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Installation Effects of Non-Displacement Piles

A study of installation-induced soil displacements from drilled steel tube piles and cast-in-place piles

Master's thesis in Infrastructure and Environmental Engineering

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CHALMERS UNIVERSITY OF TECHNOLOGY
Gothenburg, Sweden 2025
www.chalmers.se

MASTER'S THESIS 2025

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Master's Thesis 2025
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Typeset in LaTeX
Printed by Chalmers Digitaltryck
Gothenburg, Sweden 2025

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Abstract

In urban geotechnical design, understanding installation effects of pile foundations is essential, particularly in dense urban areas with challenging soil conditions such as those found in Gothenburg. This thesis explores the installation effects of two types of non-displacement piles; drilled steel tube piles and cast-in-place piles. For drilled steel tube piles, a qualitative interview study was conducted with industry professionals, supported by numerical simulations in Plaxis 2D to assess the impact of soil volume loss through excessive washout. For cast-in-place piles, soil displacements generated by concrete pressure during casting was evaluated using the analytical cavity expansion method (CEM) and the shallow strain path method (SSPM), along with numerical modeling. Both linear elastic and plastic (Soft soil) constitutive models were used and evaluated for each pile type.

Simulations of drilled steel tube piles demonstrated that excessive washout can notably affect ground settlements. For cast-in-place piles, analytical and numerical models aligned well for both lateral and vertical deformations, depending on the chosen method. Simplified pile group simulations further indicated that significant ground displacements may occur even at considerable distances. Interviews confirmed that excessive washout is a known issue among contractors. Method selection should be project-specific, as no solution is universally superior. The critical role of the drill operator in minimizing installation effects was also highlighted. Diverging views on current contract formulations revealed uncertainty in responsibility allocation. Additionally, a tendency to prioritize faster production methods may elevate the risk of installation effects, which can be mitigated by setting limits on allowable ground displacements.

Keywords: geotechnics, non-displacement piles, installation effects, cast-in-place pile, drilled steel tube pile, soil displacements, washout, cavity expansion.

Installationseffekter av icke-massundanträngande pålar
En studie om installationsorsakade jordrörelser från borrade stålrörspålar och grävpålar
ANTON LJUNGBERG
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Sammanfattning

I urban geoteknisk design, är förståelsen för installationseffekter inom pålgrundläggning viktig, särskilt i täta stadsmiljöer med utmanande markförhållanden, såsom de som finns i Göteborg. I detta examensarbetet utforskas installationseffekter för två icke-massundanträngande pålar; borrade stålrörspålar och grävpålar. För borrade stålrörspålar genomfördes en kvalitativ intervjustudie med branschföreträdare, med stöd av numeriska simuleringar i Plaxis 2D för att bedöma påverkan av en markvolymförlust till följd av överdriven urspolning. För grävpålar utvärderas markrörelser orsakade av betongtryck under gjutning med hjälp av cavity expansion metoden (CEM) och shallow strain path metoden (SSPM), samt numerisk modellering. Både linjär elastiska och plastiska (Soft soil) konstitutiva modeller användes och utvärderades för respektive påltyp.

Beräkningar av stålrörspålar visade att överdriven urspolning kan ha en betydande inverkan på sättningar. För grävpålar visade de analytiska och numeriska modellerna god överensstämmelse för både laterala och vertikala deformationer, beroende på vald metod. Förenklade beräkningar av pålgrupper visade dessutom att stora markrörelser kan uppstå även på stora avstånd. Intervjuer bekräftade att överdriven urspolning är ett välkänt problem bland entreprenörer. Metodval betonades som projektspecifikt, då ingen metod är universellt överlägsen. Borroperatörens roll lyftes fram som avgörande för att minimera installationspåverkan. Delade uppfattningar om nuvarande entreprenadformer pekade på en oklar ansvarsfördelning. Dessutom finns en tendens i branschen att föredra metoder med högre produktionstakt, vilket kan öka risken för installationspåverkan, något som kan motverkas genom krav på tillåtna markrörelser.

Sökord: geoteknik, icke-massundanträngande pålar, installationseffekter, grävpålar, borrade stålrörspålar, jordrörelser, urspolning av material, kavitetsförstorning.

Acknowledgements

We would like to express our sincere gratitude to our supervisor at COWI, Leif Jendeby, for his valuable feedback and continuous guidance throughout the course of this thesis. In addition, we thank our colleagues at COWI for their warm welcome, support and encouragement during the semester. Special thanks are also extended to our supervisor and examiner at Chalmers, Mats Karlsson, for his insightful input and reassurance provided during challenging phases of the thesis.

Finally we would like to extend our deepest appreciation to the participants of the interview study. This thesis would not have been possible without them offering up their time, along with their genuine and candid answers.

Anton Ljungberg & Oskar Åström, Gothenburg, June 2025

List of Acronyms

Below is the list of acronyms that have been used throughout this thesis listed in alphabetical order:

AB 04	General Conditions of Contract for Building and Civil Engineering Works and Building Services (Swedish standard)
CEM	Cavity expansion method
CIRIA	Construction Industry Research and Information Association (British non-profit research organization)
DTH	Down-the-hole (type of overburden drilling technique)
FEM	Finite element model
LE	Linear elastic
MC	Mohr-Coulomb
NGI	Norwegian Geotechnical Institute
RC	Reverse circulation
ROP	Rate of penetration
SGI	Swedish Geotechnical Institute
SS	Soft soil
SSPM	Shallow strain path method

Nomenclature

Below is the nomenclature of parameters that have been used throughout this thesis. The parameters have been group based on the section they first are mentioned in.

Parameters

Theory

g	Gravitational acceleration $[m/s^2]$
T	Temperature [°C]
K_{temp}	Temperature coefficient [-]
C_i	Constant [-]
z	Depth [m]
h_{crit}	Critical depth [m]
h_{pour}	Vertical pour height [m]
H_{form}	Vertical form height [m]
R_{rise}	Rise rate [m/min]
p	Internal pressure [kPa]
P_{max}	Maximum pressure [kPa]
v	Velocity $[m/s]$
ρ	Fluid density $[kN/m^3]$
γ_b	Slurry density $[kN/m^3]$
γ_c	Concrete density $[kN/m^3]$

Hellman/Rehnman

b	Width of piling area [m]
l	Length of piling area [m]
d	Pile length [m]
V_{piles}	Volume of installed piles $[m^3]$

V_{plugs}	Volume of pulled plugs [m^3]
η	Heave factor [-]
αx	Countervailing factor [-]
βx	Countervailing factor [-]
γx	Countervailing factor [-]
δx	Countervailing factor [-]

Shallow Strain Path Method

R_{pile}	Pile radius [m]
L_{pile}	Pile length [m]
$2w$	Wall thickness [m]
r	Radial distance [m]
δ_r	Radial displacement [m]
δ_v	Vertical displacement [m]

Cavity Expansion Method

R	Initial cavity radius [m]
a	Radial distance from center of cavity [m]
b	Radial distance to outer boundary [m]
p_0	External pressure [kPa]
G	Shear modulus [kN/m^2]
ν	Poisson's ratio [-]
ϵ_0	Tangential strain [%]

Material Model Parameters

φ' / φ	(Effective) Friction angle [°]
c'	Effective cohesion [kN/m^2]
c_u	Undrained shear strength [kPa]
ψ	Dilatancy angle [°]
M	Critical stress ratio [-]
σ' / σ	(Effective) Stress [kPa]
σ_t	Tensile strength [kN/m^2]
u	Pore pressure [kPa]
p'	Mean effective stress [kPa]

p_p	Isotropic pre-consolidation stress [kPa]
K_0 / K_0^{NC}	At-rest earth pressure coefficient [-]
OCR	Overconsolidation ratio [-]
λ^*	Modified compression index [-]
κ^*	Modified swelling index [-]
C_C	Compression index [-]
C_S	Swelling index [-]
M_0 / M_L	Compression modulus [-]
e_{init}	Initial void ratio [-]
k_x / k_y	Permeability [m/s]
ν_{ur}	Poisson's ratio in unloading/reloading [-]
E	Elasticity modulus [kPa]
E'_{ref}	Reference elasticity modulus [kPa]
E'_{inc}	Elasticity modulus increase [kPa/m]
K	Bulk modulus [kPa]
γ_{sat} / γ_{unsat}	Saturated / unsaturated unit weight [kN/m^3]
EA_1	Axial stiffness [MN/m]
EI	Bending stiffness [$MN/m^2/m$]
w_{plate}	Unit weight [kN/m/m]

Volume Calculations

h	Height [m]
R_{cavity}	Radius of an expanded or created cavity [m]
$R_{cluster}$	Radius of soil cluster [m]
V_{cavity}	Volume of an expanded or created cavity [m^3]
$V_{cluster}$	Volume of soil cluster [m^3]
V_{actual}	Realized volume loss [m^3]
$V_{prescribed}$	Prescribed volume loss [m^3]
V_{factor}	Volumetric factor [-]
$\bar{\delta_r}$	Average radial distance [m]
$\bar{\delta_v}$	Average vertical displacement [m]
ϵ_V	Volumetric strain [%]

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1

Introduction

During the 20th and 21st century, the global population has experienced significant urban development. This global trend is expected to continue, and Gothenburg, Sweden's second-largest city, is no exception. Gothenburg's population is projected to grow by 100 000 inhabitants between 2025 and 2050, driving an urban planning focused on densification (Göteborgs Stad, 2025). This requires additional housing and infrastructure within an already compact urban landscape. However, accommodating this urban growth presents significant challenges due to Gothenburg's underlying soil conditions. The city is situated on deep deposits of clay, dominated by glaciofluvial deposits. This clay is characterized as soft with low shear strength and high sensitivity. These clay layers typically range from 0 to 100 m in depth, with some areas extending even deeper (Bergström et al., 2022).

Given the city's challenging soil conditions, selecting appropriate foundation solutions is critical in ensuring a safe and sustainable urban expansion. Deep foundations are commonly used due to their advantages in low-strength or low-stiffness soil conditions. Piles are favored for their ability to transfer structural loads to deeper soil layers, where higher bearing capacity is achieved.

Piles are generally divided into two categories: displacement and non-displacement. This classification refers to how the soil is affected during installation. Displacement piles, typically driven, displace surrounding soil radially outward, increasing lateral stresses. On the other hand, non-displacement piles are installed through soil removal, which reduces lateral stresses in the ground, before the pile is placed. Therefore, non-displacement piles are generally considered favorable in projects where strict limitations on allowable soil displacements are in place, as the risk of damaging adjacent structures is reduced (Fleming et al., 2008).

Drilled steel tube piles and cast-in-place piles (also referred to as bored piles or drilled shaft piles) are typically categorized as non-displacement piles. In general, these pile types are perceived to not cause "significant" soil displacement. However, recent projects in the Gothenburg area have demonstrated a noticeable impact on the surrounding soil during installation. These observations highlight the need for a deeper understanding regarding these pile types installation effects, particularly given the increasing use of drilled steel tube piles in Sweden (Swedish Commission on Pile Research, 2024).

Previous studies, including a comprehensive research and development project conducted in collaboration between the Norwegian Geotechnical Institute (NGI) and the Norwegian construction sector called BegrensSkade (Eng. *Damage Limitation*), have identified key factors influencing soil disturbances during overburden pile installation, such as change in pore pressures, remolding of the surrounding soil, and volume loss (Baardvik et al., 2016). While these effects have been acknowledged, awareness in the industry remains limited, particularly in soft soil conditions.

Other installation effects, such as noise emission, vibrations, and consolidation settlements due to pore pressure reduction, are well documented, but fall outside the scope of this thesis. Instead, this study focuses on excessive soil washout from drilled steel tube piles and lateral soil displacement and cavity expansion induced by concrete placement in cast-in-place pile shafts. By narrowing the scope of these specific aspects, the research aims to develop a greater understanding of their impacts, ultimately contributing to more reliable and safer construction practices in the future.

1.1 Aim

The goal of this thesis is to collect and evaluate current views, practices, and knowledge on two types of non-displacement piles, cast-in-place piles, and drilled steel tube piles. Information about drilled steel tube piles will be collected by interviewing pilling contractors. In addition to this, numerical calculations will be performed to better understand the effects generated by concrete placement in cast-in-place piles and the creation of a cavity due to the excessive soil washout for drilled steel tube piles. The results from the numerical simulation of cast-in-place piles will be compared with common analytical methods. The aims are explained more precisely as:

- **Drilled steel tube pile**
  - gather and summarize knowledge and experience about current practices around drilled steel tube piles mainly within the Gothenburg area,
  - create a Plaxis 2D model to investigate how the creation of a cavity caused by soil washout from a drilled steel tube pile will cause settlements in nearby soil,
  - investigate the impact on settlements depending on varying cavity shapes due to the washout,
- **Cast-in-place pile**
  - create a Plaxis 2D model that captures and quantifies the horizontal and vertical soil displacements caused by the concrete filling of a cast-in-place pile,
  - compare analytical methods of calculating displacements with the results of the numerical results.

1.2 Limitations

This study is limited to analyzing the effects caused by excessive washout from drilled steel tube piles, as well as the soil displacements and cavity expansion resulting from the concrete filling of cast-in-place piles. A simplified installation approach is considered for both pile types, where the analyses studies the installation-induced effects in a short-term case for singular piles.

2

Theory

The following chapter presents an overview of non-displacement pile types, specifically cast-in-place piles and drilled steel tube piles. This includes their unique challenges, application scenarios, and documented studies related to installation effects, with emphasis on a geology with soft clay deposits overlying glacial till. Analytical calculation methods are introduced for estimating soil displacements due to pile installations. Additionally, the relevant theory regarding numerical calculations, constitutive models and considerations for the finite element modeling process is presented.

2.1 Non-Displacement Piles

Within the literature, various perspectives exist on how pile systems should be defined. In this study, the classification of pile systems outlined by the Swedish Geotechnical Institute (SGI) is adopted to accurately reflect the characteristics of the Swedish market (Olsson and Holm, 1993).

A pile is considered as non-displacing when it does not "significantly" displacet surrounding soil. Unlike driven piles, non-displacement piles are installed by soil removal, either through different forms of boring or drilling, to form the pile shaft. This process is characterized by the soil being loosened and continuously withdrawn during pile installation. Typically, the borehole is supported with either a casing or a drilling fluid (Johnson, 2013; Knappett and Craig, 2019). However, in stable soils, such as stiff clays, the pile shaft may be excavated without additional support and is then considered as an unsupported excavation.

Non-displacement piles can be installed using one of several different methods, however, a key distinction lies in whether the pile is constructed using a mixed-in-place technique or through a boring process (Olsson and Holm, 1993). The bored method is characterized, as its name suggests, by a borehole in the ground being formed either through a boring or excavation procedure. The method is then further distinguished based on if the pile installation is made with either prefabricated components or is cast-in-place. Mixed-in-place methods are commonly executed using an auger screw, where the pile is formed during the installation process through concrete or grout injection (Fleming et al., 2008). Depending on the technique, the soil is either removed or mixed with the filling material to create the pile. This pile type is distinguished from other bored piles by the fact that the borehole is never left open during the entire installation process, therefore it will not be further analyzed in this study. Although the auger screw is used in some cases for the excavation process of cast-in-place piles.

2.2 Cast-In-Place Piles

Cast-in-place piles are a deep foundation technique especially suitable for high and concentrated loads. The method is characterized by the formation of an open hole, either drilled or excavated, which is then directly cast with concrete. These piles are distinguished from other piling techniques due to them often being considerably larger in size, commonly in the range of 0.8-2.0 meters in diameter, but there are examples of both larger and smaller pile diameters being used. Due to their large size, this technique enables the creation of foundations capable of supporting significant axial forces through a combination of side shear and end bearing resistance. When reinforced, the piles are also capable of providing substantial resistance to lateral loads due to their high bending stiffness. The method is also well suited for high rise structures with concentrated loads in urban environments. In Gothenburg, foundations for the Ullevi stadium and the 246 meter tall Karlatornet are examples where cast-in-place piles have been used.

Cast-in-place piles can be adopted in most soil conditions, but are considered favorable in cohesive soils (Brown and Castelli, 2018). The typical installation procedure includes the following steps:

- excavation: The borehole of the pile is either excavated or drilled. This procedure can be performed using different techniques depending on the ground conditions at site and pile requirements,
- stability control and preparation: During excavation it is important to ensure that the borehole remains stable and to minimize the risk of soil loosening, which could lead to collapse. To improve stability different support systems can be used, such as casings or drilling fluids (often a bentonite slurry). Once design depth is reached, the borehole is cleaned to prepare for the installation of reinforcement and concrete,
- placement of reinforcement and concrete: Reinforcement is typically utilized to further enhance the structural capability of cast-in-place piles. The rebar cage (or multiple cages in to-be-welded sections) is lifted down into the shaft, a tremie pipe is then lowered down to the bottom, supplying the concrete. If a casing has been used this is then lifted up consecutively as the concrete level rises. Importantly it should however always be below the concrete level.

In addition to the general installation procedure, the choice of construction method is mainly governed by the soil conditions at the site, which influence the installation technique adopted. An open hole excavation is suitable if the ground conditions on site allow the shaft to be produced without external support, such conditions may be present in stiff clay settings. Otherwise, a support structure is utilized, either in the form of a casing or drilling fluid, or a combination of both. A casing is advantageous if the borehole must extend through soft and unstable soil, or through open water, but it can also be left in place, utilized as a part of the bearing structure.

The primary function of drilling fluids is to stabilize the borehole by counteracting the groundwater head pressure in surrounding soil, this is particularly important under artesian conditions (Brown and Castelli, 2018). This reduces the risk of seepage into the borehole, which could otherwise lead to loss of support around the casing or ground subsidence. Such instability is caused by inflowing water that forms voids around the casing and washes fine materials into the borehole.

To address these challenges, different types of drilling fluids are employed depending on soil and groundwater conditions at site. Water is the simplest and most cost-effective choice to maintain a positive fluid head over the piezometric head pressure and mitigate seepage into the borehole. However, it provides no structural support and is more difficult to control during the drilling process, due to fast dissipation into permeable strata. To counteract this, water mixed with additives such as minerals or polymers are commonly used, forming a fluid referred to as a “slurry” (Brown and Castelli, 2018).

The most common mineral additive is bentonite clay, when mixed with water a colloidal mixture is formed which results in a fluid with higher viscosity than water and a small increase in unit weight (Brown and Castelli, 2018). These properties enhance the control of seepage. Additionally, the fluid improves borehole stability by the formation of a “filter cake”. Once the fluid is added, some minerals from the slurry are filtered out along the borehole walls and enter the surrounding soil. This process decreases permeability, which greatly improves borehole stability during construction. While effective, the usage of bentonite slurries requires careful handling and disposal of fluids, along with installation equipment such as surface reservoirs. Prior to construction these factors should be considered due to additional costs and environmental concerns.

Polymer-based fluids offer an alternative approach. Similar to bentonite, the fluid provides a higher viscosity than water but lacks the formation of a filter cake. Although some indications show a polymer membrane being formed, it has shown to be less impermeable over time compared to a filter cake. Thus, it provides lower support during construction, particularly in coarse grained soils or non-circular excavations. However, this absence may also be beneficial, as the formation of a filter cake weakens the bond between the interfacing soil and concrete. This reduction negatively impacts side resistance, leading to a slight decrease in bearing capacity. Additionally, polymer fluids offer a more practical disposal procedure.

2.2.1 Concrete Casting Pressure

Fresh concrete is a granular liquid before it hardens, and as such develops pressures in a hydrostatic manner, where the pressure increases linearly with depth as long as it remains a liquid. As the concrete starts to hydrate and harden it becomes difficult to determine the imposed pressures. Especially if additional steps are introduced to improve the workability of the concrete through out the pouring process, such as a retarding admixture to slow the hydration process.

The Construction Industry Research and Information Association (CIRIA) published in their 108th report (Clear and Harrison, 1985) a calculation method to estimate the exerted lateral concrete pressure on the formwork. Clear and Harrison (1985) introduced the concept of a critical depth, h_{crit}, explained by the development of effective stresses within the concrete caused by consolidation and cement hydration. The concrete pressure was expected to follow a bi-linear envelope, with the envelope following the full fluid pressure down to the critical height after which the pressure reaches a constant maximum value, see Figure 2.1. Equation 2.1 can be used to calculate the maximum pressure according to the CIRIA guidelines, taking into account factors such as temperature, concrete rise rate and workability.

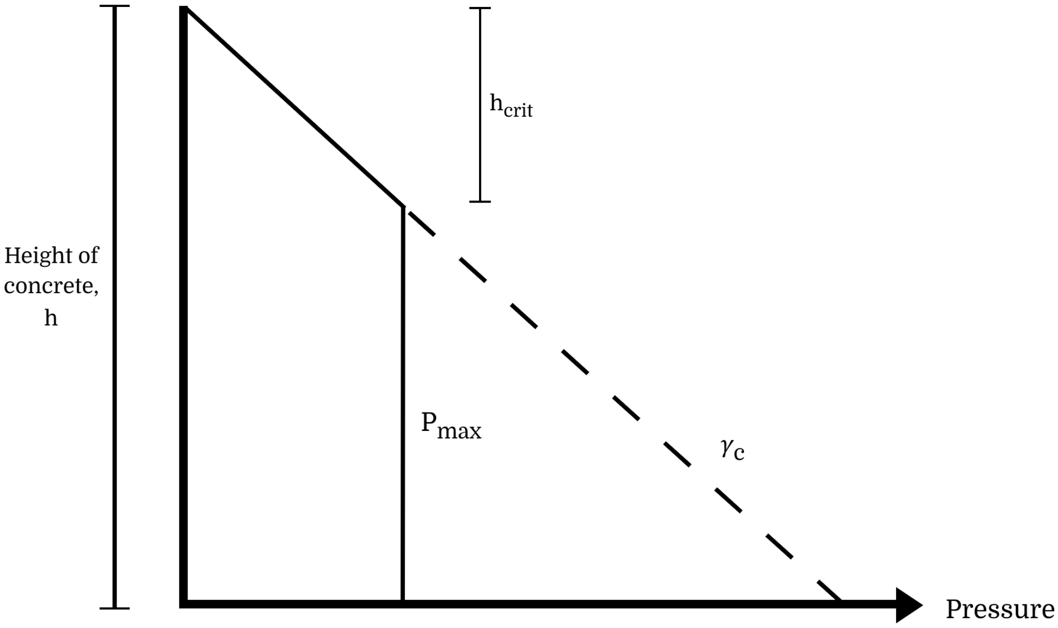


Figure 2.1: Concrete pressure envelope as suggested in CIRIA report 108, γ_c indicating the hydrostatic concrete line (Recreated from Clear and Harrison, 1985).

$$P_{max} = \min \begin{cases} \gamma_c(C_1\sqrt{R_{rise}} + C_2K\sqrt{H_{form} - C_1\sqrt{R_{rise}}}) \\ \gamma_c h_{pour} \end{cases} \tag{2.1}$$

Where
C$_1$ and C$_2$ are tabulated constants depending on the form shape and concrete mixture (including the use of a retarder)
γ_c is the density of concrete $[kN/m^3]$
H is the vertical form height [m]
h is the vertical pour height [m]
R_{rise} is the rise rate [m/min]
T is the concrete temperature [°C]
K_{temp} is a temperature coefficient calculated according to Equation 2.2

$$K_{temp} = \left(\frac{36}{T+16}\right)^2 \tag{2.2}$$

The critical height in meters is then given by Equation 2.3.

$$h_{crit} = \frac{P_{max}}{\gamma_c} \tag{2.3}$$

Lings et al. (1994) investigated the casting pressure in diaphragm walls cast using a tremie pipe and compared their findings to the theory presented by Clear and Harrison (1985). Additionally, they considered the impact of using a support slurry, such as bentonite, where now the pressure envelope follows the hydrostatic slurry pressure below critical depth, see Equation 2.4 and Figure 2.2. Based on three case studies, Lings et al. found that for walls up to 20 meters deep the CIRIA equations give reasonable predictions of the critical depth. For deeper walls the method underpredicts the critical depth and an estimation of one-third the wall height is a more appropriate assumption.

$$P = \begin{cases} \gamma_c z & \text{for } z \leq h_{crit} \\ \gamma_b z + (\gamma_c - \gamma_b)h_{crit} & \text{for } z > h_{crit} \end{cases} \tag{2.4}$$

Where
γ_c is the concrete density $[kN/m^3]$
γ_b is the slurry density $[kN/m^3]$

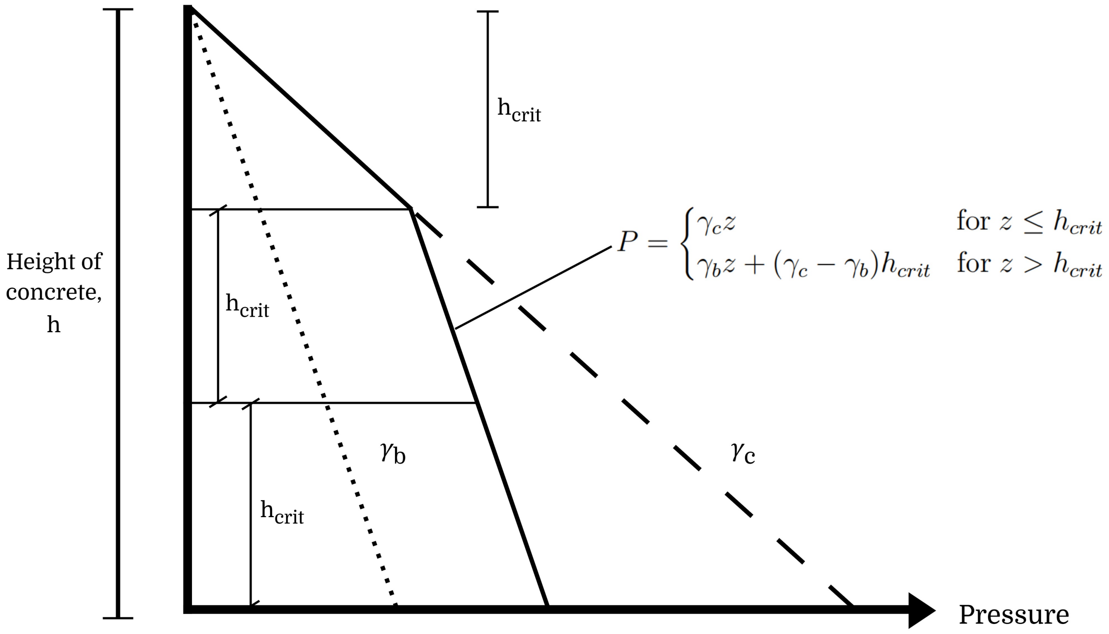


Figure 2.2: Pressure envelope utilizing Equation 2.4 with γ_c and γ_b representing the hydrostatic concrete and slurry lines respectively (Recreated from Lings et al., 1994).

