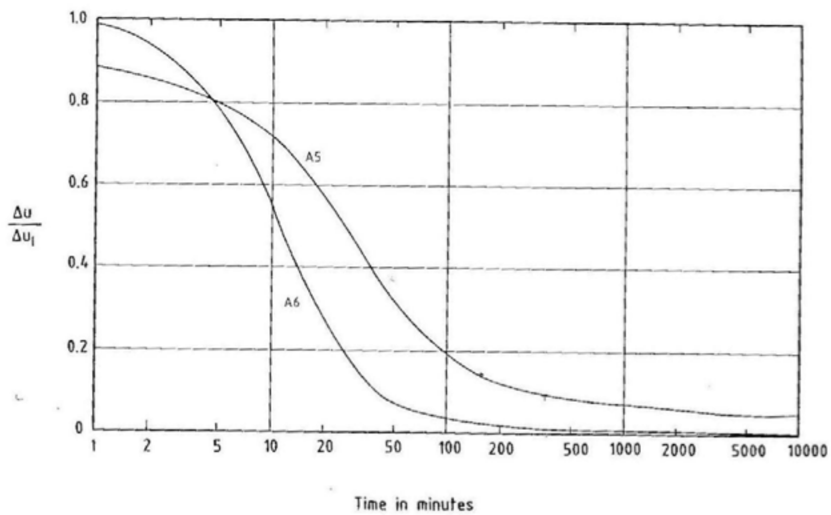

Figure 4.2: Observed pore pressure dissipation for instrumented piles (Karlsrud 2012)

the consolidation, for the logarithmic distribution derived for the EP soil model in Chapter 3. Excess pore pressures close to the pile are initially quickly reduced, but the equalization progresses more slowly after the initial rapid decrease. This corresponds well with reports of rapid increases in bearing capacity shortly after pile installation. The excess pore pressure quickly spreads beyond the initial plastic zone(Randolph & Wroth 1979). The initial excess pore pressure closest to the pile shaft is proportional to the shear strength c_u, and pore pressure and the consolidation time both increase with increasing ratios of G/s_u.

Figure 4.2 shows observed pore pressure values during consolidation for an instrumented pile. Measured results like these are useful for calibrating analytical and numerical models of the consolidation process. The curves by Karlsrud (2012) show similarity in shape to the calculated distributions above.

4.1.2 Consolidation Time

The required consolidation time for full bearing capacity is amongst other factors dependent on the pile diameter, and whether the pile is open- or closed-ended (NGF 2012). The time required for full setup of a closed-ended pile is much larger than for an open-ended pile, and is proportional to the square of the pile diameter (Karlsrud 2012).

Karlsrud (2012) proposes calculating the consolidation time using the following method:

$$t = \frac{T \cdot r_0^2}{c_{h;re}} \tag{4.2}$$

where $c_{h;re} = \frac{M \cdot k}{\gamma_w}$ is an empirical consolidation coefficient and $M = 4 \cdot m_0 \cdot \sigma_c^{'}$. $\sigma_c^{'}$ is preconsolidation pressure, m_0 is the modulus number at 100 kPa above the preconsolidation pressure, k is in-situ vertical permeability, and T is a dimensionless time factor.

4.1.3 Coupled finite element analysis

Some geotechnical problems can be simplified by assuming either fully drained or fully undrained soil behaviour, but real soil behaviour is usually time-related. Pore pressure response is dependent on soil permeability, the rate of loading and the hydraulic boundary conditions (Potts & Zdravković 1999). A coupled analysis combines the equations governing soil deformation due to loading with the equations governing the flow of pore water through the soil skeleton. Each element node has both displacement and pore fluid pressure degrees of freedom.

4.2 Post-consolidation soil properties

After assessing the effect of pile driving on static and dynamic properties of a soft clay, Hunt et al. (2002) report increased stiffness in the soil surrounding the pile, except for the area closest to the pile where stiffness was lower than the initial value. Laboratory tests show that specimens collected after pile installation are denser and lower in water content than pre-installation specimens. Specimens also show a 3-4 times increase in ductility in triaxial shear, though no strength increase is observed.