Nissen et al. (2021) have conducted research on the concrete pressure for cast-in-place piles constructed utilizing a temporary casing and oscillation/rotation method, and compared this to the results presented by Lings et al. (1994). Nissen et al. performed measurements on four construction sites in Austria and Germany were piezoresistive pressure transmitters where installed within the reinforcement cages of the piles. Their findings showed that the pressure envelope suggested by Lings et al. under-predicts the casting pressure. Additionally, Nissen et al. did not find any indication of a critical depth phenomenon, as suggested by both Clear and Harrison, and Lings et al. The recorded data instead suggests that the pressure can be expected to fall somewhere between the hydrostatic concrete pressure and the pressure enveloped suggested by Lings et al. Further, their findings show that the pressure at the pile toe is significantly less then the pressure a small distance above it. Nissen et al. goes on to suggest that the hydrostatic concrete pressure is a good estimation of the pressure envelope, especially considering the fact that concrete is typically poured to a slightly higher point than surface level.

2.2.2 Installation Effects of Cast-In-Place Piles

Any adverse effects from the installation of cast-in-place piles have not been widely reported upon, but experiences from some major construction projects in the Gothenburg area where cast-in-place piles were utilized indicates a couple of potential problems. These problems stem from the fact that an estimated 15-20% of additional concrete is needed to be poured to fill the shaft compared to its expected volume (over-consumption). One factor causing this is the drill having set teeth, effectively increasing the drill cross-section by roughly 2-3%, this alone could lead to an extra 6% of concrete being used. Another factor is believed to be a expansion of the shaft cavity due to the additional pressure introduced by the concrete casting.

Other issues include heaving at the surface due to the cavity expansion and subsequent soil displacements, or that fresh concrete can vacate from the shaft at either the bottom of the pile or in a very soft soil layer. A schematic illustration of the soil displacements caused by the casting pressure is shown in Figure 2.3. Also shown is an arching effect between the soft soil and a very stiff soil at the bottom, effectively reducing the expansion potential in the soft soil as the distance between the layers decreases.

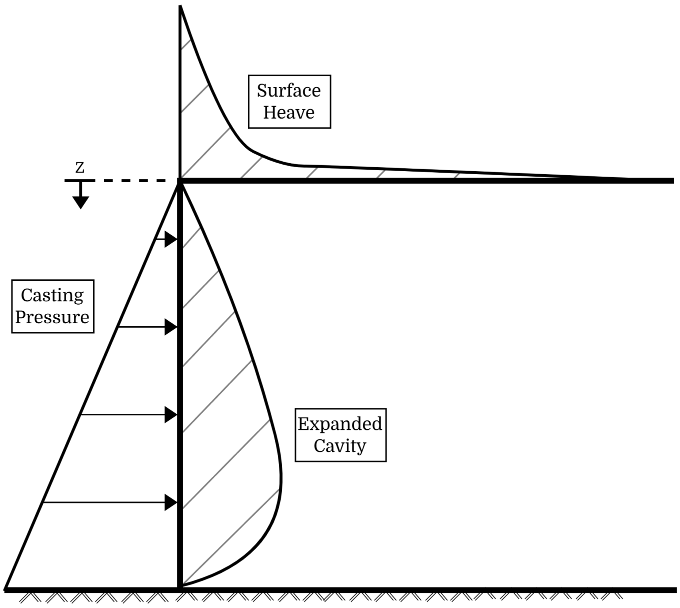


Figure 2.3: Conceptual drawing of surface heave and cavity expansion caused by the casting pressure during installation of a cast-in-place pile. An arching effect between a stiff material and the soft soil results in less expansion close to the stiff material.

2.3 Drilled Steel Tube Piles

Drilled steel tube piles, often also referred to as RD-piles or micropiles, are one of the most widely used piling techniques in Sweden. Its usage has increased in the late 2010s and, according to data from 2023, they constitute 23% of all pile installations in Sweden (Swedish Commission on Pile Research, 2024). Typically, drilled steel tube piles consists of prefabricated cylindrical steel pipe elements that are drilled into the soil until firm strata is reached, in most cases rock. There are two different types of drilled steel tube piles, distinguished either by a solid core section enclosed by a casing, or a hollow section where the casing itself functions as the bearing structure, i.e the pile. The latter configuration is the most widely adopted method today in Sweden (Bredenberg et al., 2010). In addition, the pile can be filled with either concrete or a cement slurry to mitigate corrosion and increase resistance to buckling. The latter can be further improved for the hollow profile by inserting, for instance, reinforcement bars. Usually, the top of the pile is also supplied with a steel plate to improve the transfer of loads from the superstructure (Bredenberg et al., 2010; Finnish Road Administration, 2003).

2.3.1 Drilling Procedure

Drilled steel tube piles are commonly installed using overburden drilling techniques, in Sweden there are two primary approaches used, depending on the ground conditions on site and project requirements. The method can involve either top hammer drilling (i.e. rotary percussive top drive drilling) or down-the-hole hammer drilling (i.e. rotary percussive down-the-hole-hammering, DTH). The two methods are distinguished by the placement of the hammer, mounted either on top of the pile or at its base, as their names suggest. In this study, the top hammer method will not be further discussed as its seldom used in urban environments due to high noise emissions and lower achievable driving depths.

The main components of a DTH system includes a drill rod, the DTH hammer, a drill bit with carbide inserts and a ring bit, see Figure 2.4. The hammer is mounted directly above the pilot and is powered by either a pneumatic or hydraulic pressure. The pressurized medium drives the hammer, delivering percussive strikes on the drill bit while rotational torque is applied through the drill rod. This enables hard formations to be crushed into smaller fragments, so-called cuttings. The cuttings are then flushed to the surface through gaps in the drill bit and transported upward through the annular space between the drill rod and the pile casing.

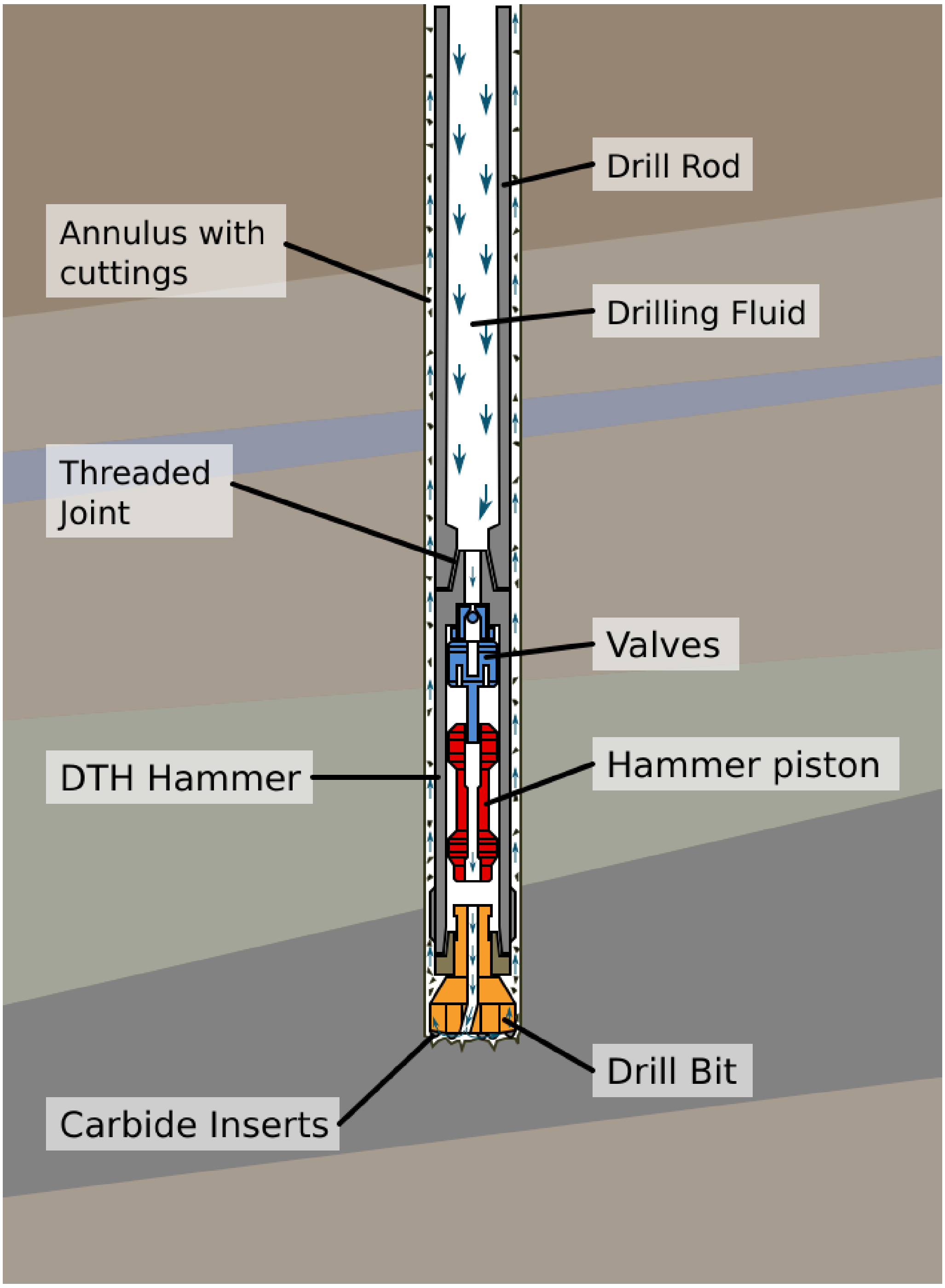


Figure 2.4: Overview of DTH components. (Image by Hartung, 2020, licensed under CC BY-SA 3.0 DE. Retrieved from https://hartrusion.com/wp-content/uploads/dth_hammer_drilling_scheme.png)

2.3.1.1 Eccentric Drilling

For DTH drilling, two different methods are commonly used: eccentric and concentric drilling. Figure 2.5 illustrates the eccentric drilling process. This method consists of a pilot bit and an eccentric reamer. In some cases, a ring bit can be welded to the tip of the casing, which functions as a drive shoe to aid penetration. As the drilling operation begins, the pilot initiates penetration while an eccentric reamer enlarges the hole, allowing the casing to advance. Due to this process the borehole size will exceed that of the casing diameter, forming a small cavity around the casing. Once the target depth is reached, the drill rod rotates in opposite direction, enabling the reamer to close and the hammer to be extracted. The process is finalized by driving the casing with the attached drive shoe to rock and casting concrete in the void made by the drill bit. This is done to avoid the pile being situated at a "shelf" formation. If larger depths are desired, the drilling may continue further into rock with an alternate drill bit designed for drilling in rock (Bredenberg et al., 2010; Finnish Road Administration, 2003). A cost-effective benefit of eccentric drilling is that the pilot can be recovered and reused in other drilling works (Algmark and Eskilsson, 2014).

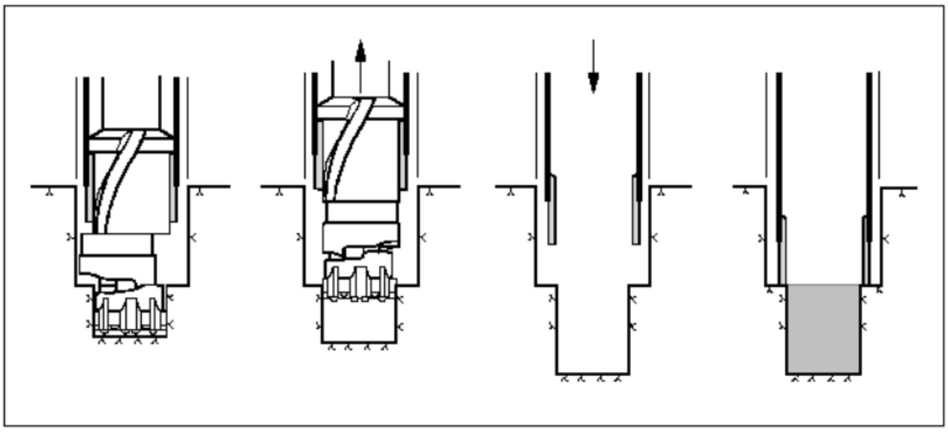


Figure 2.5: Visualization of eccentric drilling with *ODEX* equipment, Atlas Copco (From Bredenberg et al., 2010).

2.3.1.2 Concentric Drilling

In concentric drilling, a drill bit and a concentric reamer (i.e. the ring bit) are used. In older versions, the reamer was attached to the casing. However, today it is more common for the reamer to be mounted on to the pilot during drilling. This adaptation requires less torque, which is advantageous when drilling larger pile diameters. Concentric drilling allows for faster penetration and straighter boreholes compared to eccentric drilling. However, once the target depth is reached, the ring bit is left in place while the pilot is centered and retracted (Finnish Road Administration, 2003), see Figure 2.6. This loss of equipment adds to the overall installation cost (Bredenberg et al., 2014).

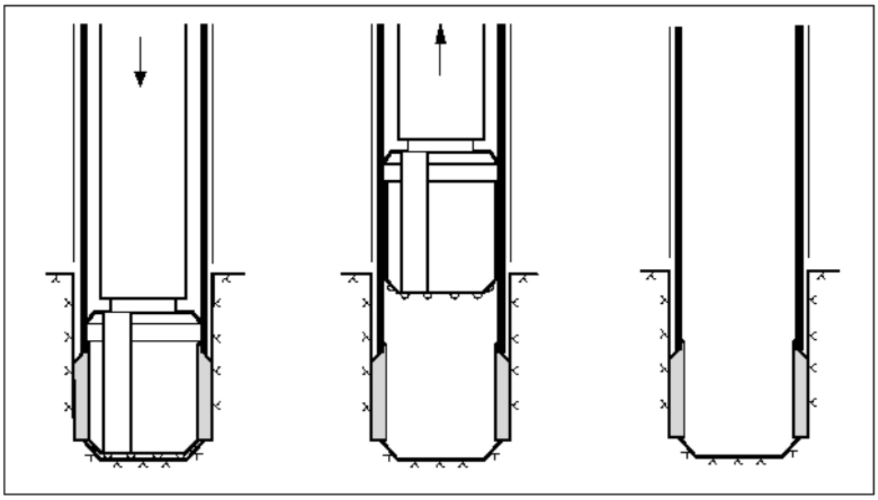


Figure 2.6: Visualization of concentric drilling (From Bredenberg et al., 2010).

Another variation of concentric drilling is the "wing drive" method. This type of drilling uses a pilot equipped with retractable wings that expand laterally, allowing for drilling of holes bigger than the casing diameter, often used when a larger casing is to be installed afterwards. The system functions similarly to an eccentric system, but the concentric method provides improved borehole alignment. If the pilot breaks down during drilling it is inconvenient to retrieve it due to the wing formation (Asplind, 2017; Veslegard et al., 2015).

2.3.1.3 Pneumatic DTH Drilling

As mentioned above, a pressurized medium is used to power the hammer and flush the cuttings. This medium can be either water or air. In Sweden, DTH drilling operations predominantly utilize pressurized air, a method known as pneumatic drilling.

In pneumatic systems, pressurized air enters through a check valve, which regulates the airflow to a cylinder containing a piston that transfers the percussive blows to the hammer. Once the piston is displaced, the pressurized air is released into the borehole and quickly expands due to the pressure differences between the surrounding soil and the borehole. This reduces the density of the air-water mixture in relation to the groundwater, which generates an uplifting force similar to an air-lift pump that allows the cuttings to be transported upward through the annular space between the drill string and pile casing.

An alternative pneumatic drilling technique, which is less commonly used in foundation engineering, is the reverse circulation (RC) method. This system, often pneumatic in nature, uses a double-walled drill rod. This configuration enables cuttings to be transported through the center of the hammer via the additional annular space in the drill rod and hammer, while a support fluid, typically water, is used in the annular space between the casing and drill rod. This approach reduces air being released into the surrounding soil, and the extraction and control of cuttings is improved (Veslegard and Simonsen, 2014). However, practical experience with this technique remains limited, and data from various project applications is scarce. Furthermore, field observations suggest an increased risk of clogging in the pilot.

2.3.1.4 Hydraulic DTH Drilling

For hydraulic drilling operations, a pressurized fluid is utilized to generate impact energy. The most widely known and applied hydraulic drilling system for drilled steel tube piles in Sweden is a water-powered drilling system developed and distributed by Wassara. The company was founded in 1988 as a subsidiary to LKAB, a large international mining company, and the technology was originally developed for mining operations (Wassara, 2023). Since then, its application has expanded and is now used in different types of drilling operations. Similarly to pneumatic drilling, the water is pressurized and drives a piston which delivers impact energy to the hammer, which enables the pile to advance. Once the piston strikes, water is released into the borehole to flush the interface and remove cuttings. Because water is an incompressible medium, the pressure drops as soon as it is released into the borehole and reaches hydrostatic pressure. The released water then creates an upward water flow through the annular space in the casing, induced by the percussion of the hammer which induces the transport of cuttings to the surface.

2.3.2 Installation Effects of Drilled Steel Tube Piles

The extent of research on the installation effects of drilled steel tube piles remains relatively limited. However, one of the most comprehensive investigations to date is the Norwegian BegrensSkade project, developed in collaboration between the Norwegian Geotechnical Institute (NGI) and industry partners. This large-scale research and development initiative focused on minimizing construction-related damage related to ground conditions and foundation work (Baardvik et al., 2016). Throughout the project, several sub-reports were produced, and in the final report, it was highlighted that significant damage could be caused by the installation of drilled steel tube piles, primarily due to settlement-inducing effects. The following factors were found to have the largest influence on damage-causing effects:

- alteration of pore pressures and groundwater level,
- disturbance of clay leading to re-consolidation and volume loss in disturbed zone,
- volume loss through soil uptake and washout.

Furthermore, Baardvik et al. points out that the choice of drilling method dictates the amount of deformation in the surrounding soil, which is supported by a large amount of collected data from construction sites in Norway. Similar effects from the installation of drilled steel tube piles have been reported in Sweden by Bredenberg et al. (2010, 2014).

2.3.2.1 Excessive Washout and Erosion of Soil

From the Norwegian project, Baardvik et al. (2016), conclude that pneumatic DTH drilling operations in soft clay and silt conditions can impose considerable remoulding and disruption in the soil structure. This is partly caused by an uncontrolled discharge of pressurized air into the soil if the return channels on the drill bit become blocked, requiring temporary increased pressure to reopen them. Once released into the surrounding soil, the pressurized air expands and follows the path of least resistance, potentially forming cavities. If this occurs at depths where fractures or permeable layers are present, the air may escape through these pathways, leading to significant soil erosion. This process often goes unnoticed until damage has already occurred in the surrounding soil or structures. Field observations have also shown air can escape along the outside of the casing or even through adjacent piles. The leakage of air into the surrounding soil not only disturbs the soil structure but also contributes to a temporary increase in pore pressures.

Another risk during pneumatic drilling is that the negative pressure generated through the release of pressurized air can induce groundwater flow toward the drill bit, through the air-lift effect. This may also be enhanced by the "Venturi effect", as described by Bredenberg et al. (2014) in relation to overburden drilling, which generates a suction effect around the drill bit resulting in excessive soil uptake, hereafter termed excessive washout. This effect is based on Bernoulli's principle and the resulting suction effect can be described by Equation 2.5, assuming an incompressible fluid with steady viscous flow in a closed system without energy loss (Song, 2018).

$$\frac{p}{\rho g} + \frac{v^2}{2g} + z = C \tag{2.5}$$

Where
p is the pressure [kPa]
ρ is the fluid density [kN/m^3]
g is the gravitational acceleration [m/s^2]
v is the flow velocity [m/s]
z is the height above a reference point [m]
C is a constant for the particular system being studied (Bernoulli's constant)

This phenomenon is caused by the higher water flow generated within the casing that creates a pressure difference relative to the surrounding soil, where zero water flow occurs at a large distance from the pile, see Figure 2.7. As a result, a lower static pressure is created around the drill bit, which may lead to excessive washout, soil erosion, and reduction in pore pressures, with an influence zone extending up to 20 meters (Baardvik et al., 2016; Bredenberg et al., 2014). This effect is most prominent when the borehole enters permeable and erodible layers such as silt, fine sand, glacial till, or pre-existing fractures at greater depths below the groundwater table. In these soil conditions, the rate of penetration (ROP) decreases, prolonging the exposure time and increasing the risk of excessive washout. In reports within the BegrensSkade project, the Venturi effect has been documented, but the associated risks have likely been underestimated. According to Bredenberg et al. (2014), a test was conducted in Sweden where careless drilling was deliberately performed in sensitive conditions, leading to excessive washout. Based on field measurements, the extracted soil volume reached up to ten times the theoretical pile volume. Similar experiences have been reported from piling projects in Gothenburg in recent years.

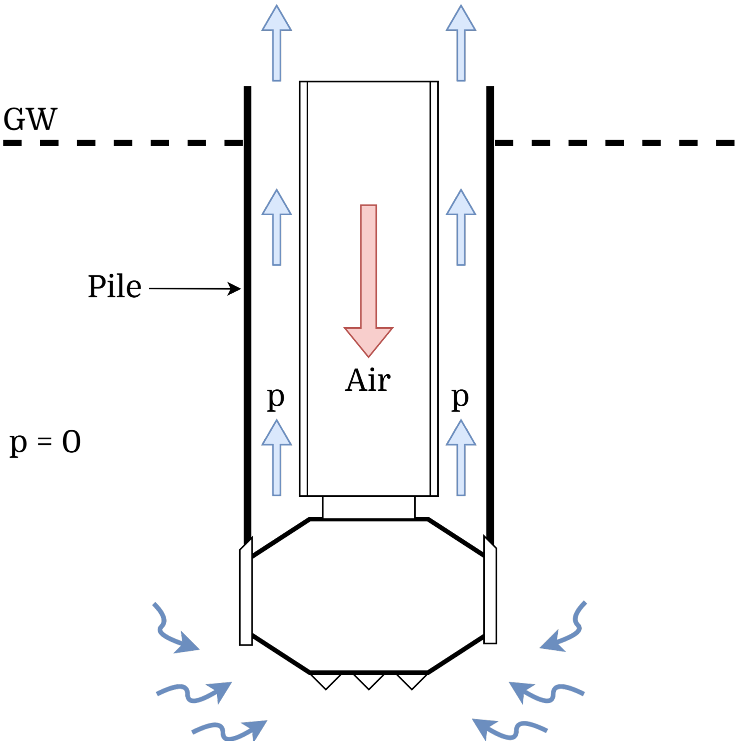


Figure 2.7: Groundwater flow towards the drill bit due to the the Venturi effect (Based on Bredenberg et al., 2014).

The Venturi effect is prominent in pneumatic DTH drilling, however, the magnitude depends on the drill bit and ring bit configuration used. Older drill bits, such as conventional ODEX drilling, will result in more soil washout due to the air being released in a cone shape away from the drill bit. In more recent developments, such as the Elemex drill bit, the release of drilling fluid is concentrated along the drilling interface. These adaptations have been shown to reduce the pressure differences imposed on the surrounding soil, thus reducing the probability of excessive washout occurring (Bredenberg et al., 2014). However, Baardvik et al. (2016) emphasize in their research that there are only two viable options to minimize the risk of excessive washout, which is to use either a water-powered drilling system or a RC-system.

2.3.2.2 Mass Displacement

In contrast to the effects related to excessive washout and soil erosion, the BegrensSkade project also explored the effect of soil displacement during the installation of drilled steel tube piles. Despite these piles being considered "non-displacing" and regarded as favorable in sensitive settings. Observations from the project also identified that in some cases the volume of soil removed was less than the theoretical volume of the pile when drilling in clay, indicating the soil is displaced (Baardvik et al., 2016). Similar results were observed in a Swedish study that tested both hydraulic and pneumatic systems (Ahlund and Ögren, 2016). This displacement effect is governed by the ROP, regardless of the system used. Utilizing a too high ROP will cause soil displacement, leading to a temporary build-up of pore pressures. Field measurements have shown that the pore pressure build-up and influence zone could become considerably larger than that of driven piles. However, the buildup of the pore pressures usually dissipates in a couple of days. The final BegrensSkade report suggests a recommended range of ROP to be $0.5 - 1$ m/min in order to avoid excessive displacements.

2.4 Analytical Calculation Methods

Based on empirical studies and measurements various solutions for calculating the soil displacements, primarily due to pile driving, on both the surface and subsoil have been developed. The Hellman/Rehnman and shallow strain path methods are both based on geometric relationships, and soil parameters are neglected. The cavity expansion method however involves antagonistic pressures acting on a cavity wall, along with the soils shear modulus, to estimate a lateral (radial) displacement.

2.4.1 Hellman/Rehnman Method

A method first suggested by Hellman (Hellman, 1981) and later expanded upon by Rehnman (Olsson and Holm, 1993) is considered the current praxis in Sweden to predict mass displacement and surface heave caused by pile driving. The method is completely based on empirical values and only considers geometric conditions, no soil parameters are required. With Rehnman's revision the method could also be used to predict lateral displacements of the soil, see Equation 2.6. This is achieved by equating the heave at the surface to the lateral movement at the surface, then performing a simplified geometric distribution of the displaced volume, see Figure 2.8. This ratio is given as 1:1 by Rehnman, but experiences from projects in Gothenburg suggests that a ratio of 1:1.5 (vertical to lateral spread) is more appropriate for the clay found in the region.

$$x = \frac{\eta(V_{\text{piles}} - V_{\text{plugs}})}{d\left[(\alpha x + \beta x)\left(\frac{l}{2} + \frac{d}{3}\right) + (\gamma x + \delta x)\left(\frac{b}{2} + \frac{d}{3}\right) + \frac{b \cdot l}{d}\right]} \tag{2.6}$$

Assumptions can be made that the countervailing factors $\alpha x = \beta x = \gamma x = \delta x = 1$, $\eta = 1$ and $V_{plugs} = 0$, which gives a simplified Equation 2.7.

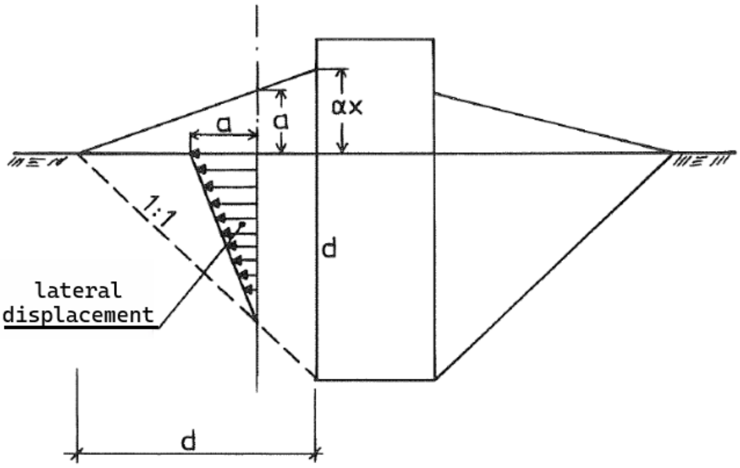


Figure 2.8: Principal sketch of lateral displacement (Modified from Olsson and Holm, 1993).

$$x_v = \frac{V_{\text{piles}}}{\frac{4d^2}{3} + b \cdot l + b \cdot d + l \cdot d} \tag{2.7}$$

Where
V_{piles} is the volume of all piles in the pilling area [m^3]
b the pilling areas width [m]
l the pilling areas length [m]
d is the length of the pile [m]

This method has been shown unreliable at predicting lateral displacements over depth (Andersson and Karlström, 2010; Edstam, 2011; Nyström and Persson, 2016). As such this thesis will not be performing calculations according to this method.

2.4.2 Shallow Strain Path Method

Sagaseta et al. (1997) further developed a method to estimate the surface heave and lateral displacements, which was originally developed by Baligh (1985). The method considers a series of expanding spheres along a vertical line as an approximation of the installation of a driven pile, creating a displacement field within the soil, see Appendix A for more information. While calculating lateral displacements with this method is complex and more comparable to a numerical approach, the displacements at the surface, both radially and vertically, can be analytically solved using the Equation 2.8 & 2.9 respectively. No soil parameters are needed to perform the calculations, instead geometric and volumetric relationships are considered.

$$\delta_r = \frac{R_{pile}^2}{2} \cdot \frac{L_{pile}}{r \cdot \sqrt{r^2 + L_{pile}^2}} \tag{2.8}$$

$$\delta_v = -\frac{R_{pile}^2}{2} \left(\frac{1}{r} - \frac{1}{\sqrt{r^2 + L_{pile}^2}} \right) \tag{2.9}$$

Where
R_{pile} is the pile radius [m]
L_{pile} the pile length [m]
r the radial (lateral) distance from the pile [m]

Equations 2.8 and 2.9 are specifically for full displacement piles. If instead an open ended tube or caisson is installed, with wall thickness $2w$, only the volume of the wall will displace the soil (assuming that no plugging occurs). As such, Equation 2.10 is used to calculate the maximum heave caused by this type of pile.

$$\delta_v = -2wR_{pile}(\frac{1}{R_{pile}} - \frac{1}{\sqrt{R_{pile}^2 + L_{pile}^2}}) \tag{2.10}$$

While the shallow strain path method is designed to predict the impact of specifically driven piles and walls, it can however also be used to model a case where soil removal occurs before pile installation via a "negative" pile.

2.4.3 Cavity Expansion Method

For tunneling, wellbore stability and pilling, a method known as the cavity expansion method can be utilized. The installation of a pile can be considered a radial displacement by the expansion of a cylindrical cavity with infinite length, within a medium of constant volume (Randolph et al., 1979). Hence, if the soil is also not allowed to drain, it must deform while retaining its volume. See Appendix B for more information on the original formulation of the method.

This thesis will utilize the closed form linear elastic solution of a cylindrical cavity expansion problem presented by Yu (2000). This allows for calculations on the lateral displacement of a soil wall caused by the difference between internal and external pressures acting on it. Equation 2.11 links the lateral displacement, δ_r, to the tangential strain, ε_θ, in the case of a fully elastic finite medium.

$$\delta_r = -R\varepsilon_\theta = \frac{p - p_0}{2G(\frac{1}{a^2} - \frac{1}{b^2})}[\frac{1-2\nu}{b^2}R + \frac{1}{R}] \tag{2.11}$$

For an infinite medium, the outer boundary b can be substituted by ∞, which results in the simplified Equation 2.12.

$$\delta_r = \frac{p - p_0}{2G}(\frac{a}{R})^2 R \tag{2.12}$$

Where
G is the soils shear modulus $[kN/m^2]$
p and p_0 are the internal and external pressures acting on the cavity wall [kPa]
R is the initial cavity radius [m]
a is the radial distance from the cavity center where the displacement should be calculated, hence $a = R$ represents the cavity wall [m]

Figure 2.9 shows a schematic representation of the pressures and geometry for this case.

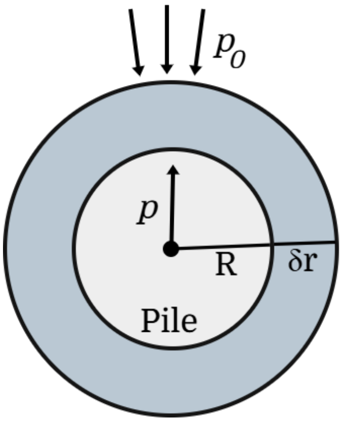


Figure 2.9: Schematic sketch of the cavity expansion method, where a external and internal pressure act on a cavity wall situated at a distance of R from the center of the pile (Based on Yu, 2000).

2.5 Numerical Modeling

Numerical analysis enables geotechnical engineers to tackle complex problems that conventional methods cannot solve, such as intricate geometries, flow conditions, and soil-structure interactions. There is a plethora of software available to geotechnical professionals to perform numerical analysis, this thesis will make use of Plaxis 2D and further information on numerical modeling will reflect this.

2.5.1 Plane Strain and Axisymmetric Geometric Models

The choice of geometric model can be of great importance depending on the problem that needs to be analyzed. It is however important to recognize that these are methods of representing an inherently three-dimensional problem in two dimensions. These geometric models are only valid so long as the problem is homogeneous in the out-of-plane direction, which is rare in real world scenarios.

A plane strain model is used when a uniform cross section can be assumed for a length perpendicular of this cross section. Meaning that the defined cross section will extend infinitely on the Z-axis, see Figure 2.10. The software will consider any strains or displacements in this direction to be zero. Typical problem applications for this geometric model would be embankments or excavations.

The axisymmetric model allows the user to form circular structures by defining a cross section that makes up the radius and height of the structure, which will then be rotated around the Y-axis (i.e. the axis of symmetry), see Figure 2.10. Stresses and deformations will be equal in all radial directions. This could be used to model, for instance, a single pile or circular foundations.

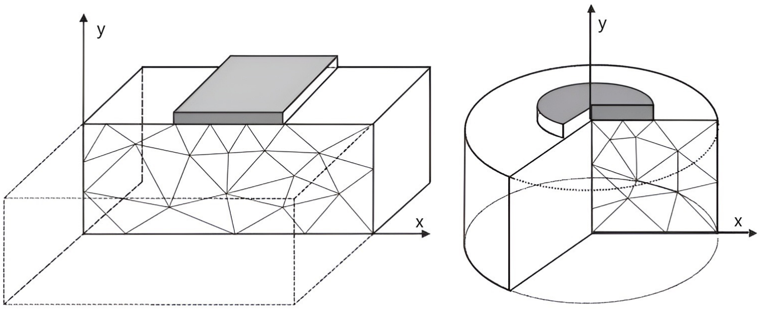


Figure 2.10: Visualization of a plane strain (left) and axisymmetric (right) geometry (From Bentley Systems, 2024b).

A method of modeling a pile or pile group is by simply replacing a soil volume element with a material that corresponds to the pile-material, e.g. concrete. In plane strain the pile group could be represented as a "superpile" by means of superposition, summing all the piles into one. In axisymmetric conditions this could constitute a singular pile, however only axial loading can be applied to it, since lateral loading would void the axisymmetric condition.

2.5.2 Constitutive Models

The constitutive models define how the stress-strain response of a soil is represented using mathematical formulations. A numerical model requires a constitutive model to solve the physical problem being simulated. Its function is to explain the physical interactions of soil using mathematical equations. Idealizations of the physical world must be made to assemble the model, as such different models end up being better for certain behavior predictions (e.g. settlements versus retaining structures) than others.

Early constitutive models considered soils as linear elastic, where a materials elasticity modulus, E, and its Poisson's ratio, ν, ensures that stress and strain are linearly related indefinitely. An extension of this is Mohr-Coulomb, which is a linear elastic perfectly plastic model. Mohr-Coulomb includes a failure criterion, where the material yields from linear elastic to perfectly plastic. This failure criterion is based on the apparent cohesion, c', and friction angle, φ', of the soil and is usually visualized in the principal stress space, see Figure 2.11. The line represents the Coulomb failure envelope and the half-circles are Mohr circles, if a circle extends above the envelope this indicates that plastic failure has occurred.

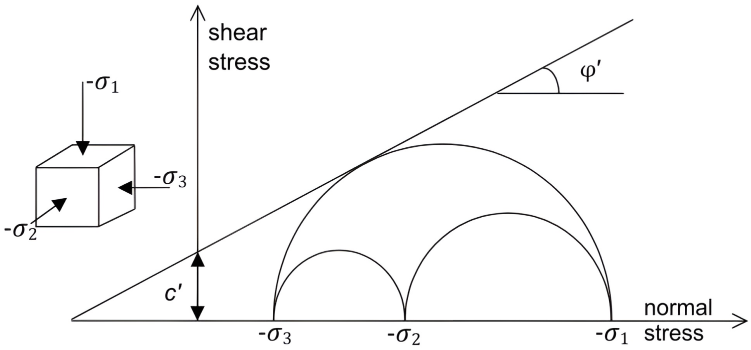


Figure 2.11: Mohr-Coulomb failure criterion (From Bentley Systems, 2024a).

Linear models are generally regarded as sufficient to model rock and concrete, but soil exhibits a non-linearity that these “simple” models cannot capture. Improved models have been developed that can allow different failure surfaces for volumetric and shear strain hardening (Carter, 2024). These are aimed at capturing the complexity of real soils, allowing for dilation and contraction as shearing occurs. Other models have implemented kinematic or collapsing yield surfaces, allowing the yield surface to translocate or transform depending on the measured anisotropic response or degradation of soil particle bonds.

While the more complex constitutive models can capture the more intricate behavior of soils, they typically require more input parameters to function (Carter, 2024). This adds to the lack of knowledge about numerical modeling, as the geotechnical engineer now also must choose an appropriate constitutive model for the task at hand. Potentially resulting in even less understanding of the calculations due to the increased complexity of these advanced soil models. Another issue to consider is the amount of other simplifications or estimations around the problem formulation that the constitutive model does not capture. A more "correct" soil behavior prediction might not make up for these other aspects.

2.5.2.1 The Soft Soil Constitutive Model

Near-normally consolidated clays, peat and clayey silts are generally considered as soft soils. Typically these soils showcase a tangent stiffness moduli much lower then that of normally consolidated sands in oedometer tests, a fact that is owned due to soft soils compressibility. The Soft soil model was derived from the Modified Cam-clay model by implementing a Mohr-Coulomb failure criterion in lieu of a Drucker-Prager criterion, which overestimates undrained strength (Karstunen, 2024). Additionally, a critical state line referred to as the M-line is introduced, see Figure 2.12. The M-line determines the peak stress state once failure has been reached, i.e. after the Mohr-Coulomb failure line has been passed. M relates to the at-rest earth pressure coefficient, K_0^{NC}. Within Plaxis, the user inputs either a K_0^{NC} or friction angle value depending on the setting, and M is automatically calculated based on Equation 2.13 and Jaky's formula, Equation 2.14.

$$sin\varphi' = \frac{3M}{6+M} \tag{2.13}$$

$$K_0^{NC} = 1 - sin\varphi' \tag{2.14}$$

Where
φ' is the effective friction angle [°]
M is the critical stress ratio [-]

A cap based on the isotropic pre-consolidation stress, p_p, represents the stress history of the soil and limits the extent of the yield surface ellipse. The threshold ellipse limits how small the yield surface can be, ensuring that the cap remains in the compressive zone.

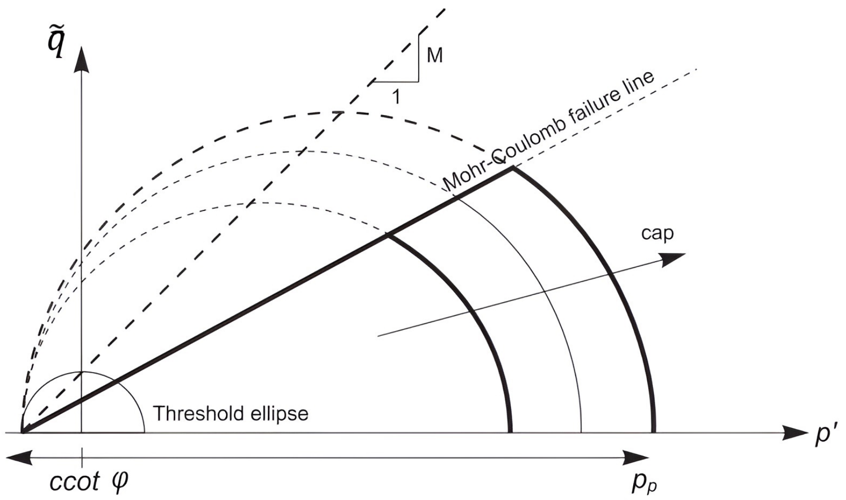


Figure 2.12: A depiction of the Soft soil yield surface (M-line), failure line (Mohr-Coulomb) and cap (From Bentley Systems, 2024a).

Instead of directly utilizing a stiffness modulus, the soils stiffness in primary loading and unloading/reloading is determined from the effective mean stress and the modified compression index, λ^*, and modified swelling index, κ^*, respectively. These indexes represent the logarithmic relation between volumetric strain and mean effective stress and can be linked to the more standard one-dimensional compression and swelling indexes, C_C and C_S (or the commonly used M_0 and M_L parameters in Sweden). This approach allows for the model to develop stress-strain paths in a more realistic manner than Mohr-Coulomb as the materials elasticity is now non-linear, see Figure 2.13. Mechanical input parameters for the Soft soil model in Plaxis can be found in Table 2.1.

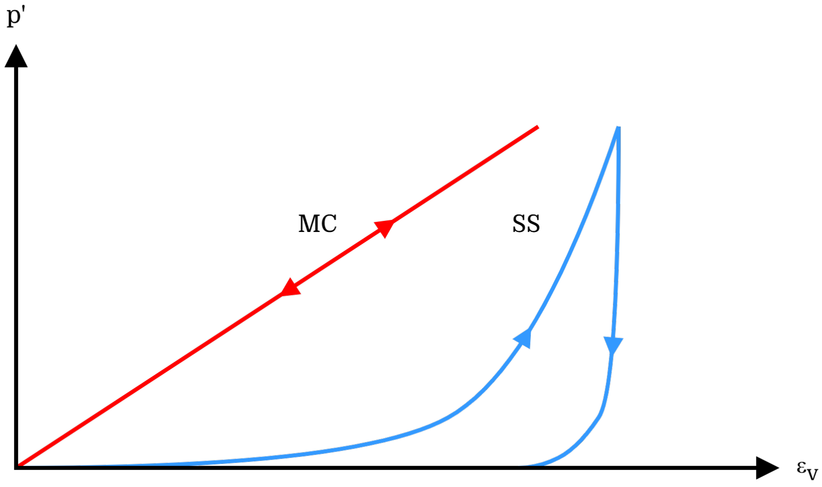


Figure 2.13: Comparison of Soft soil (blue) and Mohr-Coulomb (red) stress-strain paths (Recreated from Karstunen, 2024).

Table 2.1: Soft soil input parameters (Bentley Systems, 2024a).

Required parameters

λ^*	Modified compression index	[-]
κ^*	Modified swelling index	[-]
ν_{ur}	Poisson's ratio in unloading/reloading	[-]
c'	Effective cohesion	[kN/m^2]
φ'	Effective friction angle	[°]
ψ	Dilatancy angle	[°]
σ_t	Tensile strength	[kN/m^2]

Optional parameters

K_0^{NC}	Coefficient of lateral stress in normal consolidation	[-]
M	K_0^{NC}-parameter	[-]

2.5.3 Drainage Conditions

The effect of pore-water pressures on the behavior of soil is well established. Terzaghi proposed that stresses in the soil can be split into effective stresses, σ', and pore pressures, u, according to Equation 2.15.

$$\sigma = \sigma' + u \tag{2.15}$$

This principle is considered the most important concept in soil mechanics and is one of the first things taught in undergraduate geotechnical courses. Depending on the ground water level, drainage conditions and soil type, a material can be considered drained or undrained (realistically an intermediate partially drained state also exists). This relates to pore pressures existing in the soil or not, and consequently if consolidation can occur.

Within Plaxis all material models are based on the relationship between strain, ε', and effective stress, σ' (Bentley Systems, 2024a). As such it is important to define whether or not the material is drained or undrained, since this will effect the effective stress. Plaxis has three different drainage models for undrained behavior, although not all constitutive models can use every drainage model. These are called Undrained A, B or C. Undrained A and B are based on effective stresses, while Undrained C is based on total stresses. Additionally, one can define the material as Drained.

Undrained A combines the effective strength parameters of effective friction angle, φ', and the effective cohesion, c', to determine the materials undrained shear strength, c_u. When using this drainage model Plaxis will automatically append the stiffness of water into the stiffness matrix, this allows for the delineation of effective stresses and pore pressures. Additionally, effective values for the materials stiffness parameters will have to be used.

Undrained B functions similarly to Undrained A, with the change that undrained shear strength is now a directly used as a parameter instead. This is useful when the profile of undrained shear strength through out the layer is known and results in Plaxis setting the friction angle to zero and the cohesion being equal to the undrained shear strength. This approach allows for $\mathrm{c_u}$ to be independent of effective stresses and pore pressures as is the case with Undrained A.

Undrained C differs from A and B by using total stresses to perform the analysis. This will require undrained parameters for stiffness parameters, while the undrained shear strength and $\varphi' = 0$ will be used. This drainage type is generally not recommended due to there being no established contrast between pore water pressure and effective stress. As such it is illogical to perform a consolidation analysis using this model, since no pore pressures are developed, but it could potentially be used for very short-term problems.

3

Methods

This section outlines the research methodology adopted in the thesis. The choice of methods is motivated by their suitability for addressing the research questions. In Figure 3.1, a flow chart of the framework of the methods is presented.

3.1 Research Approach

The study utilized a mixed-methods approach, combining a literature review, interview study, analytical calculations, and numerical modeling, see Figure 3.1. The rationale behind this methodology was to establish theoretical knowledge, gather empirical insights, and validate the findings through numerical simulations. The research was structured in sequential phases, where the literature review served as the foundation in the design and development of the interviews, while also providing the knowledge that guided the analytical calculations and numerical simulations. Through the combined approach the study aimed, within its set limitations, to develop a comprehensive understanding of the installation effects of cast-in-place piles and drilled steel tube piles.

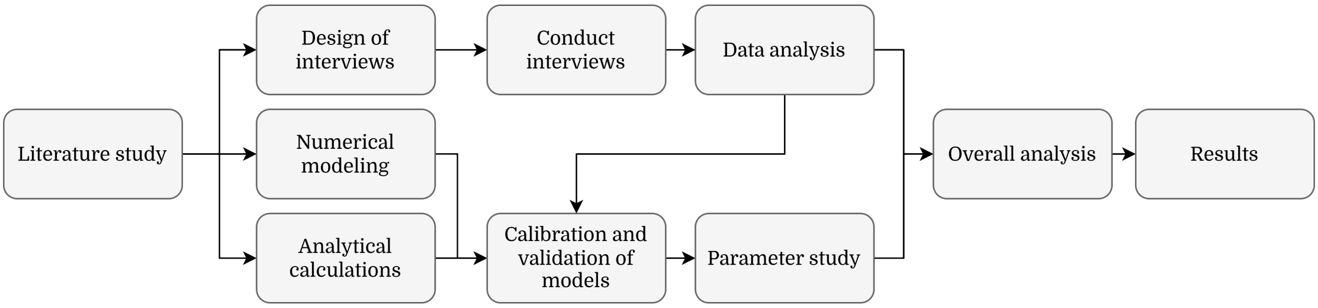


Figure 3.1: Visualization of the research approach adopted.

3.2 Interview Study

As part of the mixed-methods approach, a qualitative interview study was conducted. The choice of a qualitative method was motivated by its capacity to explore non-numerical data and gain in-depth empirical insights. This was considered advantageous to the aim of the study, which is to develop a deeper understanding of the piling industry's knowledge and perception of drilled steel tube piles. The interviews were conducted in a semi-structured format, which is often adopted in qualitative research, as it allows the conversation to be guided by predefined questions, while still maintaining flexibility for the participants to elaborate on relevant empirical insights (Brinkmann, 2024). In Appendix C the predefined questions are outlined.

Interview participants were selected based on their expertise and experience on contracting work of geotechnical retaining structures and deep foundations utilizing drilled steel tube piles within the Gothenburg area. The selection of candidates was made through personal contact with companies or with assistance from the academic supervisors, who helped refer to suitable individuals with relevant experience. Once a sufficient pool of potential candidates had been established, all candidates were contacted with an invitation to take part in the study. In total five interviews were completed, conducted either through site visits or digital meetings. The interviews lasted approximately 60 to 90 minutes and were held in either Swedish or English. Once a participant had accepted to take part in the study, they were informed of the predefined questions. Prior to conducting the interview, each participant was asked and gave their consent to have the conversation recorded to ease transcription and gave permission to use their company name and role, to strengthen the trustworthiness of the study.

The participating companies were the following:
Aarsleff Ground Engineering
Hercules Grundläggning
Keller Grundläggning
Nordisk Grundteknik
Skanska Grundläggning

The roles of the participants included two construction managers, one project manager and two technical specialists.

After the interviews were transcribed, the data was analyzed and summarized in relation to the main topics outlined in the interview questions. This was done to retain most of the data and ensure that industry experiences and insights were represented fairly, while also providing support for the numerical modeling and objectives of the study.

3.3 Numerical and Analytical Calculations

This section covers the methods used for both the analytical and numerical calculations. The soil profile and relevant parameters that were used during the different calculations are also presented here.

3.3.1 Soil Profile

A simplified profile was considered, including a single layer of clay overlying a 5 meter thick till layer before bedrock is reached, see Figure 3.2. The thickness of the clay layer varied depending on the pile length being studied (this is covered in a later section). A unit weight of 16 kN/m^3 and 22 kN/m^3 was determined for the clay and till respectively. Effective friction angles for the clay was 30° and 35° for the till. A constant elasticity modulus of 150 MN/m^2 was used for the till, while the clay was determined from a relationship between elasticity and undrained shear strength, referred to as the $E - c_u$ factor forwards. Gothenburg clay can be generalized as having a undrained shear strength increasing with depth according to the following expression:

$$c_u = 13 + 1.3z$$

Guidelines from the Swedish Transport Administration (2023) for highly plastic clay (such as the one found in Gothenburg) states that an elastic modulus of $E = 250c_u$ is recommended. However, to avoid unrealistically low stiffness values, associated with near-mud conditions, a slightly higher factor of $375c_u$ was chosen to serve as a baseline for the parameter study.

The parameters presented here gives an overview of the most relevant parameters for both the numerical and analytical calculations as a whole, but modeling in Plaxis requires more input parameters. These additional parameters will be introduced in Section 3.3.2.1.

3.3.2 Numerical Calculations

This section explains the model considerations and the setup for the two pile types. First, the geometric and soil parameters shared by both models are presented. This is followed by individual sections for each respective pile type, which cover aspects unique to that particular pile.

3.3.2.1 Material Models and Geometry

Each soil material was tested for two types of behavior: fully elastic and plastic. This was achieved by using different material models within Plaxis. For the clay layer, a linear elastic (LE) model and the Soft soil (SS) model were used. For the till, the LE model and the Mohr-Coulomb (MC) model were applied.

Both the cast-in-place pile and the drilled steel tube pile were analyzed using the same soil parameters. However, certain distinctions were introduced to account for

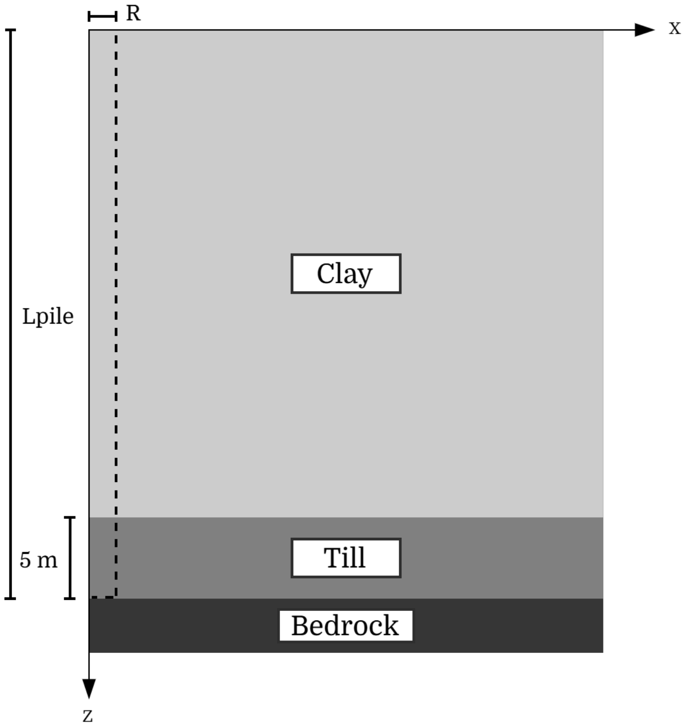


Figure 3.2: Overview of the soil profile used for both the cast-in-place pile and the drilled steel tube pile models. Important dimensions have been marked.

their differing installation effects and behavior. The models have been constructed as 2D axisymmetric, using 15-noded elements, with the pile aligned along the axis of symmetry. This approach provided a quasi-3D representation of the problem, offering computational efficiency compared to full 3D modeling. A known limitation of this method is that all elements are assumed to rotate around the axis of symmetry. As such, no structural elements beyond the pile itself are included, and pile-structure interaction effects are excluded from this study.

A shared baseline array of parameters was determined for both pile types and for all constitutive soil models (LE, MC, and SS). The base parameters, listed in Tables 3.1, 3.2, and 3.3, reflect typical properties of the soft clay and glacial till found in the Gothenburg region as explained in Section 3.3.1. Undrained A conditions were used throughout all the soil variants, with the groundwater level set at ground level. Undrained analysis is appropriate when quick or instantaneous events occur, such as in the cases in this thesis, as pore water does not have time to dissipate.

To calculate appropriate κ^* values for the SS models Equation 3.4 was used by matching the elasticity modulus at the bottom of the clay layer using the $E - c_u$ factor. Consequently, this approach results in a lower elasticity modulus at the top of the clay layer compared to the corresponding LE model. The value of κ^* is also dependent on the soil unit weight and Poisson's ratio, along with where the groundwater table is located, implicating that caution should be used if these needs to be changed.

Table 3.1: Linear elastic material parameters for the clay and till layers. Both used Undrained A.

Clay

$\gamma = \gamma_{sat}$	Unit weight	16	[kN/m^3]
e_{init}	Initial void ratio	1.93	[-]
E'_{ref}	Reference elasticity modulus	4875 (375c$_u$)	[kN/m^2]
E'_{inc}	Elasticity modulus increase	487	[kN/m^2/m]
ν	Poisson's ratio	0.3	[-]
$k_x = k_y$	Permeability	9.4E-10	[m/s]
K_0	Earth pressure coefficient	0.5	[-]

Till

$\gamma = \gamma_{sat}$	Unit weight	22	[kN/m^3]
E'_{ref}	Reference elasticity modulus	150	[MN/m^2]
ν	Poisson's ratio	0.3	[-]
$k_x = k_y$	Permeability	1.16E-5	[m/s]
K_0	Earth pressure coefficient	0.43	[-]

In terms of geometry, a baseline pile length of 60 meters was selected with variations of 30 and 90 meters also analyzed in the parameter study. The models extended in the X-axis direction to a length of five times the studied pile length. This was done to eliminate any potential impact of the far vertical boundary on the results. The 5-meter-thick till layer was modeled above a fixed boundary representing solid bedrock, with the remaining soil profile consisting solely of clay. The piles are assumed to be end-bearing on the fixed bottom boundary, with penetration into rock not being considered. Additionally, to satisfy the symmetry of the model the leftmost vertical boundary was set to be impermeable for groundwater flow.