Borchtchev (2015) demonstrates that increasing strain leads to decreasing residual shear strength, until it reaches remoulded shear strength for completely remoulded material. After reconsolidation, all remoulded samples demonstrate a 1,7-3,6 times increase in shear strength compared to the undrained shear strength of the intact material. However, the strength of all moderately and severely prestrained specimens are lower than the intact strength for reconsolidation at low stress levels. For reconsolidation at stresses in the range 100-200 kPa these also show an increase of 1,2-1,7 s_u.

Borchtchev (2015) also found that the water content after remoulding and reconsolidation is reduced, and that volumetric strain appears to increase linearly with increasing reconsolidation stress. Significant volume changes are observed in moderately and severely prestrained samples. Finally, oedometer testing demonstrates that prestraining and remoulding reduce the coefficient of consolidation considerably in for overconsolidated samples.

The volumetric compressibility of soil depends strongly on the effective stress level and will therefore change during consolidation (Karlsrud 2012). The primary compression index was lower in oedometer testing of remoulded reconsolidated samples, compared to undisturbed samples. The large shear strains and severe remoulding also have a large impact on compressibility characteristics. Karlsrud (2012) found the impact to be relatively small for imposed shear strains below 10 %, and Zheng et al. (2010) found that compressibility and permeability can be assumed constant when the ratio between the compressibility index and the permeability index is small. When this ratio is large, however, the impact on the pore pressure dissipation is found to be significant.

Hunt et al. (2002) found that post-consolidation samples were denser and lower in water content than samples from before pile installation. The remolding caused by pile installation has erased the prior stress history of the samples and they have been compacted. Triaxial shearing tests showed post-consolidation samples were more ductile than pre-installation specimens, with failure strains three to four times higher.

Chapter 5

The Modified Cam Clay Model

Before any numerical modeling is done, a choice must be made between the available PLAXIS material models. These range from the simple elasto-plastic Mohr-Coloumb model to more advanced models such as the Modified Cam Clay model. Pile installation and subsequent consolidation presents a varied set of challenges for a constitutive model. Real soil displays complex behaviour, and the choice of material model will in large part determine how well real soil behaviour can be approximated numerically.

According to Muir Wood (2004), a constitutive elasto-plastic material model must contain the following four components:

- Elastic properties
- Yield Criterion
- Flow rule
- Hardening rule

The modified Cam clay model is one of the most commonly used advanced models. It is based on the concept of Critical State Soil Mechanics (CSSM), developed at the University of Cambridge in the 1960's. The original Cam clay model was presented by Roscoe and Schofield in 1963, while the modified Cam clay model was presented a few years later in 1968 by the same authors. The first numerical implementations were conducted in the early 1970's (Potts and Zdravkovic, 1999).

Critical state soil mechanics assume that the soil will fail when it reaches a critical stress state. In the critical state, large shear deformations can occur without any change in volume or

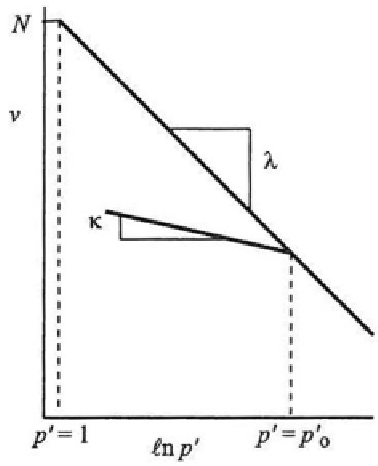

Figure 5.1: Normal isotropic compression line and unloading-reloading line in MCC model (PLAXIS 2015).

effective stresses (Muir Wood, 1990). The critical state combines a failure stress state (*p',q*) with a critical void ratio *e*.

The MCC model is formulated in the p'-q plane, with effective mean stress defined as $p' = \frac{1}{3}(\sigma_1^{'} + \sigma_2^{'} + \sigma_3^{'})$ and the deviatoric stress defined as $q = (\sigma_1^{'} - \sigma_3^{'})$.

The MCC model assumes a natural logarithmic relationship between the void ratio *e* or specific volume *v* and the mean effective stress *p'*, Figure **??**. The response in unloading or reloading is stiffer than in primary loading.