Table 3.2: Mohr-Coulomb material parameters for the till layer. Undrained A was used for this material.

$\gamma = \gamma_{sat}$	Unit weight	22	[kN/m^3]
E'_{ref}	Reference elasticity modulus	150	[MN/m^2]
ν	Poisson's ratio	0.3	[-]
φ'	Effective friction angle	35	[°]
$k_x = k_y$	Permeability	1.16E-5	[m/s]
K_0	Earth pressure coefficient	0.43	[-]

Table 3.3: Soft soil material parameters for the clay layer. Undrained A was used.

$\gamma = \gamma_{sat}$	Unit weight	16	[kN/m^3]
e_{init}	Initial void ratio	1.93	[-]
λ^*	Modified compression index	0.083	[-]
κ^*	Modified swelling index	0.0125 (375$\mathrm{c_u}$)	[-]
ν_{ur}	Poisson's ratio in unloading/reloading	0.2	[-]
c'	Cohesion	2	[kN/m^2]
φ'	Effective friction angle	30	[°]
K_0^{NC}	NC earth pressure coefficient	0.5	[-]
$k_x = k_y$	Permeability	9.4E-10	[m/s]
K_0	Earth pressure coefficient	0.67	[-]
OCR	Overconsolidation ratio	1.2	[-]

3.3.2.2 Cast-In-Place Pile Model

An illustration of the Plaxis model for the cast-in-place pile can be seen in Figure 3.3. The baseline model for the cast-in-place pile was made up of a pile with a radius of 1 meter and 60 meters in length. The pile and its installation were modeled in Plaxis by deactivating a soil cluster along the axis of symmetry corresponding to the pile radius, creating an empty shaft. The deactivated cluster was allowed to be filled with water and a perpendicular line load was applied along the cavity wall. This line load represents the effective hydrostatic concrete pressure of 15 kPa/m as if the borehole was completely filled with liquid concrete. The hydrostatic water and effective concrete pressures collectively add up to the full casting pressure of 25 kPa/m.

In the second phase the line load was switched off and the cluster making up the pile was reactivated and replaced with a material of cured concrete, see Table 3.4. Overall the model consists of three construction stages, these are outlined in Table 3.5.

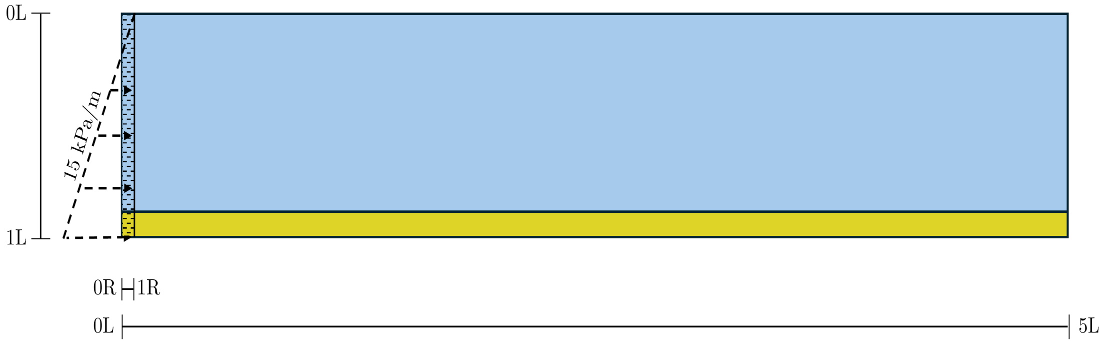


Figure 3.3: Illustration of the Plaxis model for the cast-in-place pile. The blue material is clay, the yellow is till. The dashed area represents the soil volume that will be removed and then switched to a hardened concrete material.

Table 3.4: Linear elastic, non-porous concrete material parameters.

γ_{unsat}	Unit weight	25	[kN/m^3]
E'_{ref}	Reference elasticity modulus	30E3	[MN/m^2]
ν	Poisson's ratio	0.2	[-]

Table 3.5: Construction stages in Plaxis for the cast-in-place pile.

Name	Description	Type
Initial phase	Calculation of initial stresses	K_0-procedure
Phase 1	Pile soil cluster deactivated, line load applied on the cavity wall	Plastic
Phase 2	Line load switched off, pile soil cluster activated and replaced with concrete	Plastic

3.3.2.3 Drilled Steel Tube Pile Model

The baseline drilled steel tube pile model was made up of a pile, represented as a plate element with a radius of 0.16 meters and a length of 60 meters, see Figure 3.4. This represents a simplification of a typical drilled steel tube pile with a diameter of 323 mm. The plate material parameters are presented in Table 3.6. The pile installation was modeled by activating the plate element and applying a prescribed displacement set to zero at the ground level, while the soil cluster adjacent to the axis of symmetry was deactivated and set to dry. An interface element was activated along the plate element with the strength determination set to manual with $R_{inter} = 0.66$. Once the pile was in place, a prescribed uniform contraction (i.e. negative volumetric strain) was applied to a soil polygon, representing a volume loss equivalent to ten times the theoretical volume of the pile. This represented a worst-case scenario of excessive washout, inspired by field measurements of careless drilling from Bredenberg et al. (2014). The soil polygon was positioned adjacent to the pile, within the till layer, with a thickness equal to the full height of the till and a width corresponding to 50 times the pile radius. In total, three construction stages were performed and are listed in Table 3.7.

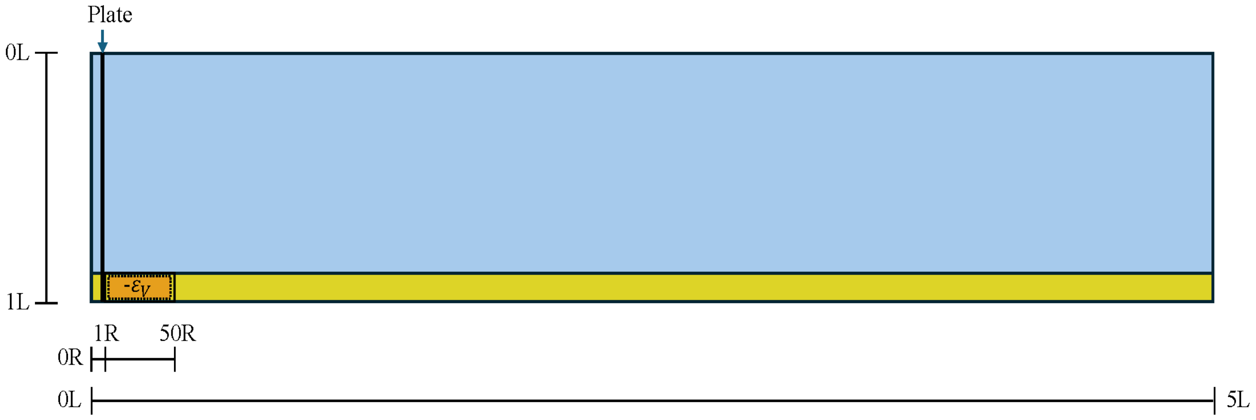


Figure 3.4: Illustration of the Plaxis model for the drilled steel tube pile. The blue material is clay, the yellow is till. The orange area represents the soil volume that will have a contraction applied to it. The plate elements makes out the wall of the steel tube pile.

Table 3.6: Plate material parameters.

w_{plate}	Unit weight	0.77	[kN/m/m]
EA_1	Axial stiffness	65	[MN/m]
EI	Bending stiffness	25.5	[MN/m^2/m]
ν	Poisson's ratio	0.2	[-]

Table 3.7: Construction stages in Plaxis for the drilled steel tube pile.

Name	**Description**	**Type**
Initial phase	Calculation of initial stresses	K_0-procedure
Phase 1	Nil step (reset displacements and small strains)	Plastic
Phase 2	Pile soil cluster deactivated and set to dry, plate element activated, volumetric strains applied	Plastic

3.3.2.4 Mesh Generation

To ensure numerical accuracy and reduce uncertainties related to the mesh itself, the mesh was refined in both pile models until the results converged, i.e. the results were no longer affected by additional refinement of the mesh size. The "Update mesh" feature was then also tested, yielding the same results as the refined mesh, signifying that the mesh did not need additional refinement.

3.3.3 Analytical Calculations

Estimates of the expansion of a circular cavity due to a prescribed increase of internal pressure following the concreting of a cast-in-place pile was performed using the cavity expansion method. The surface heave caused by this expansion was also estimated by adopting the shallow strain path method for an open ended tube.
Calculations of the vertical displacements caused by a pile group has also been studied for both the cast-in-place pile and drilled steel tube pile. This was done by taking the heave or settlement curve predicted for a singular pile and using the superposition principle to sum the displacements caused by multiple identical piles within a given piling area.

3.3.3.1 Cavity Expansion Method

The linear elastic solution for cavity expansion, as given in Equation 2.12, was used to model the lateral displacements experienced during concrete casting. To perform these calculations an internal pressure was needed. This pressure was represented by the effective hydrostatic casting pressure, increased linearly with depth at a rate of $25 - 10 = 15$ kPa/m. The internal pressure follows the findings of Nissen et al. (2021), whose work aligns with the objectives of this thesis. Adopting the full hydrostatic casting pressure can be considered a conservative approach, representing a worst-case scenario.

The external pressure on the cavity wall corresponds to the geostatic effective horizontal stress in the surrounding soil. This is calculated by multiplying the effective vertical stress by the (normally consolidated) at-rest earth pressure coefficient, K_0^{NC}, as defined by Jaky's formula, see Equation 2.14.

To ensure consistency and comparability between the analytical and numerical calculations, key parameters including the friction angle, unit weight, Poisson's ratio, and elastic modulus were maintained identical across both methods.

Since the SS constitutive model was used in Plaxis and this does not directly have an input for elasticity modulus, it had to be backwards calculated from the modified swelling index, κ^*, to be used for the analytical cavity expansion calculations. This can be achieved using the relationship between the bulk modulus, K, and the elasticity modulus seen in Equation 3.1.

$$K = \frac{E}{3(1+2v)} \tag{3.1}$$

The bulk modulus can be derived by using Equation 3.2. p' is the mean effective stress, calculated with Equation 3.3 (assuming $\sigma'_2 = \sigma'_3$). For $z = 0$, resulting in a mean effective stress of zero, a minimum value equal to a unit stress was implemented to mimic the behavior of Plaxis.

$$K = \frac{dp'}{d\varepsilon} = (1+e)\frac{dp'}{de} = \frac{(1+e)p'}{\kappa} = \frac{p'}{\kappa^*} \tag{3.2}$$

$$p' = \frac{1}{3}(\sigma'_1 + 2\sigma'_3) \tag{3.3}$$

Equations 3.1, 3.2, 3.3 then give the expression in Equation 3.4 for the elasticity modulus. A changing elasticity modulus with depth is achieved due to the mean effective stress increasing with depth.

$$E = \frac{3(1+v)p'}{2\kappa^*} \tag{3.4}$$

By splitting the pile and subsequent cavity expansion into sections, the variation of soil parameters with depth could be captured with acceptable accuracy. Further, this allows for the calculation of the volume increase of the pile cavity in a simplistic manner, as the cavity expansion is assumed to be axisymmetric. Calculating the cylindrical volume of the original pile, with radius R_{pile}, and the sum of all the subsection volumes with the expanded radius $\delta_{r,i}+R_{pile}$ provides enough information to calculate the change in pile volume caused by the casting pressure, see Equation 3.5.

$$V_{cavity} = \sum[(\delta_{r,i} + R_{pile})^2(z_i - z_{i-1})\pi] - V_{pile} \tag{3.5}$$

Where
$\delta_{r,i}$ is the lateral displacement for the current subsection [m]
R_{pile} is the original pile radius [m]
z_i is the depth of the current subsection [m]
z_{i-1} is the depth of the previous subsection [m]
V_{pile} is the theoretical pile volume ($= R_{pile}^2 L_{pile}\pi$) [m^3]

3.3.3.2 Shallow Strain Path Method

When the soil is pushed away by the expansion of the pile shaft it will advance upwards to surface, heaving it. Since only the volume of the expanded cavity will cause soil displacements and not the entire pile shaft, a situation similar to that of the open ended tube occurs, where the cavity can be seen as a thin wall around the original pile body. As such, Equation 2.10 can be utilized to estimate the maximum heave directly above the pile. The maximum heave can be attenuated in a similar manner as to the full displacement piles by implementing an attenuation factor based on the radial distance, r, from the pile center and pile length, L_{pile}, which is then multiplied by the maximum heave, see Equation 3.6.

$$Attenuation\ factor = \frac{1}{r} - \frac{1}{\sqrt{r^2 + L_{pile}^2}} \tag{3.6}$$

Since a constant thickness of the wall is needed for Equation 2.10, the thickness $2w$ is set to a radius based on the total volume of the expanded cavity, R_{cavity}. This radius can be calculated using Equation 3.7.

$$R_{cavity} = \sqrt{\frac{(V_{pile} + V_{cavity})}{L_{pile}\pi}} - R_{pile} \tag{3.7}$$

Once the heave and its distribution over the surface has been calculated the volume of heaved soil can be calculated using Equation 3.8, where the volume is calculated in segments and summed together.

$$V_{heaved} = \sum \delta_{v,i}\pi(r_i^2 - r_{i-1}^2) \tag{3.8}$$

Where
$\delta_{v,i}$ is the segments vertical displacement [m]
r_i the segments radial distance [m]
r_{i-1} the previous segments radial distance [m]

As any SSPM estimated heave will never fully become zero, no matter the distance away from the pile evaluated, a maximum distance had to be established. In theory the volume of the soil pushed away should match the volume of soil that is heaved. This served as a reference point to calibrate the radial distance away from the pile to study. By matching the volume of heaved soil to the volume of the expanded cavity a radial distance of 64 meters (for a pile with $L_{pile} = 60$ m) could be found as an appropriate stopping distance considering this volumetric relationship. Note that this closely matches the 1:1 relationship that was suggested with the Hellman/Rehnman method, see Section 2.4.1, and tests show that this relationship holds even for different pile lengths.

3.3.3.3 Volume Control

To ensure an accurate simulation of excessive washout, a two-step approach was utilized. Firstly, the volumetric strain applied to the soil cluster was calibrated to replicate the volume loss of ten times the theoretical pile volume. This was achieved using Equation 3.9. The solution of volumetric strain was obtained using the Goal Seek function in Excel.

$$10 \cdot V_{pile} = \varepsilon_V \cdot V_{cluster} \tag{3.9}$$

Where
V_{pile} is the theoretical pile volume [m^3]
ε_V is the volumetric strain [%]
$V_{cluster}$ is the volume of the soil cluster [m^3]

Secondly, the volume loss applied in the Plaxis model was verified by integrating the produced settlement cruve. This was done by using the shell method of integration, revolved around the axis of symmetry through discrete approximation. The calculation was performed in Excel, through iteration of each volume segment using Equation 3.10.

$$V_i = 2\pi \cdot \bar{\delta}_{r,i} \cdot \bar{\delta}_{v,i} \cdot \Delta\delta_{r,i} \tag{3.10}$$

Where
$\bar{\delta}_{r,i}$ is the average radial distance of segment i [m]
$\bar{\delta}_{v,i}$ is the average vertical displacement for segment i [m]
$\Delta\delta_{r,i}$ is the width of the segment in radial direction [m]
V_i is the rotational volume of a segment [m^3]

The total volume loss was then obtained by summing all the segment values, which was later compared against the simulated volume loss. The first and last data points were excluded from the iteration to ensure a valid segment pairing.

3.3.3.4 Pile Group Calculations

Based on the superposition principle, a Matlab script was developed to extend the analysis from a single pile to a group of piles. Using the radial distances and corresponding vertical displacements computed for a single pile, multiple identical piles were distributed over a user-defined piling area. The total displacement is then obtained by summing the contributions of all piles.

To achieve this, the Pythagorean theorem was used to calculate the distance from each point within a predefined grid to a pile (i.e. the grid center). These distances allowed for interpolation of displacement values into a matrix based on the data from a single pile. A user-specified center-to-center spacing enabled the script to automatically populate the piling area with the maximum number of possible piles.

For each pile location the displacement matrix was offset accordingly, and all individual contributions were summed into a new matrix, representing the total displacement across the area grid. Finally, the grid and resulting displacement field were visualized using a surface plot (top-down view) and cross-sectional plots through the center of the grid.

3.3.4 Parameter Study

To better understand the influence of different parameters on the outcome of the Plaxis simulations, two separate parameter studies were carried out. The first focused on geometric parameters, and the second on soil properties. Using the base case as a reference, a selection of key parameters were changed to a smaller and larger value, relative to the original, to assess their influence on the simulation results. To explore patterns and enable better comparisons between parameter sets, the data was normalized. Data were normalized with respect to the parameters that were found to be the most suitable for each particular assessment.

Parameter variations included a lower bound of $250c_u$ and an upper bound of $500c_u$, representing high and low plasticity clay respectively. For cast-in-place pile simulations, the K_0 and K_0^{NC} for the LE and SS materials were increased and decreased by 0.1 to see how the ratio of vertical and horizontal stresses affects the results. K_0^{NC} is used in the SS model instead of K_0 since they are interpreted in the same manner between the two models using Jaky's formula, see Equation 2.14. This comparison is valid since LE does not consider the effects of overconsolidation, instead the K_0-value for an overconsolidated soil is calculated using a different Eurocode formula. The variation of the overconsolidation ratio in the clay enables the observation of how the stress history of the clay changes the results. OCR is not an input for the LE material model so this particular parameter had to be omitted from the LE model, but was tested using SS. Table 3.8 lists all the parameter variations for the clay layer and the respective material model.

The geometry study for both the drilled steel tube pile and the cast-in-place pile investigated the effect of different pile lengths. Both piles were modeled as 60 meters long for the base case, with a shorter 30 meter pile and longer 90 meter pile as variations.

The effects of the cast-in-place pile radius were also explored. From the base radius of 1 meter, additional 0.5 meter and 1.5 meter radii piles were simulated. For the drilled steel tube pile, different geometries of the soil cluster subjected to volume loss were compared. The specific soil cluster variations tested are presented in Table 5.1 in Chapter 5.

Table 3.8: Clay parameter variations for the parameter study along with their associated model.

Parameter	**-**	**Base**	**+**	**Model**
$E - c_u$ factor	250	375	500	LE/SS
K_0	0.4	0.5	0.6	LE
K_0^{NC}	0.4	0.5	0.6	SS
OCR	1.0	1.2	1.4	SS

4

Interviews

This chapter presents the results of the conducted interview study. The results are structured according to the key themes outlined in the interview questions. The answers reflect industry perspectives and insights on drilled steel tube piles. Quotes originally in Swedish have been freely translated by the authors, aiming to keep the original tone of the interviewees.

4.1 When and Why Are Drilled Steel Tube Piles Used?

Considering all factors, drilled steel tube piles are rarely selected as the first choice for piling contractors whom also offer driven piles. Conventional driven concrete piles are more cost-effective to install, both in terms of production time and material cost (SEK per metric ton and meter), compared to drilled piles. Based on these factors drilled piles are often seen as a last resort. Although, from experiences shared by multiple interviewees, they sometimes ends up being required to finish the project. Factors highlighted by the interviewees that favor the usage of drilled steel tube piles include the following:

- boulder rich soils that can cause premature pile refusal when driving piles,
- the pile must be embedded in bedrock,
- if old or otherwise sensitive structures are located nearby and soil movements need to be minimized.

4.1.1 Soil Conditions

If challenging soil conditions are experienced during production, or sometimes anticipated in advance based on geotechnical reports, the piling contractors are adamant about notifying the client to prevent issues they can foreseen based on experience. The occurrence of boulders or other formations scattered within the soil is a typical problem for driven piles, causing premature refusal or lost (damaged) piles. Refused and lost piles are taken into account in the tender calculations, with approximately 10% of the piles expected to be refused during installation. In such a case, the refused or lost pile is left in place, and an additional one is installed adjacent to it. If preliminary geotechnical reports do not accurately predict the amount of boulders in the soil, the 10% estimate can quickly be surpassed. Drilled piles on the other hand does not have this issue, as they can pass through the boulders with ease, ensuring that bedrock is reached.

If the pile needs to be embedded into bedrock at significant soil depths, drilled piles are preferable due to the longer achievable lengths compared to driven concrete piles. The current practices of deriving soil profiles within an area from just a few soundings are also pointed out by multiple interviewees as an obstacle. Depths down to bedrock are interpolated from these few points, meaning that the interpolation can be highly inaccurate. Furthermore, what seemed like a good piling choice on paper, e.g. driven piles, can turn out to be bad if the soundings were done at “lucky” points, completely missing large boulders existing in the soil.
In the Gothenburg area the clay is reasonably homogeneous and boulder-free, making driven concrete piles the preferred choice, but this can be a sizable problem in other parts of Sweden.

4.1.2 Types of Projects

A common situation for the use of drilled steel tube piles is improvements to existing foundations of old buildings. As the available space within buildings is constrained, such as the cellar, it often favors the relatively smaller size of drilling rigs compared to pile driving rigs, along with lower sound emissions and vibrations. Another important factor is that drilled piles cause less soil movements according to the interviewees, if installed correctly. Old buildings often have other sensitive foundations nearby that cannot withstand the soil displacements caused by driven piles. This aspect is also relevant for major infrastructure, such as bridges. Construction close to these structures adheres to strict values of allowable soil movement, meaning that drilled piles will typically be used to keep within the set limits. Multiple interviewees highlight that drilled piles are more commonly used for infrastructure projects or larger buildings, and that these projects usually have higher budgets for the design and installation of foundations.

4.1.3 Pile Dimensions and Demand

During recent years, the demand for drilled steel tube piles in the Gothenburg area has decreased according to most interviewees. Overall, this is believed to be linked to the downturn of the construction sector as a whole. Nevertheless, drilling is becoming more prominent on the Swedish market. The pile sizes are also moving towards larger dimensions. 320-400 mm diameter piles are seen as the standard today, but 600-1200 mm have been used. Typically small diameters are used for foundations of houses, while large dimensions are used for infrastructure projects or larger buildings. Some interviewees felt that Sweden is trailing behind the rest of Europe when it comes to drilled piles, adopting larger piles just within the last few years and not implementing techniques such as reverse circulation across the board, a practice which is more common in continental Europe.

> *"Culture plays into it, a common mentality is that ‘it’s fine as it is’."*

An interviewee stated that they would rather drill more small-diameter piles, compared to fewer larger piles, with the same combined bearing capacity. Owing to the additional logistic problems that large piles entail. More expensive pilots and hammers, a larger drill rod, and space requirements are all cited as negatives by increasing the diameter of the pile. Driving the drilled steel tube to refusal is done after typically 0.5 meters or 3 times the pile diameter has been drilled into the bedrock. This is done to ensure proper contact with the bedrock is achieved. A point discerned by the interviewees was that driving the piles to refusal becomes more cumbersome as the pile dimensions increase. The heavier weights being required to drive the pile means bigger and taller machines are needed to perform the procedure, taking up space and being more difficult to handle.

> *"I would rather drill two smaller piles than one big pile."*

4.1.4 Summary and Reflection

Drilled steel tube piles are favorable when soil conditions are difficult, mainly due to boulders in the soil, or when embedment into bedrock is needed. Swedish geology, and the Gothenburg area in particular, suit the use case of driven concrete piles very well. As such there is no surprise that they are favored compared to drilled piles. Another factor is the cost of driven concrete piles is about one third the cost of a drilled steel tube pile. One thought as to why is that the major piling contractors have their own pile factories, reducing the prices to the point where companies without factories have trouble competing on bids for projects. So even if premature refusal or lost piles are accounted for, the total cost will still be significantly lower than drilled piles.

Perhaps the beneficial conditions and supply chain investments for driven concrete piles have hampered the rate of implementation of drilled steel tube piles in Sweden. Why would the large piling contractors spend money on improved drilling techniques, when their main source of income is driven piles? Unless the cost aspect is succeeded by another factor, such as soil displacements, they have no major motivation to change their business approach.

4.2 Drilling Methods

Common among all respondents is that DTH drilling is the most widely used technique, applied both in pneumatic and hydraulic drilling operations. The preference for the DTH method is motivated by its ease of use, cost efficiency, driving force, reliability, and the companies are accustomed to using the method. Other methods, such as the top-hammer, are more prevalent for the installation of anchors or smaller pile diameters.

When using DTH hammers, concentric drilling is the most commonly adopted technique, motivated by similar advantages, namely reliability, precision, and ease of use. This is particularly advantageous due to the possibility of sourcing the pile with a drill bit already attached to the casing. Eccentric systems, on the other hand, are also mentioned but are more frequently used in the installation of RD-pile walls, often with a wing-drive system. Usage of such a system for standard steel tube pile installations is less common. Based on the respondents experiences, these systems are associated with higher risks of becoming damaged, particularly when encountering obstacles in the soil, such as boulders.

4.2.1 Choice of Drill Bit

The selection of drill bits and pilots varies depending on the properties of the soil at the site. In the current market, it was evident that the supply of different types and manufactures has expanded, and as described by one respondent:

> *"Different manufacturers have different systems, it is a jungle..."*

However, a more conventional drill bit is often selected, unless otherwise outlined in the tender documents. In sensitive environments, one respondent acknowledges that for pneumatic drilling it is important to use a drill bit that directs flow inward, thus minimizing the risk of air traveling along the outside of the casing. This was also seen to be advantageous in maintaining pressure during drilling and thus providing a sufficient air flow velocity to allow transport of cuttings. In contrast, another respondent's view on more advanced drill bits, which promises improved fluid control, was that they were merely a sales argument. If drilling is conducted in a careless manner, where mistakes are made and the return channels eventually become blocked, the same effect will occur regardless of the drill bit design.

One of the respondents described that in some projects, if time is available and its deemed potentially beneficial for the project outcome, a test field is performed. During the drilling of a test field, different drill bits are evaluated based on monitoring of the rate of penetration (ROP) and impact on surrounding soil.

In challenging and complex ground conditions or earthworks/foundation work, other adaptations can be made to drill bits or ring sets, which do not concern the blocking of return channels. One respondent described a current project where a ring set is reinforced with two impact edges, an alteration made to counteract the risk of losing a ring set during drilling. A risk that would otherwise involve delays in production.

4.2.2 RC-systems

Of the interviewees, only two had direct experience with the use of RC-systems in Sweden, while others had evaluated its potential use, but ultimately decided not to invest. This was primarily due to the high investment cost, however, a common continued interest remained and future implementation was not ruled out.

Among the respondents with experience, several common viewpoints were shared. The primary advantage highlighted was the improved handling of the cuttings, made possible by the double-walled drill rod and hammer, which provides improved return flow control. This was seen as particularly advantageous in constrained work site conditions, where space is limited or if drilling is to be carried out in sensitive environments, where strict requirements are in place to avoid contamination of the soil. Additionally, RC-systems prove advantageous when drilling larger-diameter piles. Due to its enhanced control of cuttings, smaller drill-rods can be used, as the system operates independently of the annular space between the drill rod and the pile. This in turn enables the same size of drill rod to be utilized for the installation of different pile diameters, which is not possible with conventional DTH drilling. Apart from the high investment cost, one drawback found with RC is the risk of potential leakage through the so called "umbrella", see Figure 4.1. This results in a gathering of cuttings on top of the hammer, which requires time consuming measures to remove, as the material clogs the hammer and causes damage to the equipment.

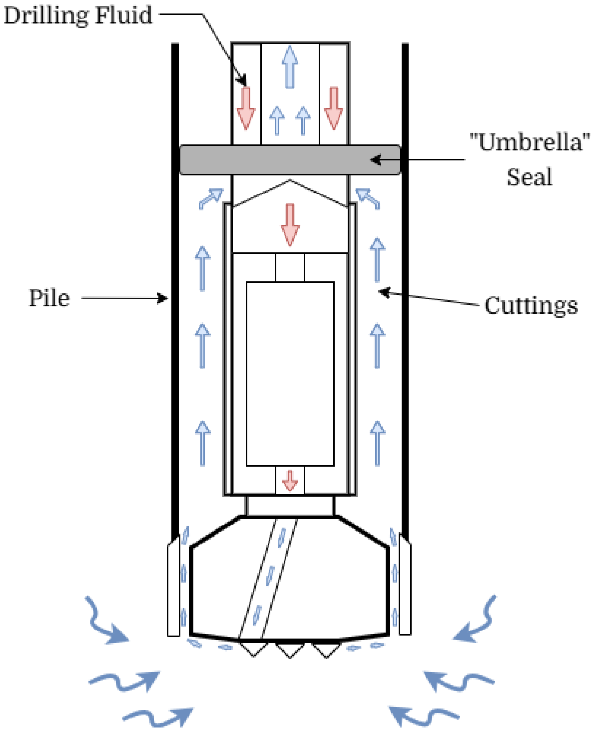


Figure 4.1: Simplified schematic of a RC-system, illustrating the flow path of cuttings and the functioning of the "umbrella" seal.

4.2.3 Summary and Reflection

In general, the gathered responses aligned well with the reviewed literature. However, what has emerged is that the availability of drill bits and pilots appears to have increased, both in terms of manufactures and the variety of types. Although careless drilling was established to result in problems, it is likely that new and improved equipment has decreased the probability that this will occur. Furthermore, viewpoints on RC-system prove valuable as little literature on the systems have previously been found, particularly in a Swedish context.

4.3 Pneumatic Drilling vs Hydraulic Drilling

Among all interviewees, there was a clear consensus that pneumatic drilling was the preferred technique in Gothenburg. The method was consistently described as the most cost-effective, easiest to use, and most familiar in the industry. Since being the favored choice, most of the available equipment and workforce have adapted to its usage, which contributes to its continued dominance in practice.

Hydraulic drilling on the other hand, often of Wassara type, was reported to only be used if it was prescribed in tender documents or similar, or if strict tolerances on allowable soil displacements are in place. If deviations are experienced during pneumatic drilling that have an impact on the surrounding soil, it could warrant a change to hydraulic drilling. However, this would require approval from the client if performed under a build contract, as it would incur increased costs.

Despite its limited use, most of the respondents had positive experiences with hydraulic drilling. In particular, the method was highlighted to provide overall good production rate, improved precision, and lower noise pollution. Despite this, reluctance towards the technique was due to its overall higher operating costs and practical challenges related to disposal of drill cuttings mixed with water. According to one respondent, the water use is approximately 800 liters per minute when drilling larger diameter piles, which means large volumes of water that must be collected, treated, and disposed in accordance with environmental regulations. All respondents agreed that this is both challenging and time-consuming, particularly in confined or complex worksites. The winter season in Gothenburg adds further complications as the water tends to freeze, increasing the logistical efforts required. Although the contractor typically performs the additional management associated with hydraulic drilling, it is often negotiated and accepted by the client to cover the additional costs related to water supply and sediment management.

In terms of environmental impact, hydraulic drilling was described as more energy efficient compared to pneumatic drilling, as less fuel is used to keep the pumps on as opposed to the air compressors. Meanwhile, concerns were raised by one respondent about his personal reluctance to use large volumes of potable water when drilling. It was considered strange to use clean drinking water for industrial purposes.

> *"It does feel quite unreasonable to consume 350 liters per minute of treated drinking water, especially when you hear about people not being allowed to water their plants."*

The same respondent recently participated in a project in which, for the first time in his experience, the water was sourced from the Göta river. Although this required the sourced water to be pretreated through special filters before usage, it worked very well according to the interviewee.

4.3.1 Summary and Reflection

In general, it is evident that the contractors are hesitant to use hydraulic drilling, despite most having positive experiences with the technique. From an economical viewpoint this seems reasonable, as the contractor strives to achieve high profit margins, this often entails a reduced willingness to take risks. Under current contract forms, hydraulic drilling appears to be associated with higher risks from the contractors side due to higher costs and the additional management, but also a lack of experience compared to pneumatic drilling.

Considering the environmental aspect, hydraulic drilling is often referred to as the more environmentally friendly option because the drilling equipment uses less energy compared to pneumatic drilling. However, the cost and environmental impact of using potable water, particularly the larger volumes needed for bigger piles often appears to be neglected. This impact could potentially be avoided by sourcing water from natural water bodies, but this would also result in other environmental aspects to consider, such as the effects it would have on the groundwater table. However, considering the ongoing and future development plans for Gothenburg this change in execution could potentially be beneficial.

4.4 Control During Installation

Throughout the interviews, a shared understanding of the importance an experienced and skilled drill rig operator brings with them is crucial to the installation process. Some key parameters to control when drilling were identified, but most expressed was the ROP along with the amount and type of material in the drill cuttings. A diminished ROP, while still actively drilling, can happen while drill cuttings are still coming up in "normal" amounts, this is indicated as a telltale sign that a washout has occurred. Controlling the amount of cuttings is a difficult task according to the interviewees. When drilling with air the cuttings are typically allowed to spew haphazardly from the rig and are then collected by a front loader off the ground. If the situation and location call for better control of the cuttings, such as when drilling close to trafficked roads, in urban areas or windy weather, a "potty" and cuttings hose can be used to gather and collect it. The potty and cuttings hose are attached above the drill rod allowing the cuttings to impact the potty at full force and then be led out through the hose in a controlled manner. However, utilizing these attachments can lead to a blowout, where pressure is built up inside the drill rod, and cause an uncontrolled ejection of material. This risk is considered too great to warrant the use of a cutting hose and potty at all times by multiple interviewees as the production loss would be severe.

4.4.1 Volume Control of Drill Cuttings

Experiences from measuring drill cuttings typically involved using a container with a known volume and then simply performing a quick ocular inspection to estimate the total volume of cuttings. Complications in this scenario arise from obvious factors, e.g. when a front loader is needed to collect the cuttings, as another pile cannot be installed in the vicinity as the cuttings would mix, leading to a production standstill to measure the cutting. Compared to pneumatic drilling, hydraulic drilling allows the return stream of cuttings to gently boil over the edge of the pile. The downside of this is that the soil will be mixed entirely with water, leading to a more difficult identification of the soil type. Sedimentation tanks are thereby mandatory to allow the used water to be separated from the soil particles before releasing it back into the system. One contractor used an additional shaker and centrifuge to separate rock and clay from the water during a project. The need for sedimentation tanks and various other processes to treat the water used for drilling requires space, which in urban environments is limited. One interviewee was involved in a project on a constricted site where water drilling was prescribed due to sensitive buildings surrounding the site. Eventually they were allowed to drill with air as the amount of water to handle became too cumbersome. While not all interviewees had personal experience with RC-drilling they all saw benefits with it in terms of handling the cuttings as covered previously.

Nevertheless, whether it is air or water drilling, the perception of what is considered a normal or expected amount of cuttings differs. One interviewees experience is that two times the pile volume is an "okay" amount, while another interviewee would not expect large deviations from the total theoretical pile volume. One contractor uses three times the volume for calculation purposes related to the handling of cuttings.

As previously highlighted, an experienced operator is crucial. When drilling they are solely responsible for the procedure. They need to control the ROP, what material is coming up and how much, that the right amount of air or water that is used, and when solid rock penetration has been achieved. Perhaps the most important aspect is that drilling is halted immediately if an unexpected stop in the ROP occurs, linked by many to result in a washout. In boulder rich soils there is a risk that the pilot only partially contacts a boulder, with the rest of the pilot exposed to a softer soil. In such cases an increased risk of a washout is expected as additional soft soil will be flushed away due to the differential drilling resistance between the boulder and soil.

> *"It's up to the drill rig operator mostly. Naturally it's not good to get to much cuttings, but it's also not good to get no cuttings. Almost everything is critical if you do not handle it correctly."*

4.4.2 Site Monitoring

For construction sites with nearby sensitive structures careful monitoring of adjacent structures is often prescribed. If displacements due to the pile installation occurs, it is important that this is detected immediately. In the case of significant effects, the contractors must halt the work and contact the client to discuss the next step. While the client is solely responsible for the design and methods used in non-turnkey projects, piling contractors typically offer their expertise in suggesting appropriate measures to adapt the work for the specific site, indicated by multiple interviewees as a form of good will towards the client.

4.4.3 Summary and Reflection

The ROP is seen as the most important parameter for a drill rig operator to control to prevent a washout effect to occur. However, the interviewees see that a control value of the ROP is not possible to implement, despite the fact that it governs the soil displacements, as the steel tube is essentially driven and drilled in soft clay. A ROP of about 6 meters per minute has been experienced at shallow depths, much greater then what Baardvik et al. (2016) recommended at 0.5-1 meter/minute to prevent soil displacements. Another important factor is the continuous stream of cuttings through the drill string. However, care should be taken if the ROP diminishes while the cuttings continue to flow as expected, as this is a tell-tale sign that excessive washout is occurring.

Controlling whetter a washout has occurred can usually never be done based on the volume of cuttings. Not much care is typically taken in collecting and measuring the cuttings, especially if hydraulic drilling is used. The amount of steps to properly control the amount and what material is drilled up is simply to cumbersome to perform if not prescribed. Perceptions about what the normal amount of cuttings that should be collected differ widely. Some say that there should not be an excess of material, while another respondent says that twice the pile volume is fine. It is likely that personal experiences play a major role in the different responses given to this question. Making statements relating the cuttings volume to the pile volume can be unwise as the type of soil is also a likely factor. In a till layer, the risk of washout is higher, as the groundwater flows easier through it, while clay is less prone to this effect.

While measuring the cuttings is typically costly and unfeasible, monitoring of adjacent structures and the worksite is often used and allows for swift intervention if significant soil displacements occur. When this happens the contractor should halt all work in the area and contact the client to discuss potential next steps. While the design of the project might be in the clients hand, the piling contractors offer their expertise to help resolve issues. As the piling contractors are experts in their area, attention should be paid to their thoughts on how a project should proceed, realistically allowing for better and cheaper solutions in the end.

4.5 Deviations

Several interviewees reflected upon common deviations experienced during the installation of drilled steel tube piles. The most frequent, was the occurrence of boulders or other formations in the soil, which can impact both the position and the inclination of the pile. From a project in Gothenburg, it was reported that old timber piles were encountered in the soil, which increased the wear on the dril-bit. Another commonly reported deviation was the discrepancy between the actual and documented bedrock level, often resulting in on-site adjustments.

In terms of the deviations considered the most severe, a range of critical issues were discussed. These have been structured in the following subsections based on their influence on the drilling progress and impact on the surroundings.

4.5.1 Impact on the Drilling Process

Several aspects were considered to have a strong impact on the progress of pile drilling works, these include the risk of obstacles, insufficient hammer performance due to wear and tear, and blowouts of drill cuttings during operation, which posed a risk to ensure a safe work environment both on site and in the surrounding areas, such as bordering sidewalks in urban projects. It is evident that views varied considerably among respondents about which deviations were deemed the most critical. This also holds true for the impact on surrounding soil conditions, suggesting that site conditions and context strongly influence the view of risk perception.

4.5.2 Impact on the Surroundings

Considering deviations that have an impact on surrounding buildings and soil, the views varied depending on the conditions at site and design of the project. However, several respondents emphasized the importance of closely monitoring the return flow, particularly in pneumatic drilling, as irregularities in material withdrawal often act as an indicator of severe installation effects in surrounding soil.

Out of the interviewees, all but one reported having experienced projects in which installation effects had caused a severe impact on surrounding soil and structures. Although most of these projects had been carried out outside the Gothenburg region, at least one critical case was reported in the area. Of the cases discussed, both pneumatic and hydraulic drilling operations have been associated with severe deviations.

4.5.2.1 Severe Deviations in Pneumatic Drilling

In the cases of pneumatic drilling, the common problem is an excessive washout of soil. Of the reported events, they have occurred exclusively at large depths where sandy and silty soils have been present. Some of the interviewees indicated that it probably occurs because of the induced pressure difference in the soil, which creates an "ejector effect", particularly prominent if the ROP drops during drilling. In one of the cases, this effect was especially prominent when drilling was initialized below the groundwater level, i.e., under artesian conditions. However, in another case, an interviewee instead experienced a reverse effect, where surface heave occurred. This was caused by the return channels at the drill interface becoming blocked, which happened when the drill bit reached a gravel layer located below a thick clay deposit. This allowed pressurized air to escape through the permeable material and lift the overlying clay as a result of the increased resistance at the return channels. Similar effects have been observed when drilling near recently installed piles, as pressurized air may escape through neighboring piles offering a lower resistance path. This is sometimes noticeable as bubbling on the surface. However, these occurrences have not been associated with the same uplifting effects. Furthermore, one of the respondents highlights the difficulty of finding the most suitable ROP. It is a common perception to reduce the penetration rate when entering a permeable soil, however if the ROP is too low, there is a high risk of the return channels becoming blocked.

4.5.2.2 Severe Deviations in Hydraulic Drilling

For hydraulic drilling, two different types of severe effects were reported, mass displacement and soil erosion. The latter was experienced in two separate cases and was characterized by an insufficient return flow of the flushing medium and cuttings. Instead the water infiltrated the soil at great depths and little to no material return was achieved. This resulted in large settlements in the surrounding soil and structures. Based on the thoughts of the respondents, the believed trigger factor for this was the erosion of fine soils at larger depths. In one of the named cases, the soil profile was characterized by mostly sand and silt.

Mass displacements, on the other hand, were observed in a project in Gothenburg, where vertical movements were recorded at adjacent foundations, exceeding the control limits. This was caused by an excessively high ROP while drilling in clay, which led to an insufficient return of the cuttings. However, in contrast to previously reported cases of settlements related to drilling, this event resulted in surface heave.

This had not been considered during the design phase of the project. In order to continue drilling within the allowable limits, adaptations were required. Instead, the first section element of each pile was pushed into the soil and then recovered to ensure sufficient extraction of soil. Once performed, drilling proceeded as usual.

4.5.3 Summary and Reflection

Established from the interviews were that drilled steel tube piles can cause a severe impact on surrounding soil and structures. This holds true for both pneumatic and hydraulic drilling operations, and the critical factors appear to be the ROP and soil conditions on site. The responses also revealed experiences with imposed settlements due to hydraulic drilling, which had not been acknowledged previously in the addressed literature. However, the described cause of this deviation is questionable, as settlements generated usually stem from the removal of soil and not solely from erosion effects, since efficient withdrawal of cuttings was not achieved.

4.6 Tender Documents and Risk

This section will present the thoughts stated by the interviewees in regards to how tender documents are structured and the accuracy of their information. The topic of risk-distribution between the client and contractor will also be presented and discussed.

4.6.1 Project Prerequisites

A point commonly highlighted by the interviewees is that drilling a steel tube pile is "easy". What poses a challenge are the soil conditions, the depth to bedrock, or other specific conditions found at the site. Soil conditions are based on geotechnical site investigations, a process in which a few select soundings are used to interpolate the rest of the site. As discussed previously, this can severely misrepresent the conditions found at the site if boulders are not discovered. One respondent indicated that more data on the soil helps to judge what the best drilling technique is and what pilot would be the most suitable, among other aspects. This problem could most certainly be handled better with additional soundings, but these can not be performed at all drilling points, as expressed by one of the interviewees.

Another factor of annoyance among contractors lies in the tender documents, where multiple interviewees highlight control requirements they feel are "pointless". In build-contract projects, typically a structural engineer hired by the client decides the pile design, method choice and the required controls. Multiple interviewees believe these designers "copy and paste" control requirements and methods between different projects, without understanding what they are requesting by their inclusion. Some have even experienced requirements that are only relevant for another type of pile and not for drilled steel tube piles. Reports from the Swedish Commission on Pile Research are seen as a driving force when it comes to what is included in the control plans. References to sections in these reports can be included in tender documents, but the referenced section can state, as one interviewee put it, "there are 4 different ways to inspect in this situation and it is up to the designer to decide", further indicating that designers do not read what they include in the tender. Other incidents that were shared during the interviews included, for instance, that in one case the requirements on slenderness were stricter than what the pile manufacturer could guarantee. Another was that the contractor had to check the straightness and roundness of the pile sections on site, but the piles were delivered directly from the factory where they perform the same test with even stricter tolerances. However, the contractors can include the costs associated with these additional controls in their bid, but some interviewees see this as an unnecessary cost in both time and money for the client and themselves.

Multiple interviewees indicated that they often see potential problems based on the information found in tender documents before submitting bids for projects. Either information needed to make correct calculations on the project are missing, or that the prescribed drilling method is problematic based on the site conditions, both of which lead to higher costs. When approaching the client and the structural engineer with these viewpoints, they are sometimes met with a lack of understanding according to some interviewees.

> *"We had a lot of discussions before we were even there drilling. I asked the question 'Should we really do it like that? Should we not do it like this instead?'. In reply I got 'No, we are doing this and that because so and so reasons'. At that point I gave up - 'You will have problems and we will have problems.'"*

> *"I think it's hard for the designers when a pile contractor comes and tells them what they have done wrong. They probably have issue taking that to heart. In this project the client eventually listened to us instead of the structural engineer and allowed for another method to be used."*

4.6.2 Risk Taking

As highlighted, significant problems can arise during a project, even adjacent structures can be impacted. Who is responsible for these complications is not obvious based on the discussions during the interviews. Naturally, this depends on whether the project is a turnkey (design-and-build) contract or a build-contract. Yet some interviewees indicate that developers tend to assign blame towards the contractor when a prescribed method results in problems, even in the case of build-contract projects. One interviewee was involved in a project in which a nearby skyscraper, with a sensitive foundation, experienced problematic lateral loads due to soil movements introduced by the drilling work they were performing. Their opinion was that current practices where the foundation design in urban areas does not have to withstand any lateral loads are counterproductive. Currently, new foundations have to consider all adjacent buildings and pay if anything unfortunate happens, while the older adjacent buildings do not have to consider that new foundations might be built around it. Another interviewee refereed to the Swedish standard *General Conditions of Contract for Building and Civil Engineering Works and Building Services* (AB 04), stating that contractors are required to calculate their bids for the lowest cost (more on this later), based on the tender document. Further, stating that potential risks can not be included in these calculations and as such they should not be responsible for complications out of their immediate control.

> *"I see this often, that they are trying to put more and more responsibility on the contractor. They are trying to remove the risk of being a developer and that is not fair."*

4.6.3 What Can Be Done to Improve the Situation

Potential ways to improve the industry were discussed with the interviewees, and based on their own experiences from projects the responses varied. An interviewee wanted more knowledge to be shared across the industry. If a contractor experiences a particular problem in a project, the knowledge should be communicated to others in the industry to improve drilling practices. Another respondent wanted clients to stop considering water drilling as a catch-all solution to minimize installation effects. Clients should instead be more proactive in treating each site based on its own merits and choose a drilling method based on that, an opinion shared by multiple interviewees. One wished that guidelines about how much ones foundation should be able to withstand in terms of displacements and vibrations should be implemented. Such guidelines would also improve tender documents, as these guidelines could be included as requirements, improving the quality of work in the construction sector.

Aspects related to environmental impact due to pile installation were broached during the interview process. Steel tube piles are usually filled with concrete to prevent corrosion inside the pile, allowing structural capacity to be accounted for over the expected lifetime of the pile. Some interviewees also believe that the concrete prevents buckling, but this opinion was not shared by everyone. Typically, the concrete used is of low quality and a low price. Pilot projects have been tried, where only the top part of the pile was filled with concrete and the rest with gravel. A current pilot project is testing whether the pile even needs to be cemented, predicting that the clay and water within the pile will be enough to prevent corrosion and fulfill the structural capacity long-term. Perhaps more important is the water consumption aspect when drilling with a hydraulic hammer. As water scarcity is increasingly becoming an more important topic, some interviewees do not feel that it is correct to consume the volumes of potable water that is required when using a hydraulic hammer.

4.6.4 Summary and Reflections

Risk is arguably the most important factor in any construction project. One recurring theme from the interviews is that discussions about risk allocation often happen after a problem arises, rather than beforehand. Responsibilities are seldom clearly defined in advance, which leads to disputes between client and contractor once things go wrong. This reactive approach could be avoided if roles and risks were clearly established from the start. Early clarity would also make the bidding process fairer, as contractors would be able to assess and price risks with more accuracy before construction begins.

One contractor stated a common misconception: that under the Swedish standard AB 04, they cannot consider or calculate based on risks, only the lowest cost. However, this interpretation is incorrect. A precedent set by the Swedish Supreme Court in the *Gotland judgment* (NJA 2015 s. 3) clarified the interpretation of Chapter 1 § 8 in AB 04. It states that "in the case of a professional assessment, the contractor should consider circumstances within the work area that are the most likely." Therefore, the idea that contractors must always base their estimates on the lowest possible cost is false. Instead, they are expected to base their calculations on what is most likely to occur on site.

Of course, determining what is "most likely" can be difficult. However, this ambiguity also gives contractors the opportunity to make their own judgments, especially when the tender documents lack sufficient detail.

Another perspective raised is whether current contractual frameworks might be too restrictive. Structural engineers, whom often have limited geotechnical knowledge, decides the drilling method to be used in lieu of the piling contractors. Allowing contractors to choose their own drilling methods, and to assume the associated risks, could be a valuable first step. Contract models such as partnering contracts (Swe. *samverkansentreprenader*) or build contracts with construction responsibility (Swe. *utförandeentreprenader med konstruktionsansvar*) may enable more efficient and cost-effective solutions by better utilizing the contractors expertise.

In addition, having clear guidelines for allowable displacements and vibrations could serve as a helpful starting point for tenders. This would encourage the design of robust foundations and the use of appropriate pile installation methods. The concept of robustness has recently been introduced in the second generation of the Eurocodes. It emphasizes the importance of accounting for "what if" scenarios, such as climate change impacts (e.g. flooding, storms), structural failures, or increased loads. Although the proclivity for robust foundations is now established, what this means in practice still needs to be defined. The Swedish Commission on Pile Research could play a key role in developing relevant, practical guidelines for implementation.

5

Calculations

This chapter will present the results from the parameter study in Plaxis for both the cast-in-place pile and drilled steel tube pile. Additionally some analytical comparisons will be presented for the case of the cast-in-place pile. Shorter comments will also be included to explain what the figures show.

5.1 Cast-In-Place Pile

This section will start with a comparison of the analytical and numerical models deemed applicable for the cast-in-place pile, namely the cavity expansion method and the shallow strain path method. This is then followed by the geometric- and soil parameter study, using the finite element model for the cast-in-place pile outlined in Section 3.3.2.1.

5.1.1 Analytical and Numerical Comparisons

Results related to whether analytical solutions can provide good estimations to the problem at hand without adopting a more advanced and time consuming numerical model will be presented in this section. The results from the linear elastic CEM and subsequent SSPM calculations will be compared to the results from the baseline linear elastic and Soft soil finite element models (FEM).

Comparing the results in Figure 5.1 it is apparent that LE FEM and CEM give essentially the same results. The discrepancies seen towards the bottom are due to an arching effect between the stiffer bed rock boundary condition, till layer and the clay, effectively increasing the strength of the soil at the bottom. A similar arching effect could potentially been implemented for the analytical calculations as well, but the results nonetheless provide a good prediction of the behavior. In terms of volume, this particular case yields an increase of the total pile volume by 4.1% (7.7 m^3), close to the expected result between 6-7% additional volume.

The volume of only the expanded cavity was then calculated followed by the heave being analyzed using the SSPM approach explained in Chapter 3.3.3.2. Inspecting Figure 5.2 it becomes apparent that the results from LE FEM does not reflect the expected behavior. Within a distance of about 10 meters from the pile, the soil settles instead of the expected heaving effect. This issue was investigated further by creating a calculation phase in Plaxis where the pile soil volume was removed, e.g.

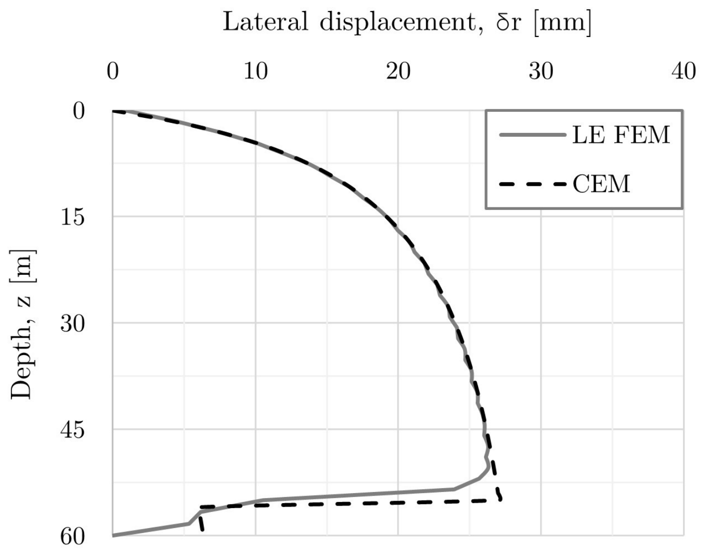


Figure 5.1: Comparison of lateral displacements according to cavity expansion theory and LE FEM.

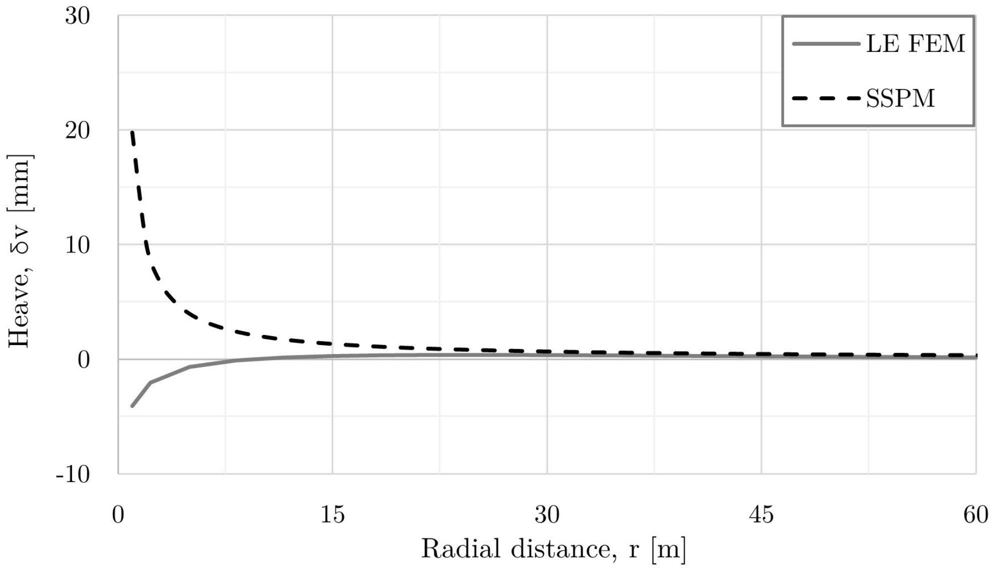


Figure 5.2: Comparison of heave according to SSPM and LE FEM.

excavated, but without any pressures applied to the cavity wall (including water pressure). The results of this test showed that while the soil wall collapsed inwards, as expected, the soil also heaved towards the axis of symmetry, as shown in Figure 5.3. These unexplainable results informed the idea that a more advanced constitutive model was warranted to fully investigate the problem. If instead SS is used the behavior of the soil conforms with the believed behavior, collapsing inwards and down.