5.0.1 Elastic properties

The slope ?of the unloading-reloading line characterises the elastic volumetric response of the soil, determined by the isotropic swelling index κ (PLAXIS 2015):

$$e - e_0 = -\kappa \ln(\frac{p}{p_0}) \tag{5.1}$$

An infinite number of unloading reloading lines exist, each corresponding to a particular value of the preconsolidation stress p_p. Total strains $d\epsilon$ are comprised of elastic strains $d\epsilon^e$ and plastic strains $d\epsilon^p$.

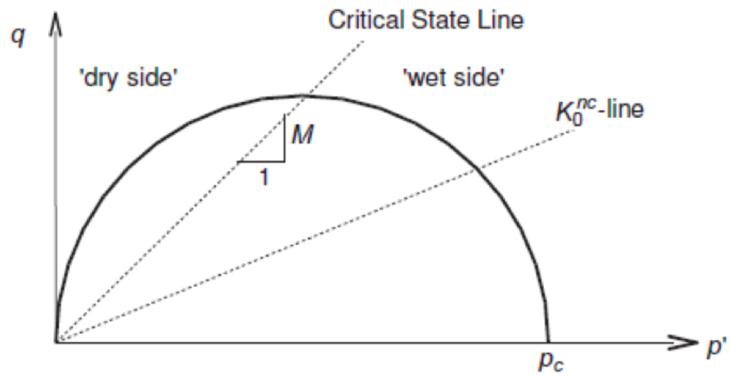

Figure 5.2: Elliptic yield surface (Bergset 2015).

5.0.2 Yield criterion

The yield criterion of the MCC model (Figure 5.2) is bounded by a critical state line (CSL) the p'-q diagram, given by $q = Mp'$, and by elliptical yield surfaces defined by

$$f = \frac{q^2}{M^2} + p'(p' - p_p) \tag{5.2}$$

An infinite number of ellipses exist, each corresponding to a particular value of the isotropic preconsolidation stress p_p. The critical state line intersects the yield surface at the vertex of the ellipse. Stress paths within the elliptic yield surface give purely elastic strains, while crossing the boundary will give both elastic and plastic strain increments.

Figure 5.2 also shows the ?dry" and "wet" sides of the CSL. On the left hand "dry" side, plastic yielding involves softening toward the CSL, and thus failure. On the right hand "wet" side of the CSL, plastic yielding is causes hardening by expanding the yield surface.

5.0.3 Flow rule

The flow rule describes how plastic strains develop when we try to increase the stress level beyond the yield line. The modified Cam clay model is defined using an associated flow rule, where the plastic strain developed is proportional to the stress increment applied.

5.0.4 Hardening rule

The MCC model is a volumetric hardening model. The preconsolidation pressure is used as a hardening parameter relating the plastic part of a change in the specific volume change to a corresponding change in the preconsolidation pressure. The hardening rule describes how the size of the yield surface changes with plastic strains.

5.0.5 Input parameters for PLAXIS

The MCC model requires the following input parameters (PLAXIS 2015):

- $\nu_u r$: Poisson's ratio
- κ: Cam-clay swelling index
- λ: Cam-clay compression index
- M: Tangent of the critical state line
- e_0 Initial void ratio

The isotropic compression index λ determines the compressibility of the material in primary loading

$$e - e_0 = -\lambda \ln(\frac{p}{p_0}) \tag{5.3}$$

M is both a friction constant, a parameter defining the geometry of the ellipse and influences the coefficient of lateral earth pressure K_0^{nc} in normally consolidated soils. To ensure the correct shear strength the M parameter should be based on the Coulomb friction angle ϕ using the following relation, with (-) for triaxial compression and (+) for triaxial extension:

$$M = \frac{6 sin\phi}{3 \pm sin\phi} \tag{5.4}$$

Poisson's ratio $\nu_u r$ can be found from its relationship with the elastic parameters shear modulus G and the non-linear bulk modulus K.

$$G = \frac{3(1 - 2\nu_u r)}{2(1 + \nu_u r)} K \tag{5.5}$$

For the numerical modeling, suitable material parameters must be chosen. These can be obtained from laboratory tests on clay samples, or from experience based values from literature or guidelines.