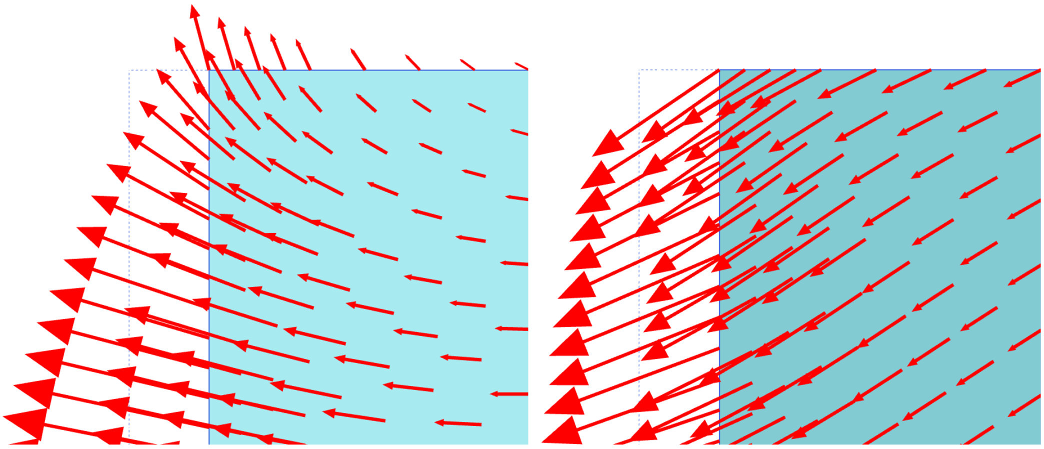


Figure 5.3: A screen grab from the LE (left) and SS (right) Plaxis models, showcasing how the soil moves when the soil volume element representing the pile is removed (indicated by the dashed outline) and no additional pressure is applied. The arrows indicate the soils movement direction.

When comparing the elastic and plastic (SS) models for the predicted lateral displacement the difference is substantial, with the plastic deformations being almost ten times as large, see Figure 5.4. The larger cavity expansion means that more soil is being displaced both laterally and vertically. A test using the baseline parameters for the pile and the SS model shows that the heave now behaves more as expected, see Figure 5.5. To better compare the heave from SSPM and SS FEM, as they represented different volumes, the SSPM curve was scaled by a factor of about 5.4 to match the volumes. This comparison allows better visualization of the predicted behavior, which from roughly 20 meters and further is very comparable.

The volume of the pile increases by 57% with the SS model based on the lateral displacements, which is out of the realm of realistic results (a volume increase of about 6-7% is expected). The model can however serve to give better insight of how the soil behaves in regards to, primarily, vertical displacements but also lateral displacements through the parameters study compared to the LE model. Despite this the validity of the results should still be viewed with skepticism when it comes to the absolute values predicted by the model.

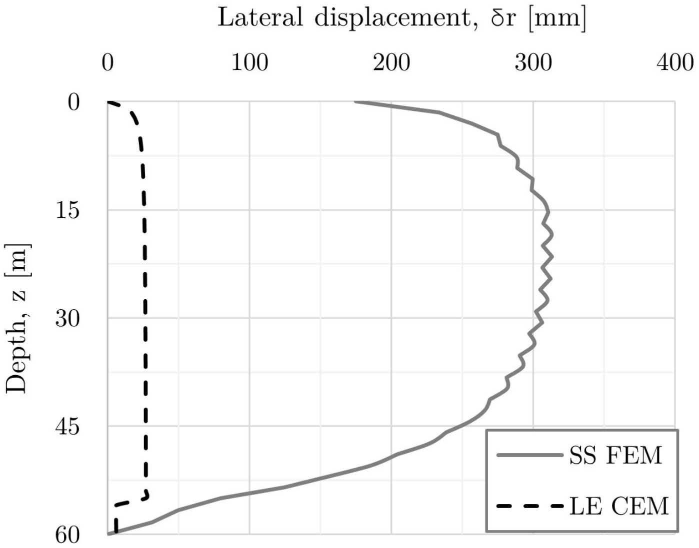


Figure 5.4: Comparison of the cavity expansion method (with κ^* derived elasticity) and SS FEM results.

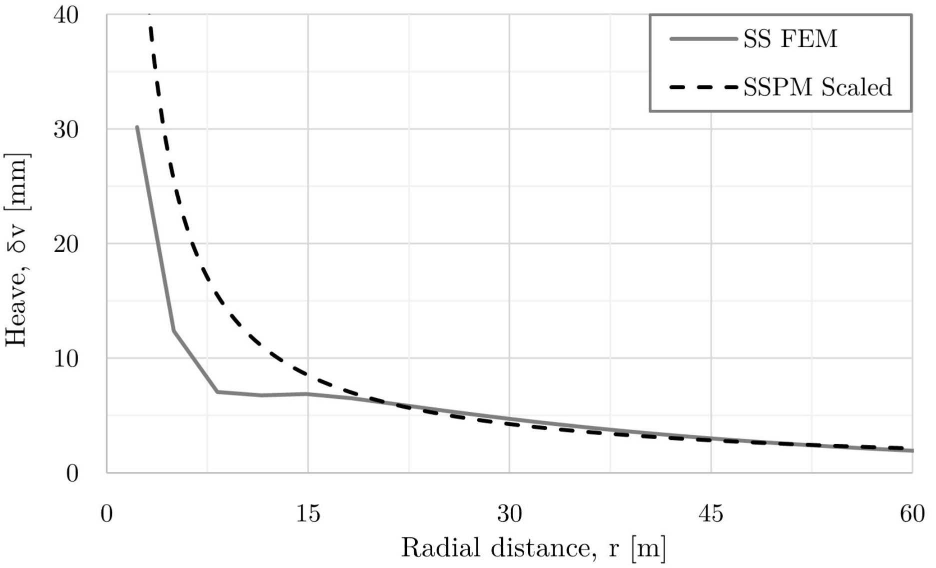


Figure 5.5: Comparison of heave calculated with SSPM and SS FEM. The SSPM curve has been scaled by a factor based on the volume difference of the original SSPM curve and the SS FEM curve.

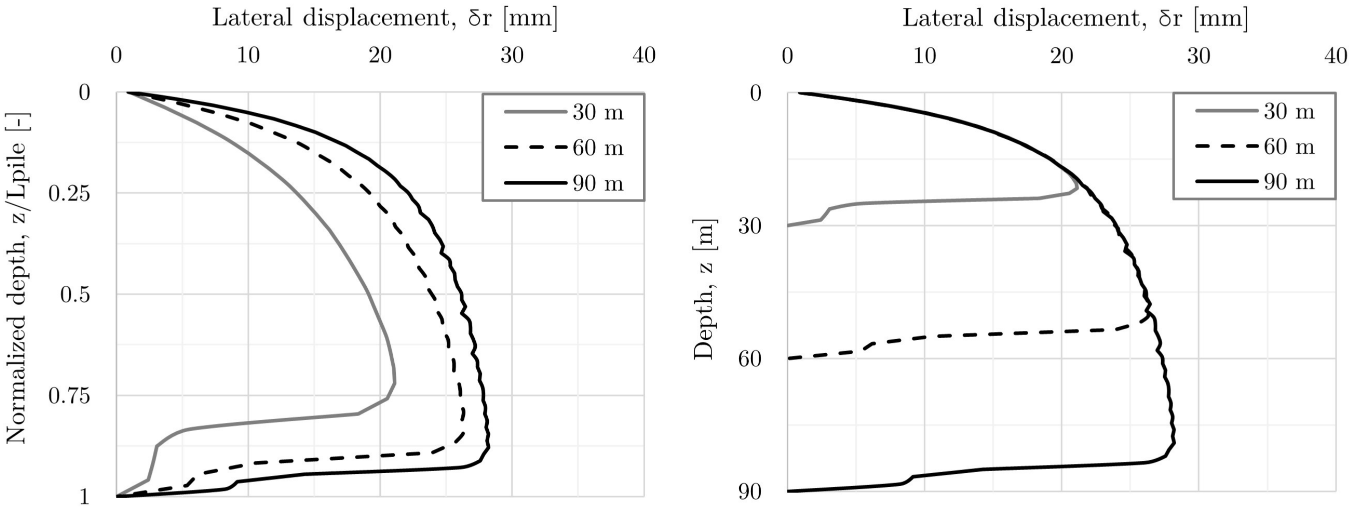
5.1.2 Parameter Study

This section contains results specific to the parameter study for the cast-in-place pile. The results will cover both the impact of different pile geometries and soil parameters on the lateral displacements and heave at the surface. For the geometric variations of the pile, both LE and SS material models will be presented. However, for the soil parameter variations only results using SS will be presented here. In most figures one axis have been normalized against the pile length to facilitate better comparisons between different figures.

5.1.2.1 Geometry

Variations of both pile length and radius have been investigated using both an LE and SS set of material models in Plaxis. For the LE comparisons only the lateral displacement will be presented here, as the heave has been been deemed to not be determined correctly. The heave results are of the LE model are instead presented in Appendix D.

Results of geometric parameters in the linear elastic model

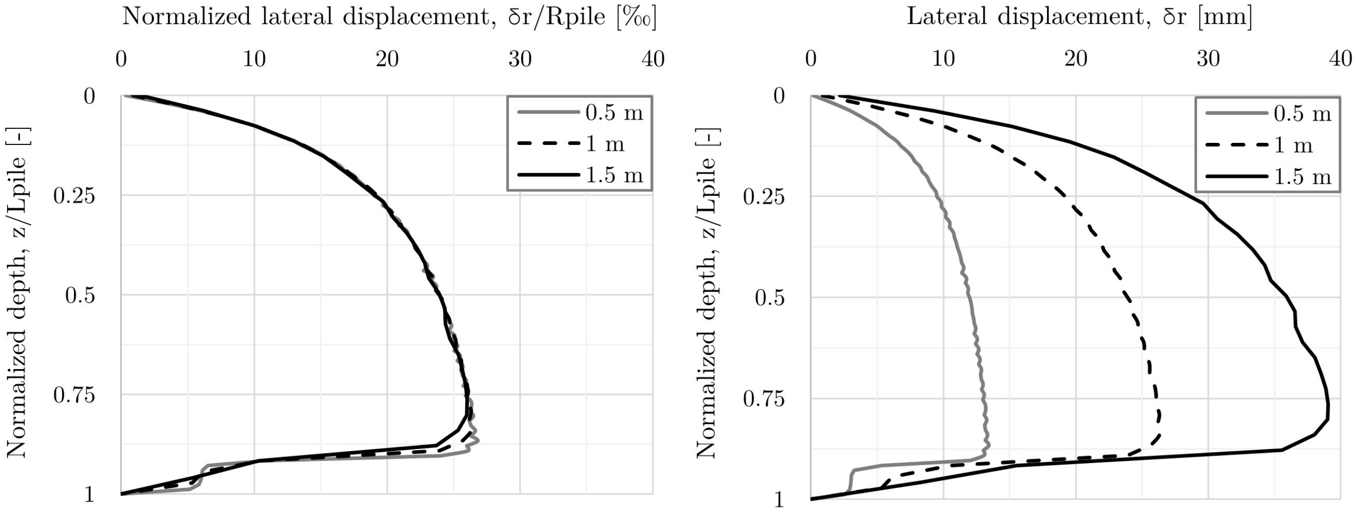


(a) Impact of pile length versus normalized depth on the lateral displacements in LE FEM.

(b) Non-normalized depth and lateral displacement for different pile lengths in LE FEM.

Figure 5.6: Left figure shows curves with depth normalized against pile length, the right figure shows the corresponding non-normalized curves.

In Figure 5.6a the results of the different pile lengths can be seen. One detail to mention is the fact that the till layer was kept at a constant thickness of 5 meters, rather than being dependent on the pile length. As a result, the arching effect induced by the stiffer till layer begins at a higher point for shorter piles. This simplification is apparent for the shorter 30 meter pile, where the curve peaks at a lower point than the other pile lengths. In fact, the lateral displacements of the three pile lengths mirror each other until the arching effect occurs for the respective pile length when not normalized, see Figure 5.6b. This conforms with the analytical solution, as the pile length does not affect either the difference in pressure or shear modulus for any given depth. Disregarding these details, the 30 meter pile does exhibit smaller lateral displacement, and the 90 meter pile displaces more compared to the 60 meter pile.



(a) Lateral displacement normalized with pile radius versus depth normalized with the pile length according to LE FEM.

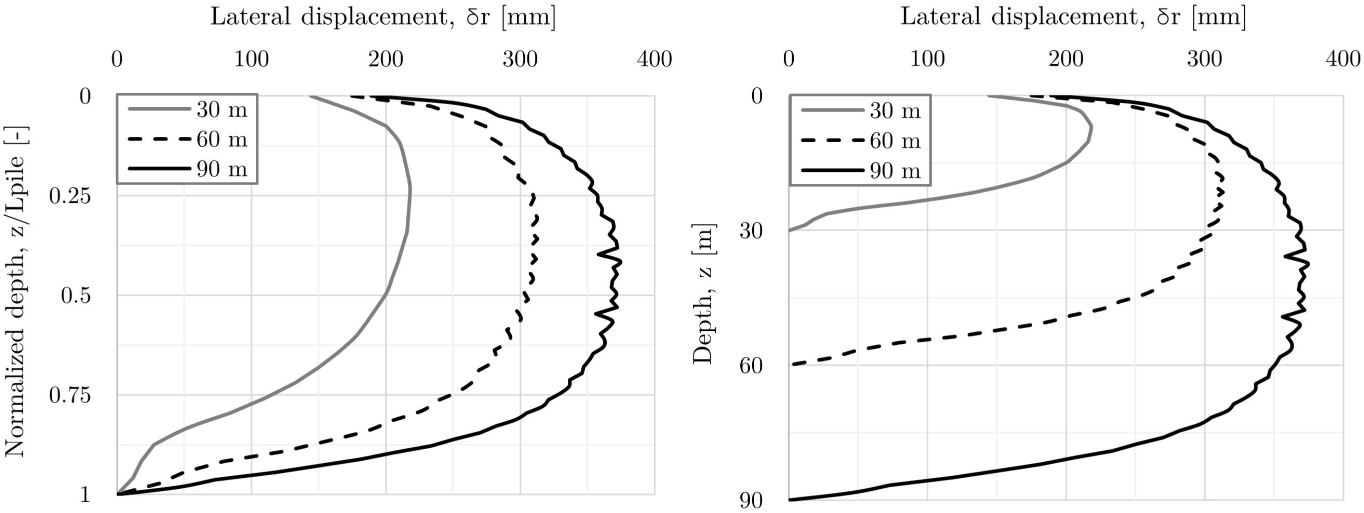
(b) Lateral displacement for different pile radii versus depth normalized with the pile length according to LE FEM.

Figure 5.7: Results for the impact of pile radius, R_{pile}, on the lateral displacement in LE FEM.

Figure 5.7a shows that the radius of the pile in an linear elastic finite element model has just negligible impact on the shape of the lateral displacement curve. In fact, this corresponds well to Equation 2.12, where the radius as a parameter is factored out when normalized against. However, it does impact the maximum displacement as seen in Figure 5.7b. This relationship follows the linear difference between the pile radii, with the 1.5 meter pile producing 1.5 times the displacement as the 1 meter pile, and likewise for the smaller pile radius.

Results of geometric parameters in the Soft soil model

In Figure 5.8 it can be seen that a shorter pile length results in a larger relative cavity. For the same comparison but in LE, the largest expansion is seen closer to the surface, whereas in LE this point is located at the bottom of the clay layer, disregarding the arching effect. This is a result of the way the conversion of the elasticity modulus has been performed between the $E - c_u$ factor and the κ^* value. These values have been matched at the bottom of the clay layer, but due to how the SS model calculates elasticity based on κ^*, the value at the top of the layer is roughly one-fifth of the LE model value, see Figure 5.9. The lower elasticity module therefore results in additional deformations, see Figure 5.9b, and will become more apparent in the soil parameter study.

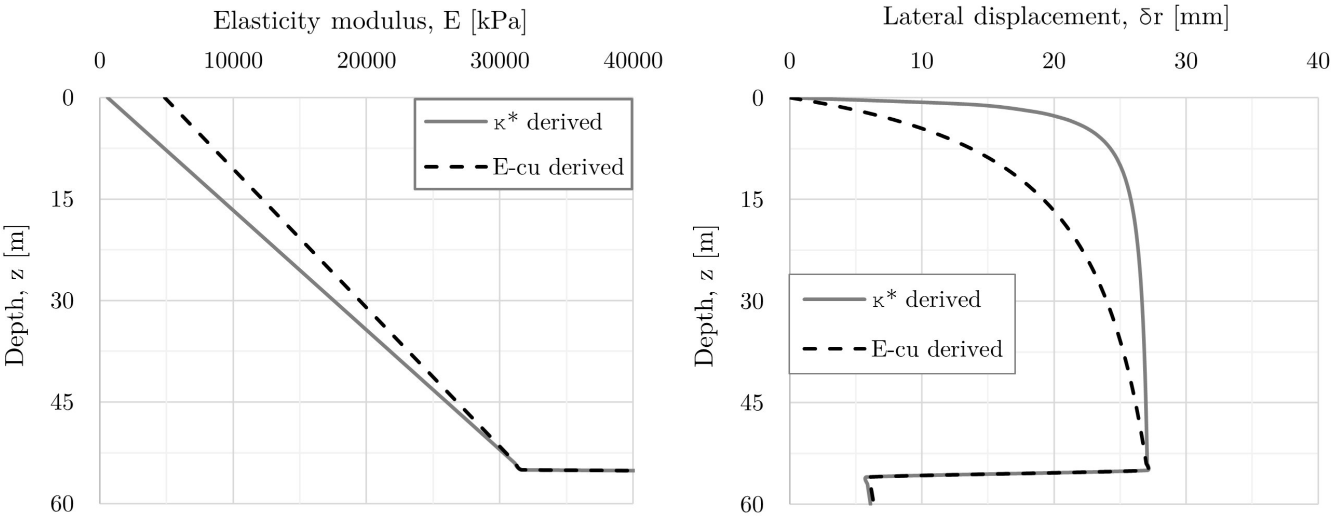


(a) Lateral displacement normalized with the pile length over depth normalized with the pile length.

(b) Lateral displacement for different pile lengths over depth normalized with the pile length.

Figure 5.8: Results for the impact of pile length on the lateral displacement in SS FEM.

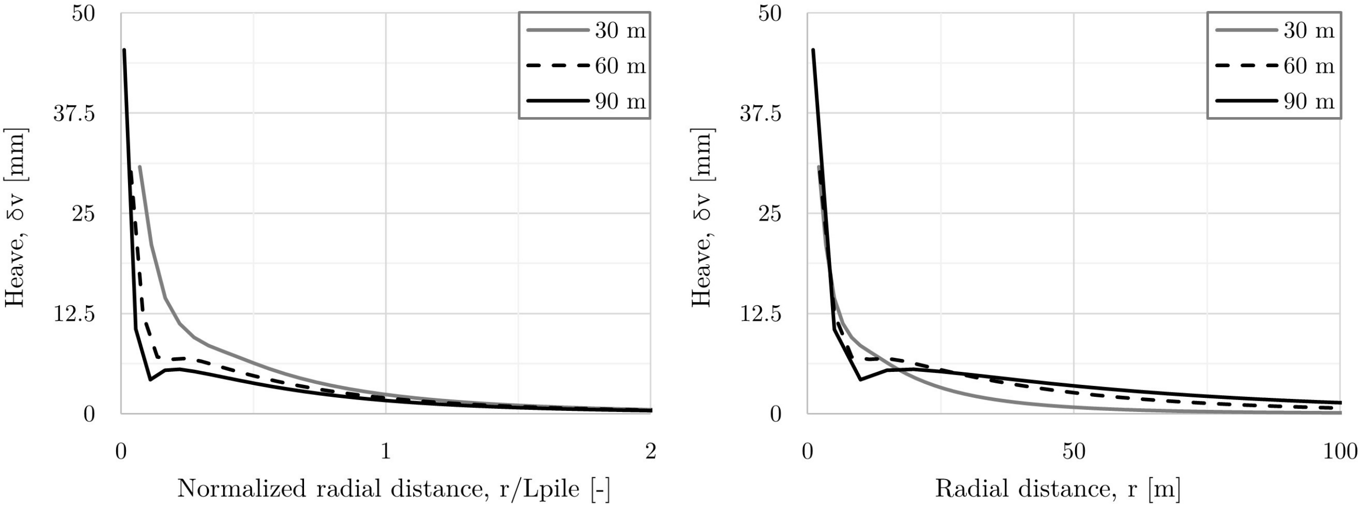
The heave for different pile lengths can be seen in Figure 5.10. The 90 meter pile results in a larger maximum value compared to the other piles, possibly due to massive amounts of soil being displaced. However, the 30 and 60 meter piles have almost identical values of maximum heave, despite displacing different soil volumes. This could possibly be related to the fact that while the short pile displaces less soil, the displacements will be more prominent on the surface, as less soil would be able to absorb the displacements before reaching the surface. On the other hand, the longer pile shows that the amount of displaced soil is the most important factor. Furthermore, Figure 5.10a shows minimal heave at a distance of between 1.5 to 2 times the pile length, a result that matches the experiences from projects in Gothenburg.



(a) Elasticity modulus over depth depending on calculation method.

(b) Lateral displacements (CEM) depending on elasticity modulus.

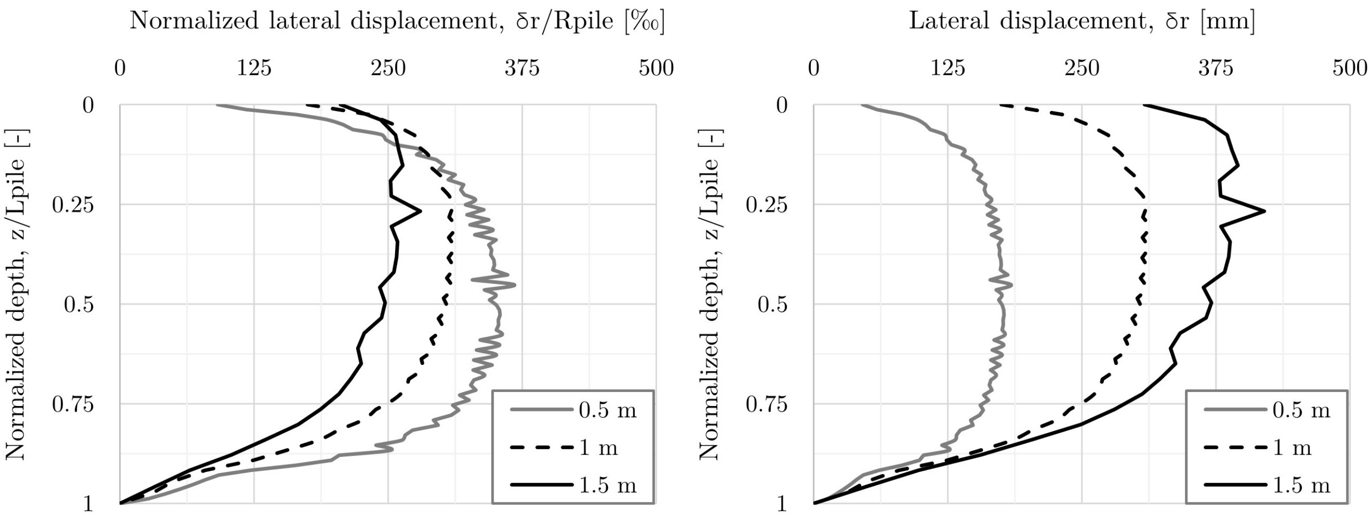
Figure 5.9: The variation of the elasticity modulus, E, over depth when derived from both κ^* and an $E - c_u$ factor of $375c_u$. At 55 meters depth the modulus rapidly increases due to the till layer.



(a) Normalized radial distance.

(b) Non-normalized radial distance.

Figure 5.10: The impact of pile length on heave according to SS FEM.



(a) Lateral displacement normalized with pile radius versus depth normalized with the pile length according to SS FEM.

(b) Lateral displacement for different pile radii versus depth normalized with the pile length according to SS FEM.

Figure 5.11: Results for the impact of pile radius, R_{pile}, on the Lateral displacement in SS FEM.

In Figure 5.11 the data points from the FEM simulation are visibly jittery due to "numerical noise", something that can be seen in other figures as well. However, the general trends of the curves show that the larger pile radii give comparatively less of relative lateral displacements. This differs from the LE model, where the normalized results were not impacted by the pile radius, conforming with the analytical solution, see Figure 5.7. This shows that once plasticity is considered, Plaxis changes how the lateral displacements are calculated.

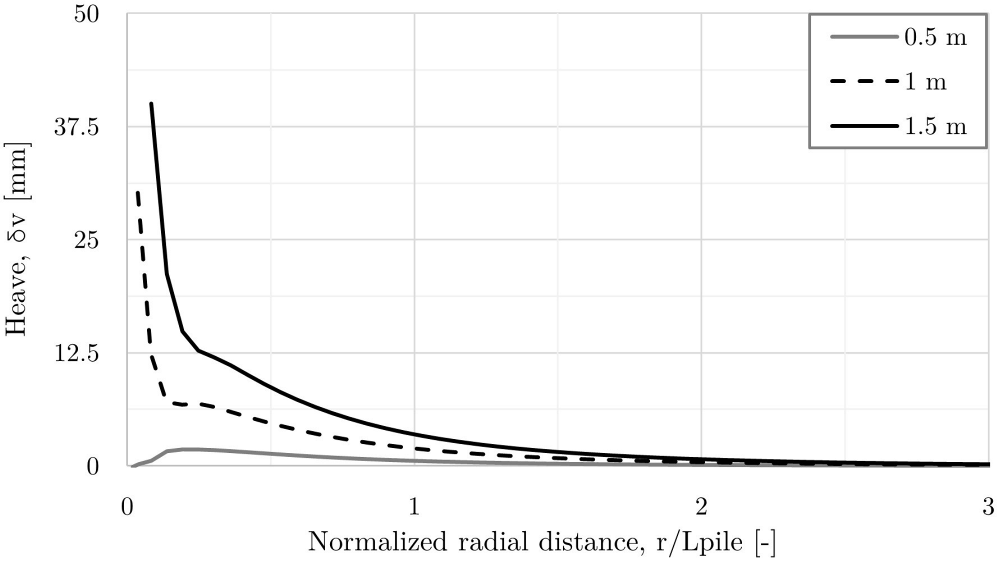


Figure 5.12: The impact of pile radius on the heave in SS FEM.