Zheng et al. (2010) found that compressibility and permeability can be assumed constant when the ratio between the compressibility index and the permeability index is small. The swelling and compression indexes λ and κ can be obtained from oedometer tests using the compression index C_c and the recompression index C_r using the following relations:

$$\lambda = \frac{C_c}{2,3} \tag{5.6}$$

$$\lambda \approx \frac{2C_r}{2,3} \tag{5.7}$$

5.0.6 Suitability

Lee et al. (2003) found that the MCC model predicts undrained strength well, but overestimates excess pore pressures at failure for low plastic clays under anisotropic loading conditions. An inherent limitation of the MCC model is that it cannot describe large-strain softening behavior exactly (Cao et al. 2001), and it allows unrealistically large shear stresses for overconsolidated states with stress paths crossing the critical state line (PLAXIS 2015)

Chapter 6

Numerical Analysis

PLAXIS 2D is a finite element program in common use for numerical analysis of geotechnical problems.

6.1 Aim of the analysis

One aim of this thesis is to use PLAXIS 2D to conduct a numerical analysis of installation and the subsequent changes in pore pressure and effective stresses during the consolidation phase by establishing using different soil parameters for the different zones in the model. The aim is to improve the ability of the soil model to fully capture the impact of the large shear strains, volume changes and stress changes from the installation and reconsolidation phases on the stress-strain and strength characteristics of a clay soil. The soil is divided into zones based on the extent of disturbance or remoulding experienced during the installation phase at different distances from the pile. The different zones have soil parameters representative of the degree of disturbance.

As described in Chapter 4, Karlsrud (2012) determined that large shear strains and severe remoulding have a large impact on compressibility characteristics, but that this impact to be relatively small for imposed shear strains below 10 %. Zones strained less than 10% therefore require no change in compressibility characteristics. It should also be taken into account that the volumetric compressibility of the thin remoulded zone near the pile shaft experiencing primary compression is not equal to that of the zones further out experiencing unloading. The coefficients of permeability and consolidation both change with changes in volumetric strains

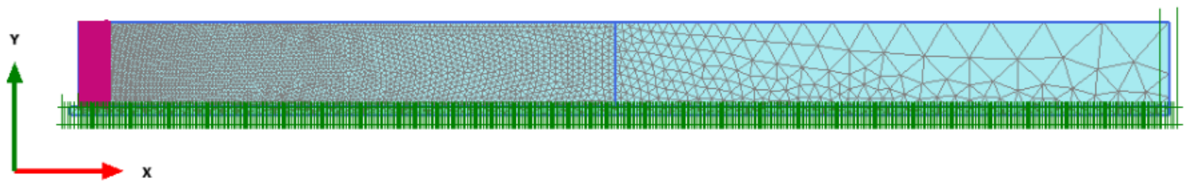

Figure 6.1: A rough approximation of the model setup in PLAXIS

(Borchtchev 2015), and stiffness is also affected. Zheng et al. (2010) found that compressibility and permeability can be assumed constant when the ratio between the compressibility index and the permeability index is small. When this ratio is large, however, the impact on the pore pressure dissipation is found to be significant.

The cavity expansion model (CEM) has been shown to correspond well with initial pore pressure distributions measured after installation, but it cannot capture the influence of the large strains imposed during installation. In this numerical analysis, pore pressures from the cavity expansion are superimposed with pore pressures generated by the varying degrees of remoulding experienced by the surrounding soil.

6.2 PLAXIS model

An axisymmetric model is used in PLAXIS 2D to model a section of the pile. The x-axis represents the radial direction, the z-axis represents the circumferential direction and the y-axis represents the vertical direction. The centre of the pile is the axis of symmetry. In an axisymmetrical model, deformation and stresses are assumed identical anywhere along the circumference of the pile.

Only a vertical section of 1m height along the pile is modeled. The aim of the analysis is to model the radial effects, and vertical effects are neglected. Modeling of the tip is intentionally avoided because it may cause numerical singularities, and since the primary area of interest is the pile shaft for consolidation and time-dependent capacity increase. Possible 3D effects such as silo effects or arching cannot be modeled in this 2D model.

Pile installation is modeled using the CEM method. The cavity expansion is assumed to occur in fully undrained conditions, due to the rapid installation of the pile and the low permeability of clay materials. Carter et al. (1979) conclude that results of sufficient quality are obtained by doubling the cavity size from a finite inital cavity, both for the elastic, perfectly plastic model and the modified Cam-Clay model.