As seen in Figure 5.12 the heave caused by two bigger piles indicate no surprising results, with a larger pile radius resulting in more heave. However, for the 0.5 meter radius pile, the vertical movements of the soil close to the pile do not conform to the expected peak. Instead the heave increases to the maximum over about 10 meters, a behavior that is similar to that of the heave in the LE models.

5.1.2.2 Soil Parameters

Various soil parameters have been studied for their influence on the results. The soil parameter study was conducted on both the LE and the SS constitutive models, but due to the aforementioned issue with the heave in LE, only the SS results will be presented here. Additionally, the lateral displacement results of the LE model mirrored the relationships seen in the SS model, so to conserve space these results were omitted in this section and are instead presented in Appendix D.

Soil stiffness

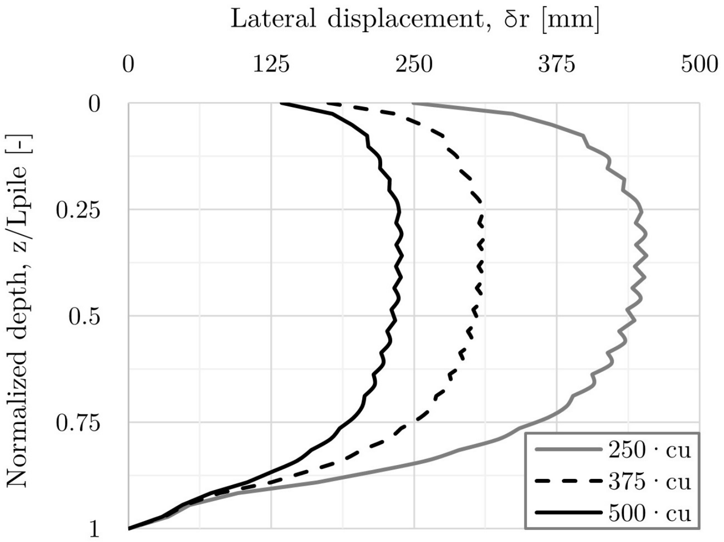


Figure 5.13: The impact of elasticity modulus factor on the lateral displacement in SS FEM.

Figure 5.13 and Figure 5.14 show that the elasticity modulus substantially impacts the results. This behavior is expected, as the elasticity modulus dictates the stiffness of the soil, allowing for more or less deformation at the same pressure level. As the radial displacements increase, so does the heaved soil at the surface. The relationship between the expanded cavity or heave and the change in elasticity modulus follows the same order of magnitude, highlighting the relationship between the radial and vertical soil displacements.

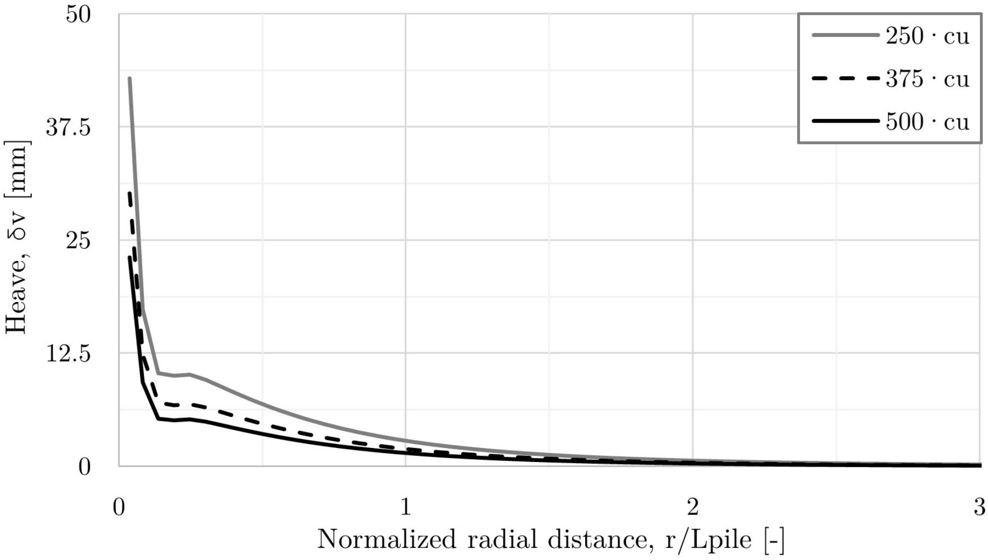


Figure 5.14: The impact of elasticity modulus factor on the heave in SS FEM.

Overconsolidation ratio

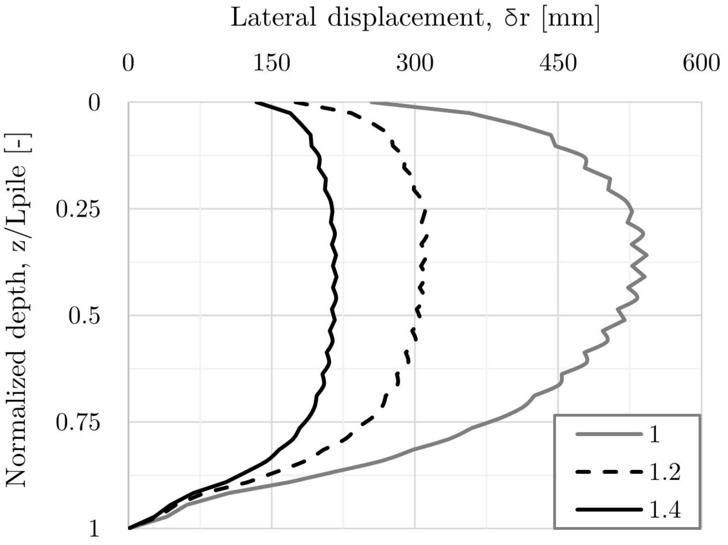


Figure 5.15: The impact of OCR on the lateral displacement in SS FEM.

OCR is the ratio between the maximum overburden stress the soil has experienced previously and the current in-situ stress level. This in turn dictates how much pressure the soil can withstand before it significantly deforms. Figure 5.15 shows this relationship, where a high OCR results in less deformations, while a lower OCR results in higher deformations. It is apparent that the normally consolidated test (OCR = 1) behaves plastic at a lower stress level than the other test, resulting in a greater lateral displacement which is not proportional to the change of OCR in percent.

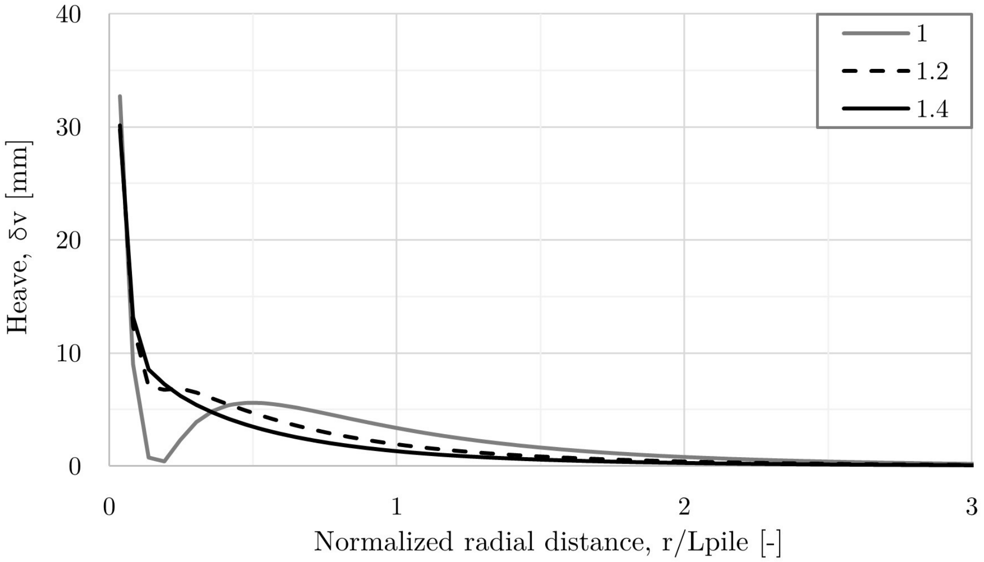


Figure 5.16: The impact of OCR on the heave in SS FEM.

Figure 5.16 shows that a lower OCR results in more heave, both adjacent and at a distance from the pile. An interesting bowl-shaped appearance of the low OCR line can be seen close to the pile. This shape is believed to be due to numerical errors and should be disregarded in favor of a smoother transition between peak heave and heave at a distance.

At-rest earth pressure coefficient

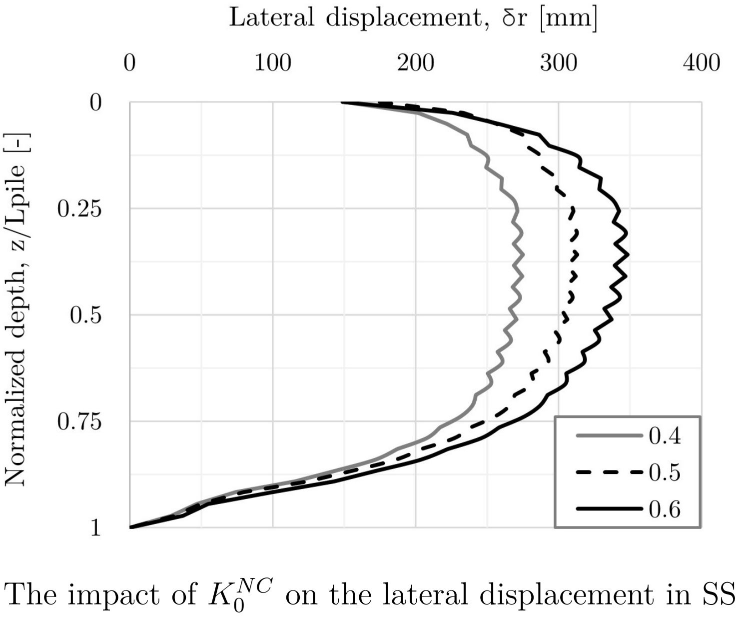


Figure 5.17: The impact of K_0^{NC} on the lateral displacement in SS FEM.

The parameter K_0^{NC} represents the relationship between vertical and horizontal stresses in a (normally consolidated) soil. Given this, the horizontal stress will act against the casting pressure of the concrete, as such a larger ratio should result in less lateral displacement. Figure 5.17 shows that this is true. As the depth approaches 55 meters and the till layer, the curves overlap since there is no longer any variation in soil parameters. In terms of differences, the results show a fairly linear relationship between the change in K_0^{NC} and the change in lateral displacement.

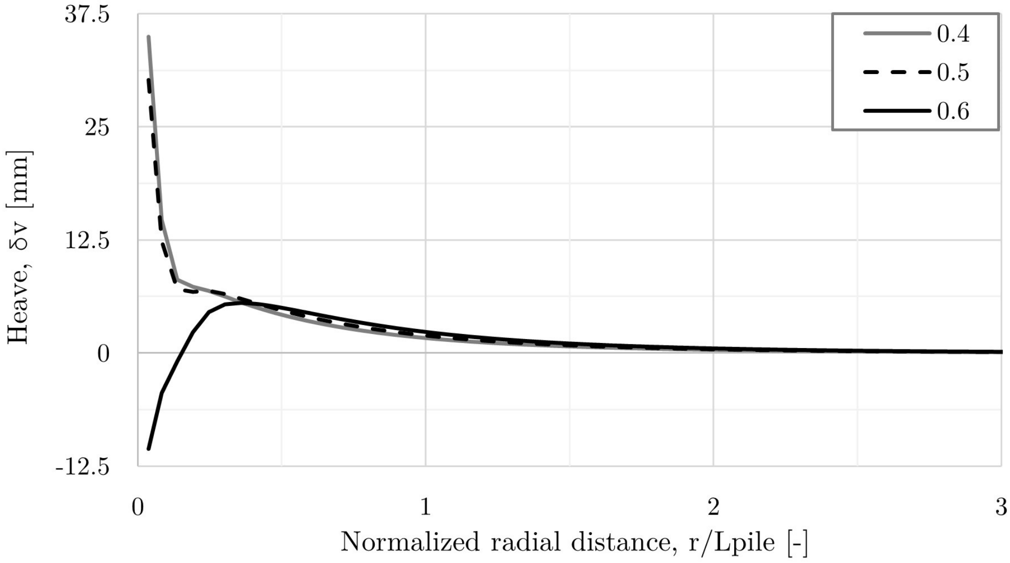


Figure 5.18: The impact of K_0^{NC} on the heave in SS FEM.

Figure 5.18 acknowledges the relationship between a larger lateral displacement and an additional heave for the lower value of K_0^{NC}. This impact is only noticeable in about 0.5 pile lengths away from the pile, before eventually the same heave is predicted. Furthermore, when the coefficient is increased, the heave behaves unexpectedly. As seen in Figure 5.12, for a pile radius 0.5 m, the heave peaks at a distance away from the pile head and the soil settles close to the pile body. This behavior mimics what the LE models predict. The reason why this occurs is believed to be that once the deformations become too low, the soil begins to collapse close to the pile. But this idea does not hold true once the maximum lateral displacement is compared between different parameter tests, as a lower expansion still appears to produce the expected results of heaving closest to the pile head.

5.1.3 Heave Caused by a Pile Group

As data for surface heave over a radial distance of a single pile are needed for input, and previous results were not considered representative due to various factors that have already been discussed, a new approach towards an analytical solution was implemented. This new approach takes the volume predicted by the CEM calculations and uses the expression for radial attenuation like in SSPM. However, instead of using the volume to calculate the wall thickness for SSPM, the volume was distributed along the surface up to a distance of $1.5L_{pile}$, as suggested by experiences in Gothenburg clay, see Section 2.4.1. The maximum and attenuated heave needed to achieve an equal volume as the lateral displacement was obtained through iteration using the Goal Seek function in Excel. Figure 5.19 schematically shows the differences in the original and adapted approaches.

For the calculations a 60 meter long pile with a radius of 1 meter was used, with 5 meters of till overlaying the bedrock. The clay layer was set to an elasticity modulus of $250c_u$, as this represents Gothenburg conditions the best and gives results in accordance with the expected lateral displacement. This resulted in am extended cavity volume of 11.5 m^3 and a maximum expansion of 41 mm. This volume was then attenuated over 90 meters as described, giving a maximum heave of about 21 mm. The original SSPM approach predicts 29 mm of heave as a comparison. The calculated data was then used as input for the pile group calculation. A piling area of 30 x 30 meters and a center-to-center (c.t.c) distance between the pile of 5 meters were assumed, resulting in a total of 49 piles within the area.

Figure 5.20 shows the spatial distribution of heave caused by the pile group, and Figure 5.21 shows a cross-section through the center. Heave within the pile area reaches values of up to 110 mm, 80 mm just outside the piling area, and the area of influence stretches to roughly 100 meters away from the center. At a distance of 50 meters from the center approximately 20 mm of heave can be estimated.

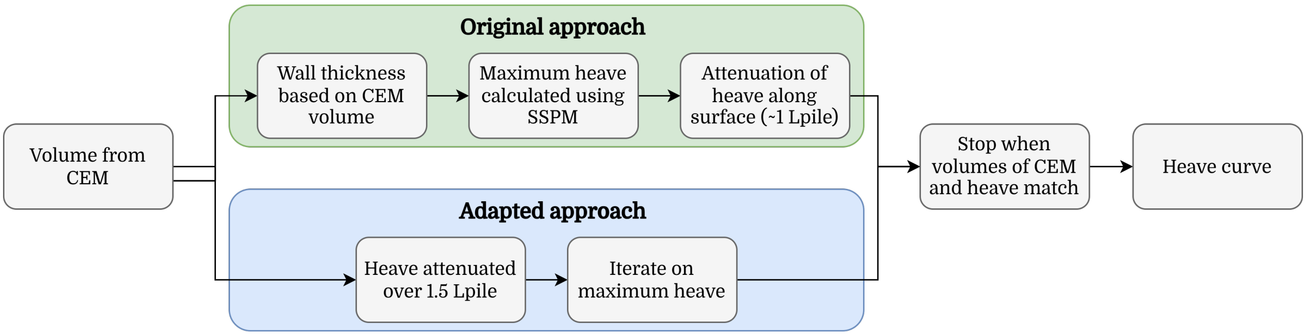


Figure 5.19: Schematic of the two approaches used for heave calculations. The adapted approach was only used for the pile group calculations.

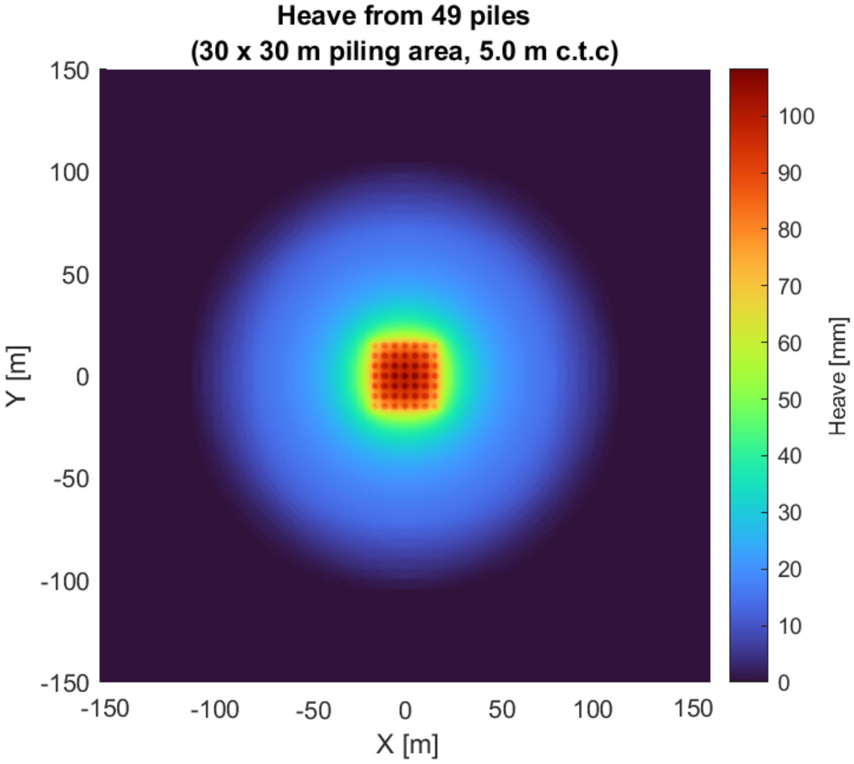


Figure 5.20: Spatial distribution of heave as caused by 49 cast-in-place piles within a piling area of 30 x 30 meters.

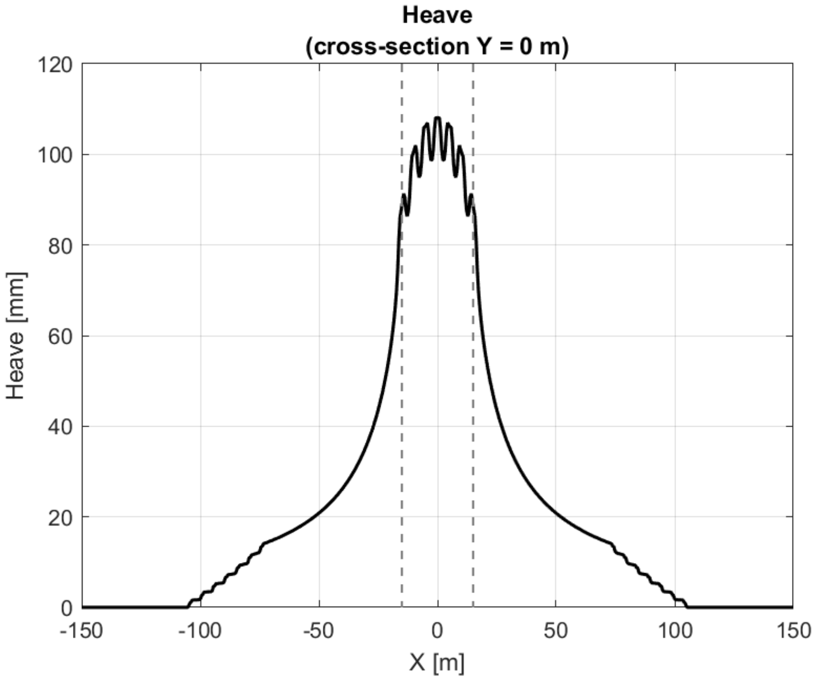


Figure 5.21: Cross-section through Y = 0 as seen in Figure 5.20, showing heave. The piling area, 30 meters across, is indicated by the dashed lines.

5.1.4 Summary of Results

This section will summarize the calculation results for the cast-in-place pile.

Calculation Methods

- Both CEM and the LE model give the same results for δ_r, disregarding the arching effect. Additionally, the volume increase of the 60 meter pile due to the expanded cavity, 4.1%, was close to the expected result of 6-7%. If instead an elasticity modulus more inline with Gothenburg conditions is used, $250c_u$ compared to $375c_u$, the volume increase instead becomes 6.1%.

- The SS model gave unrealistic results for the lateral displacement, which results up to 300 mm and extreme amounts of soil volumes being displaced. This is due to the plasticity introduced to the material. The model however allows for additional soil parameters to be studied compared to the LE model.

- Predictions of heave using a numerical model were inconclusive. The LE model and the SS model, in some cases, predicted an unexpected behavior with settlements at a close radial distance to the pile (roughly 10 meters). The SS model produced results that conformed to the believed behavior of vertical displacements, but the volumes of soil were once again unrealistic. Neither LE or SS can be assumed to be a realistic representation of what should happen in reality. Whether this is due to a limitation of the created models or the software could not be determined.

- SSPM can be used to estimate the heave by matching the volumes of displaced soil to the CEM results to a satisfiable degree. Using SSPM with a volume factor to match the volumes of heaved soil to that of SS FEM also results in similar predictions at a distance.

- According to the aforementioned assumption that the volume of the lateral displaced and heaved soil should be the same, the simulated FEM volumes of both models have been calculated and compared using Equations 3.5 and 3.8. In general, the volume of heaved soil is 4% less than the extended cavity for the LE model. For the SS model, this difference increases to roughly 20%. This highlights that the SS model allows for more compaction of the soil.

Parameter Study

- The radial displacement, δ_r, depends on the elasticity modulus, K_0 and the pile radius in a linear elastic model. It is not impacted by the pile length (for a given point $< L_{pile}$).

- The vertical displacement, δ_v, depends on the elasticity modulus, K_0^{NC}, and the pile radius in the Soft soil model. While the OCR ratio only shows a negligible impact.

5.2 Drilled Steel Tube Pile

The following section outlines the results of the parameter study performed on the numerical models for the drilled steel tube pile. The results will cover variations in geometric parameters, soil properties, and simulated volume loss, in order to assess their influence on settlements. Similarly to the previous section, the values have been normalized to allow for easier pattern recognition across the dataset, where vertical displacements are plotted along the surface profile, and the horizontal boundary is limited to three.

5.2.1 Note on Model Limitations

While processing the data, it was identified that the imposed volumetric contraction, used to simulate excessive washout of soil, was not fully realized in the numerical model. This became evident when the actual volume loss was calculated based on the settlement profiles and compared against the target value. From further analysis in the Plaxis output, it was discovered that the applied volumetric strain was altered by the program during the simulation and varies in the soil cluster exposed to volumetric contraction, see Figure 5.22. According to the reference manual, this discrepancy is believed to be due to the software not always fully enforcing the imposed volumetric strains, due to the stiffness of the adjacent soil and objects (Bentley Systems, 2024b). To address this, all vertical settlement results have been normalized with a volume factor, see Equation 5.1:

$$V_{\text{factor}} = \frac{V_{\text{actual}}}{V_{\text{prescribed}}} \tag{5.1}$$

Where
V_{actual} is the volume loss realized in the Plaxis output [m^3]
$V_{prescribed}$ is the prescribed volume loss [m^3]

This normalization allows settlement values to be scaled against the targeted volume loss, which supports continued analysis and comparison of overall trends. However, the absolute values should be interpreted with caution due to the variation in the actual volume loss. Furthermore, to remain transparent, all presented plots include the actual volume loss realized in each simulation. Appendix E presents a detailed table on the differences in volume loss and applied V_{factor}.

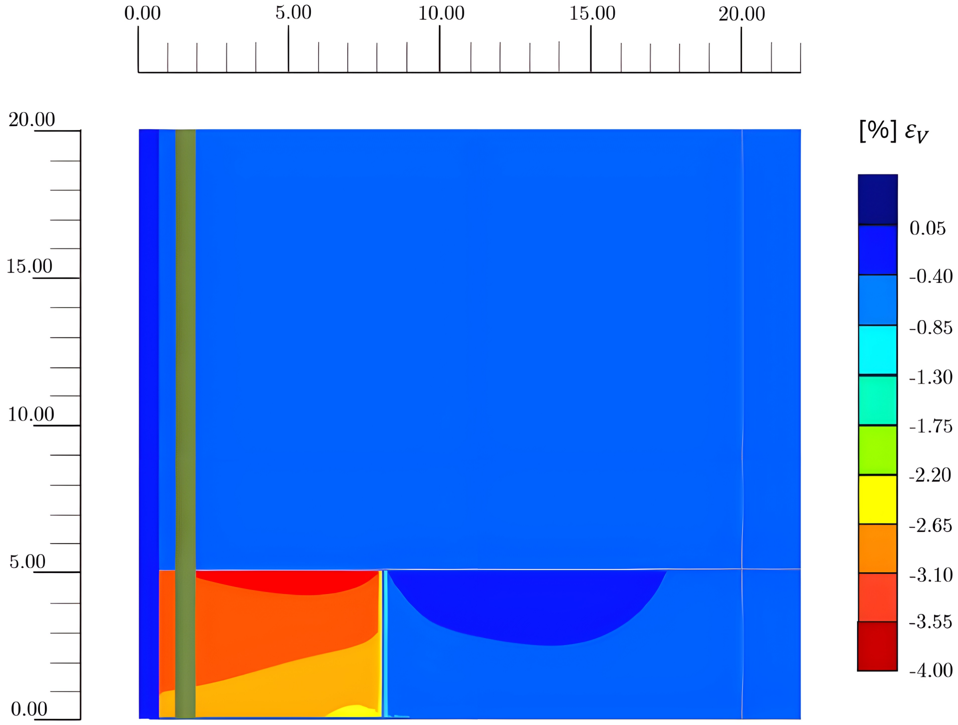


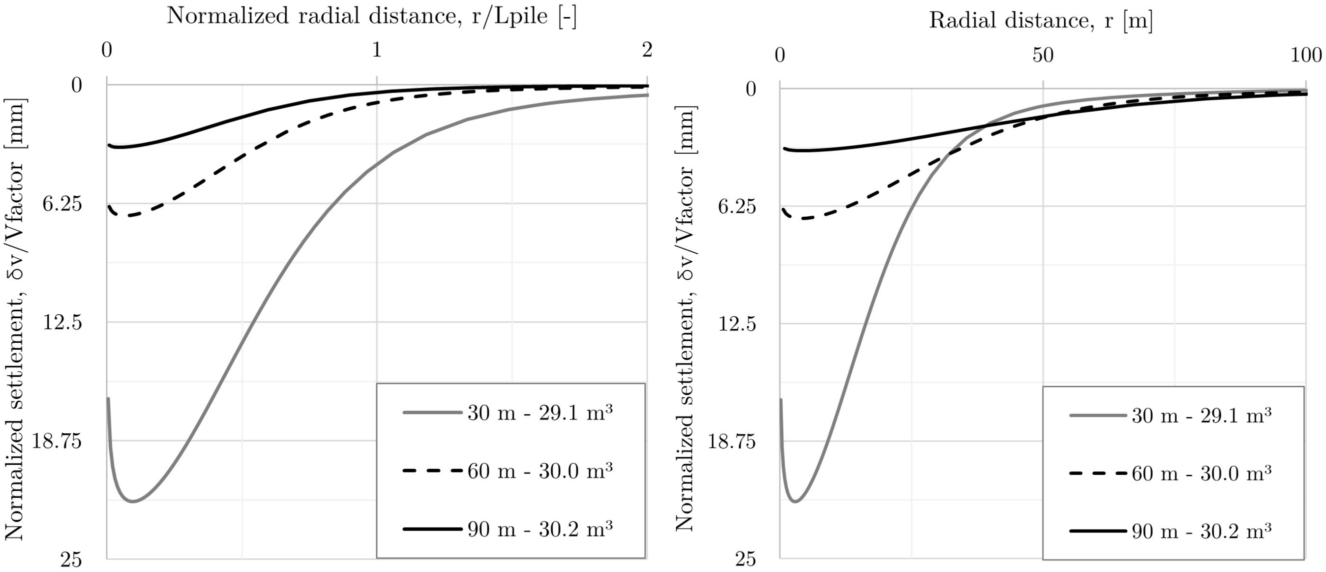
Figure 5.22: Visualization of the varying volumetric strain in the Plaxis output for the LE model, which differs from the imposed constant strain of -4.8% over the entire soil cluster.