The initial stresses and pore pressures are calculated by a K0-procedure. In the second stage, a prescribed displacement is applied along the left side of the model in Plaxis, expanding the cavity from a_0 = 25 mm to a_f = 50 mm. In the third stage, new materials are activated in the zones representing e.g. a remoulded zone, 50% strain, 10% strain and undisturbed material. In the fourth stage, the reconsolidation process is carried out, and set to last until excess pore pressures have decreased below 1 kPa.

It is assumed that there is a cylindrical cavity in the soil before pile driving and the radius of the cylindrical cavity is defined as a0 . The depth is considered the same as the pile length. During FEM analysis, determination of the maximum enlarged radius is based on the following assumptions: when the radius expands from ao to the maximum enlarged radius, the total radial pressure on the cavity surface is assumed to be equal to the total radial pressure when the radius expands from 0 to r0. Therefore, any changes in the soil volume must satisfy the above assumptions based on the following relationship: a0=3/r0

6.3 Results

The results of the initial calculations are of questionable value. No results are presented here.

Chapter 7

Summary and Recommendations for Further Work

7.1 Summary and Conclusions

The soil surrounding a driven pile is subjected to varying degrees of soil disturbance during the installation, and experiences a complex combination of loading, unloading and reloading in the course of the life cycle of the pile. No current numerical or analytical method is capable of accurately modeling the entirety of the complex deformations and stress changes occurring in real soil during pile installation and subsequent consolidation.

The main objective of this master's thesis has been to perform a numerical simulation of the installation and consolidation of a driven pile using the Cavity Expansion Method and the Modified Cam Clay material. A general approach for a proposed method of numerical simulation which should enhance the modeling of the consolidation process has been proposed. The analysis is designed to include the change in material properties and additional excess pore pressure caused by large strains and partial remoulding of the soil following pile installation, by:

- Superposing the excess pore pressure caused by soil remoulding with the pore pressure resulting from cavity expansion.
- Applying representative sets of material parameters for radial zones with decreasing degrees of soil disturbance.

The main objective has not been reached. Due to reasons briefly mentioned in the preface, no results from the numerical analysis have been presented or discussed. The proposed numerical analysis remains a hypothetical improvement of the exisiting procedures for PLAXIS simulation of pile consolidation.

Secondary objectives were to discuss soil behaviour during the installation and consolidation phases for a driven pile in clay, and to investigate the influence of overconsolidation ratio and plasticity index on the process of pile installation, consolidation and ultimate capacity.

In Chapter 2, a general introduction to pile foundations was given. The installation, consolidation and loading phases were introduced, and displacement piles were defined. A brief note about the Eurocode and pile testing was included. The concept of pile setup was introduced, and the factors contributing to the setup effect were enumerated. The secondary effects thixotropy, creep and ageing were discussed, and methods for determining the ultimate shaft friction were briefly mentioned. The main trends of the influence of OCR and I_p on soil behaviour were presented, and the non-linear unloading-reloading stiffness was discussed as a possible explanation.

In Chapter 3, the soil response to pile driving was discussed. The influence zones of soil disturbance and the generated excess pore pressures were described. The Cavity Expansion Method was presented, including a solution in elasto-plastic soil which gives a logaritmic initial pore pressure distribution. An approximate solution for Modified Cam Clay soils was also presented. The influence of OCR and I_p was mentioned for both solutions.

In Chapter 4, the soil response during the consolidation phase was described. Pore pressure distribution during consolidation, and consolidation time were briefly discussed, and the changes in soil parameters occurring during installation and consolidation were discussed in more detail.

In Chapter 5 the elastic properties, yield criterion, flow rule and hardening rule of the MCC material model was presented, and the input parameters for its PLAXIS implementation were introduced.

Finally, in Chapter 6, the general approach for conducting a consolidation analysis using CEM-MCC in PLAXIS is presented.

7.2 Recommendations for Further Work

- Completing the proposed numerical analysis using suitable material parameters. The study can be extended by performing the analysis for both high and low values of overconsolidation ratio OCR and plasticity index I_p.
- A literature review compiling all available research on the influence of OCR and I_p on pile behaviour, including identifying their correlation with other soil parameters.
- A laboratory study to establish the response of compressibility, permeability, density, stiffness, strength and water content at various strain levels, including complete remoulding and reconsolidation.
- Using Plaxis 2D to study the effects of OCR and I_p on consolidation using different soil models, and comparing to pile test results.

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