5.2.2 Parameter Study

This section contains results specific to the parameter study of both the geometry and the soil parameters for the drilled steel tube pile. The results will cover the impact of different variations in geometry and soil parameter on settlements.

5.2.2.1 Geometry

Variations in pile length have been evaluated using the LE and SS sets of material models in Plaxis. Figure 5.23, illustrates the surface settlements calculated using the LE model, with an imposed negative volumetric expansion, representing a volume loss of approximately 48 m^3, which corresponds to ten times the theoretical volume of a 60 meter pile. Figure 5.23b, presents non-normalized radial distance to better illustrate the differences over the radial distance for varying pile lengths.
As seen in Figure 5.23a, surface settlements are significantly larger when volume loss occurs closer to the surface, which is influenced by the pile length. This outcome is to be expected, as the volume loss is proportionally larger for the shallower pile. Furthermore, it can be observed that the influence area of the settlements increases with the length of the pile, since volume loss occurs deeper in the soil. The shape of the radial distribution of the settlements is similar between the varying pile lengths.

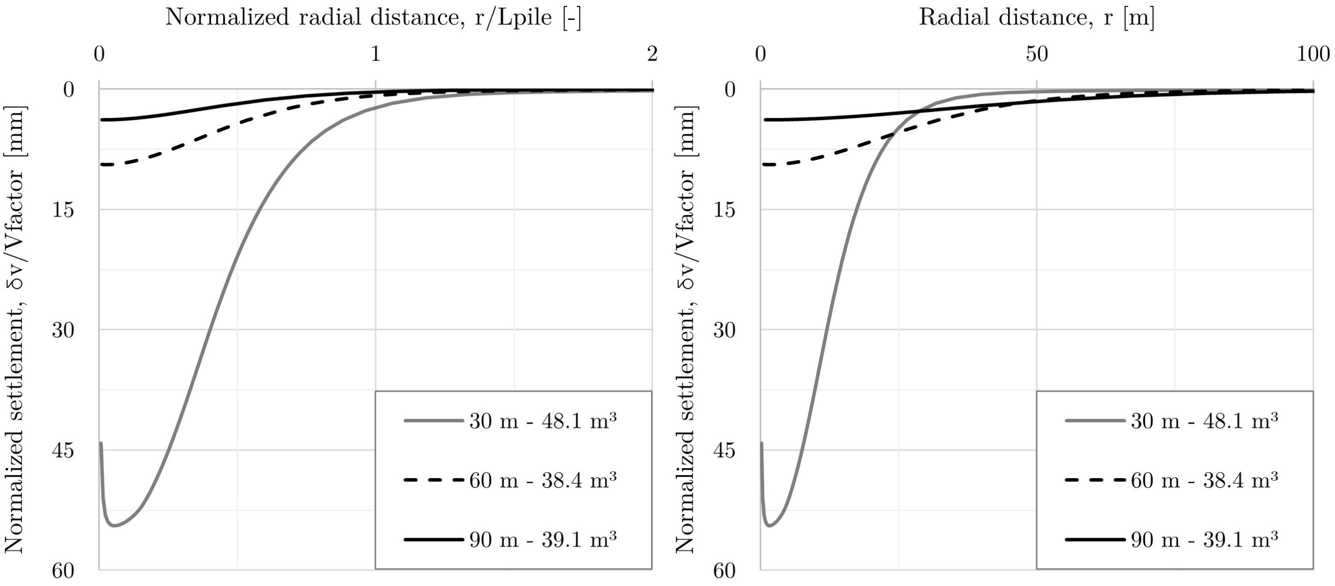


(a) Settlement distribution for different pile lengths with a volume loss representing 48 m^3 where radial distance is normalized.

(b) Settlement distribution for different pile lengths with a volume loss representing 48 m^3 along the radial distance in meters.

Figure 5.23: Results for the impact of pile length on the settlement distribution in LE FEM.

A similar trend is experienced for the SS model, illustrated in Figure 5.24, where the same volumetric strain is applied. As observed, the settlements in the SS model are larger, particularly for the shorter pile. This difference is believed to stem from the model's ability to capture plastic deformations both within the till and clay layer, and the stress redistribution through cap hardening points in the clay. Thus, resulting in larger settlements. Furthermore, as previously covered in Figure 5.9 the difference in soil stiffness contributes to larger settlements in SS. In Figure 5.25, an additional comparison between the LE and SS models is shown, for pile lengths of 60 and 90 meters.



(a) Settlement distribution for different pile lengths with a volume loss representing 48 m^3 where radial distance is normalized.

(b) Settlement distribution for different pile lengths with a volume loss representing 48 m^3 along the radial distance in meters.

Figure 5.24: Results for the impact of pile length on the settlement distribution in LE FEM.

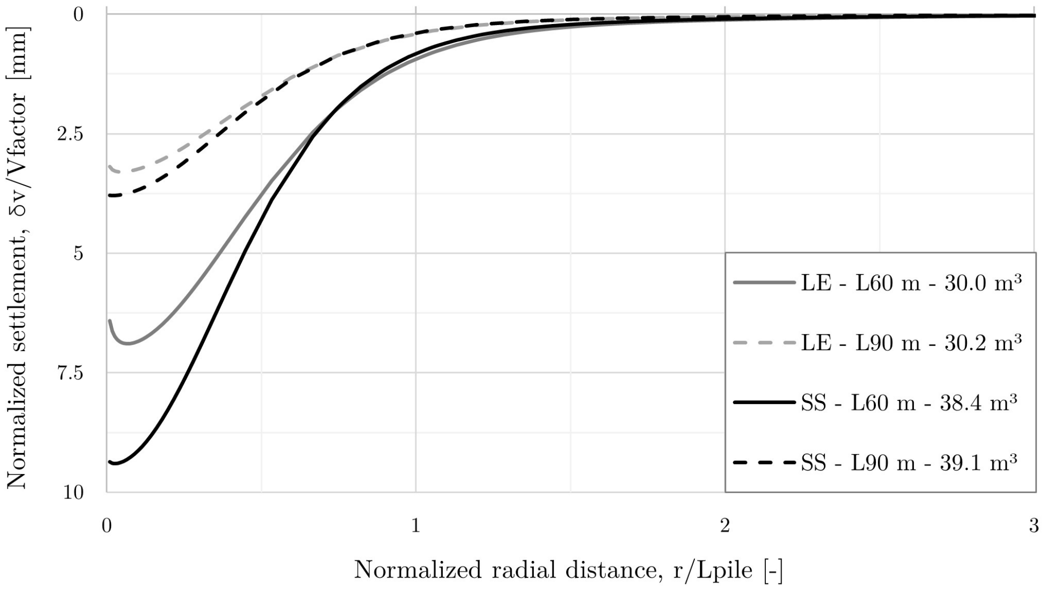


Figure 5.25: Normalized surface settlement compared between SS and LE FEM.

Due to discrepancies in the enforced volumetric strains, an alternative method with the corresponding positive volumetric strain was performed, see Figure 5.26. Due to symmetry of the linear elastic response of the LE model, the generated heave can be interpreted as settlement by changing the sign of the displacements. However, in the figure, the sign of the values is kept. As illustrated, both models respond in a similar manner, where the distribution of heave and settlements follows the same shape along the X-axis. The difference between normalized heave and settlement is considered low. The variation between models originates from the same model limitation, covered in Section 5.2.1. Since the difference between the imposed and actualized volumetric strains was not further improved, the use of negative volumetric strains continued to be adopted for the remainder of the FEM calculations.

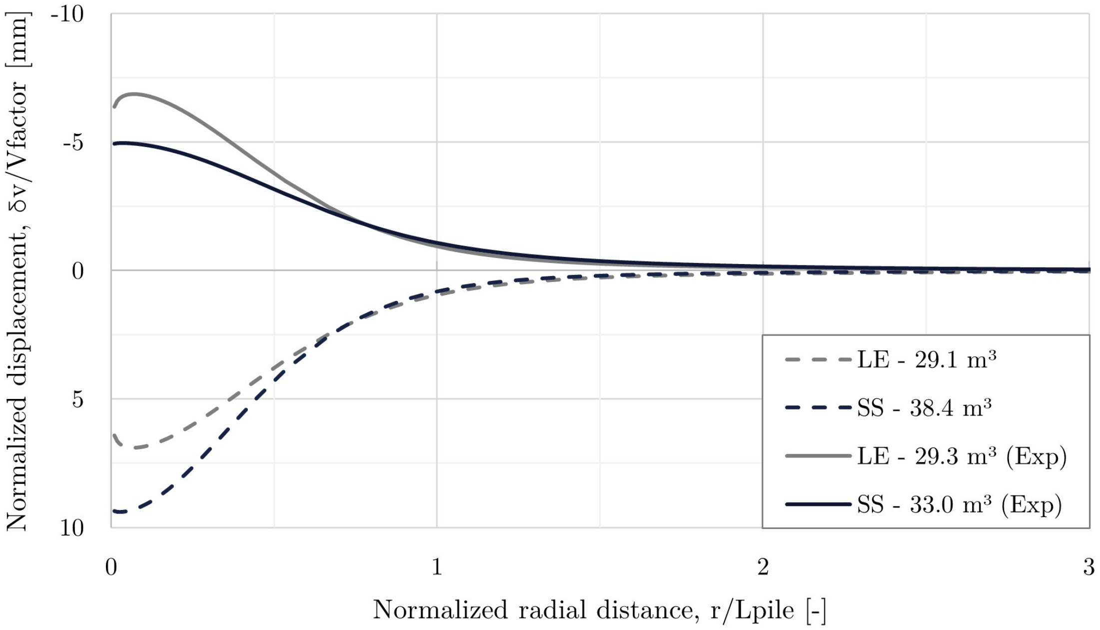


Figure 5.26: Normalized surface displacements versus normalized radial distance between SS and LE FEM using either a volumetric contraction or expansion (Exp).

To model the influence of whether the excessive washout is concentrated around the pile or is more widespread, a series of additional simulations were performed. These included variations in shape and the imposed volume contraction of the soil cluster, located in the till layer. All simulations were modeled to represent the same volume loss as the original setup, and thus the volumetric strain had to be adjusted in each simulation, following Equation 5.2. In total, four alternative shapes were simulated and compared against the base scenario. The configuration of these are listed in Table 5.1.

$$\varepsilon_v = \frac{V_{\text{prescribed}}}{h \cdot \pi \cdot (R^2_{\text{cluster}} - R^2_{\text{pile}})} \tag{5.2}$$

Where
ε_v is the imposed volumetric strain
$V_{\text{prescribed}}$ is the targeted volume loss [m_3]
h is the height of the soil cluster [m]
R_{cluster} is the radius of the soil cluster from the axis of symmetry [m]
R_{pile} is the radius of the pile [m]

Table 5.1: Dimensions based on the pile radius and volumetric strains applied to variation of the soil cluster exposed to the excessive washout.

Name	**Radius [m]**	**Height [m]**	ε_V [%]
Base	8	5	-4.802
Alt 1	16	5	-1.200
Alt 2	16	2.5	-2.400
Alt 3	32	5	-0.300
Alt 4	64	5	-0.075

The purpose of these simulations was to try to identify potential correlations between the geometry of the cavity exposed to volume contraction and surface settlements. As seen in Figures 5.27 and 5.28 the settlements are lower in magnitude in the zone adjacent to the pile for geometries with greater width, while the reduction of settlements over the radial distance is higher for smaller geometries.

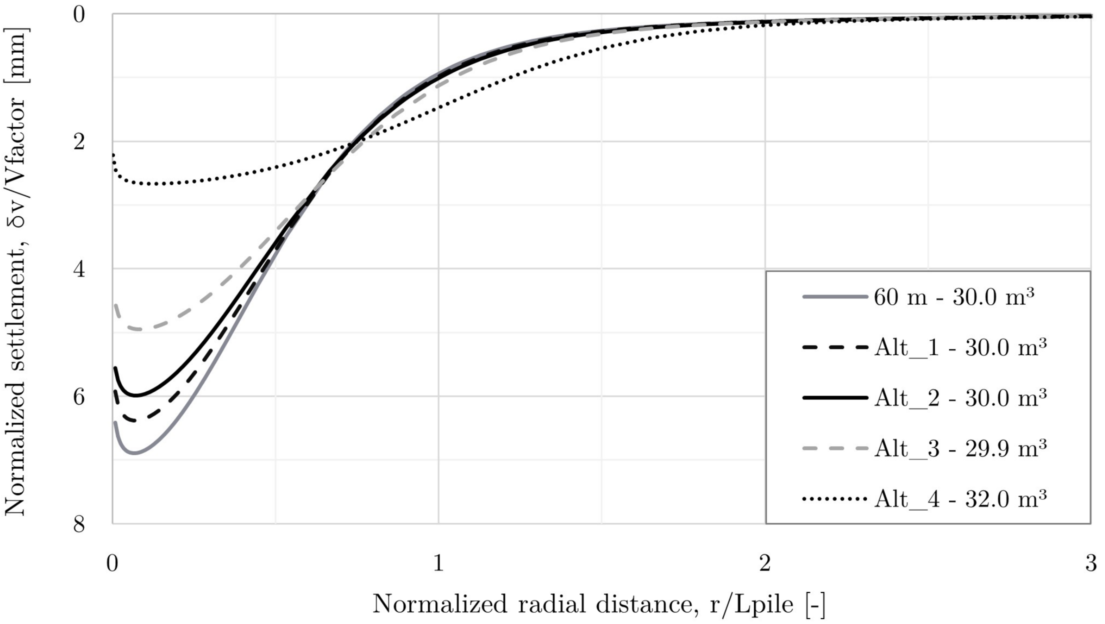


Figure 5.27: Normalized surface settlement for alternating soil cluster sizes with applied volumetric strains in LE FEM.

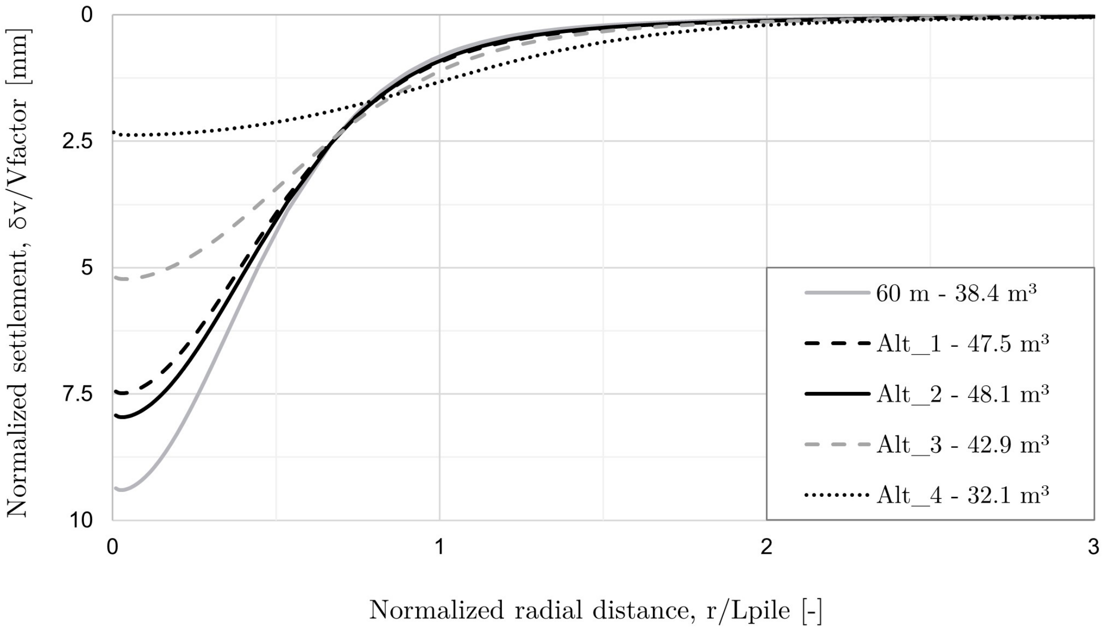


Figure 5.28: Normalized surface settlement for alternating soil cluster sizes with applied volumetric strains in SS FEM.

5.2.2.2 Soil Parameters

As part of the parameter study, a series of variations in soil parameters were carried out. This was performed to study the sensitivity of surface settlements to different soil stiffness parameters. The investigated parameters include the elasticity modulus or κ^* value, OCR and apparent cohesion in the till layer. However, due to design constraints of the LE model, only the elasticity modulus could be studied.

In Figures 5.29 and 5.30 the stiffness of the clay layers is altered, by varying the $E - c_u$ relationship. As seen, both models react similarly, where the variations in settlements are very small and the curves converge into one after the maximum settlement is reached. The believed reason for this, following Hooke's law (1D-form) in Equation 5.3, is that the prescribed strain is imposed directly, creating a strain-driven response. Although the elasticity modulus is altered, the imposed strain remains constant, resulting in a corresponding change in stress. However, a change in soil stiffness affects the volumetric strain realized in the Plaxis output for the SS model, but by applying the V_{factor} the target strain is reached, as previously explained in Section 5.2.1.

$$\sigma = E \cdot \varepsilon \tag{5.3}$$

Where
σ is the stress [Pa]
E is the elasticity modulus [Pa]
ε is the strain [-]

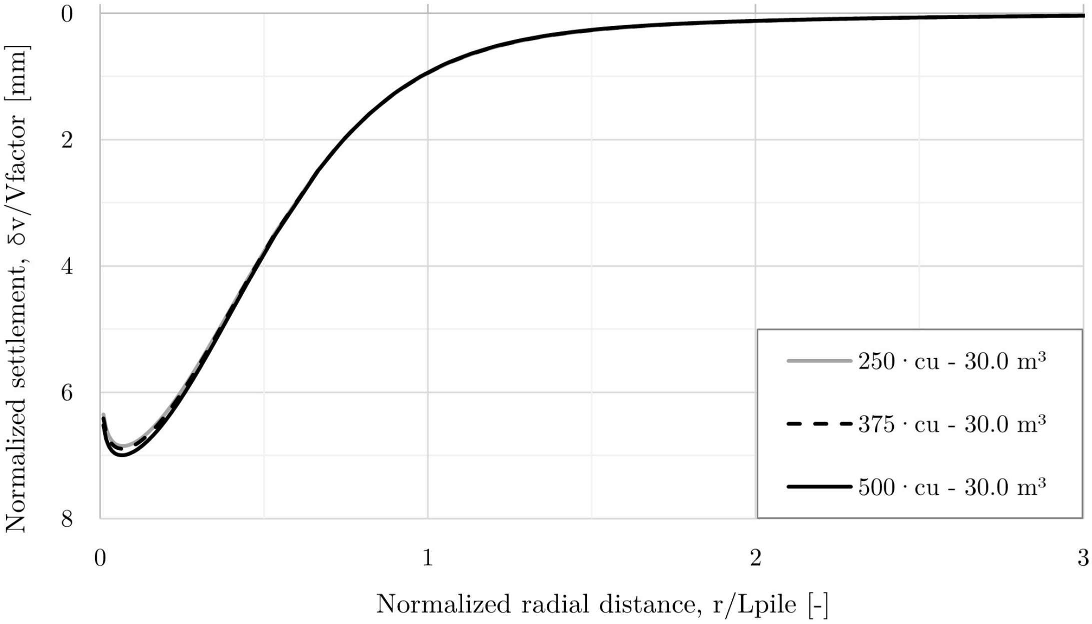


Figure 5.29: Normalized surface settlement for alternating elasticity modulus based on the E-c_u factor in LE FEM.

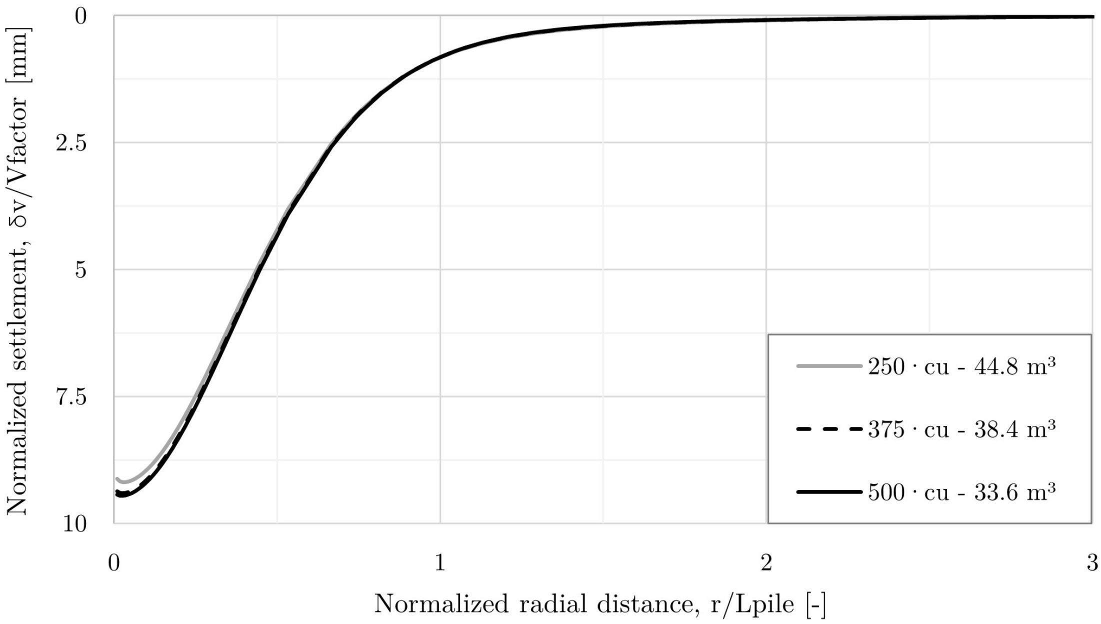


Figure 5.30: Normalized surface settlement for alternating κ^* value based on the E-c_u factor in SS FEM.

Figure 5.31 displays the effect of alternating OCR, where a decrease in OCR leads to greater settlements, while an increase leads to less soil deformation. It is interesting to note that the OCR influences the settlements generated compared to the value κ^*, based on the $E - c_u$ relationship. This is believed to be due to the change in preconsolidation pressure, where a higher OCR increases its value. As seen in Figure 2.12, the preconsolidation pressure affects the size of the yield surface of the clay layer. As this expands, the elastic response increases, thus delaying the onset of plastic deformations and reducing settlements. Based on the Plaxis output, a considerable difference in generated cap points was observed between the variations in OCR.

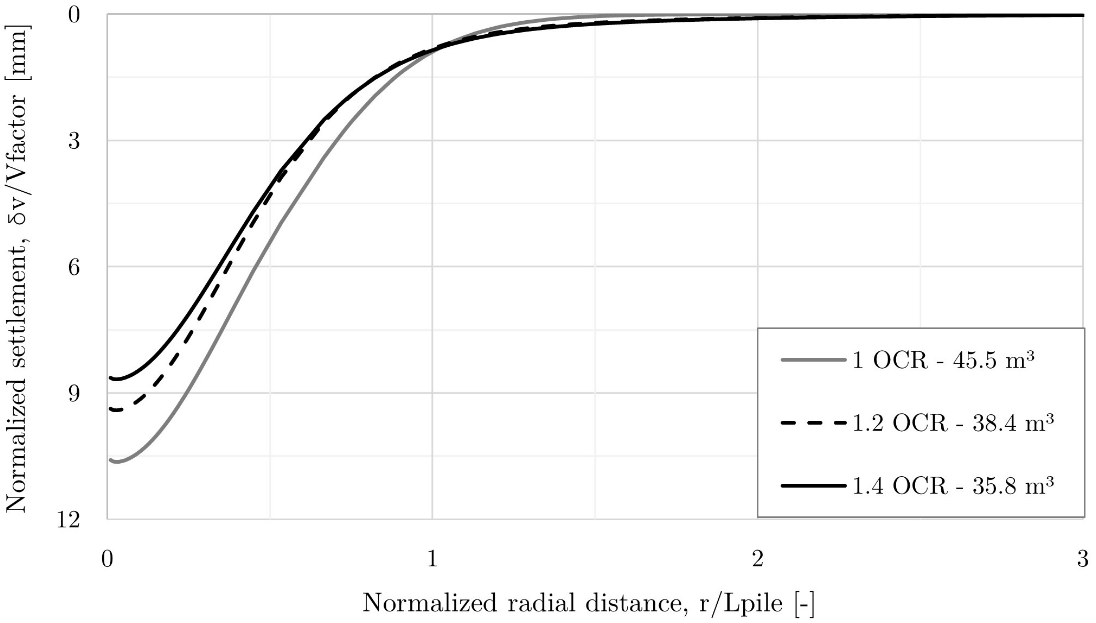


Figure 5.31: The impact of alternating OCR on surface settlements in SS FEM.

Variations in apparent cohesion, c', were performed for the till layer, modeled with a Mohr-Coulumb material. Studying this parameter was of interest because of its potential impact in reducing plastic deformations. As seen in Figure 2.12, the c'-value governs the Y-intersect of the failure line and following Equation 5.3 the imposed volumetric strain would in theory significantly reduce the principal stress in the soil, effectively shifting the Mohr's circle closer to the origin in the stress space. As a result, the till layer would become more sensitive to variations in c', however, as seen in Figure 5.32, the variations have little to no impact on the settlements.

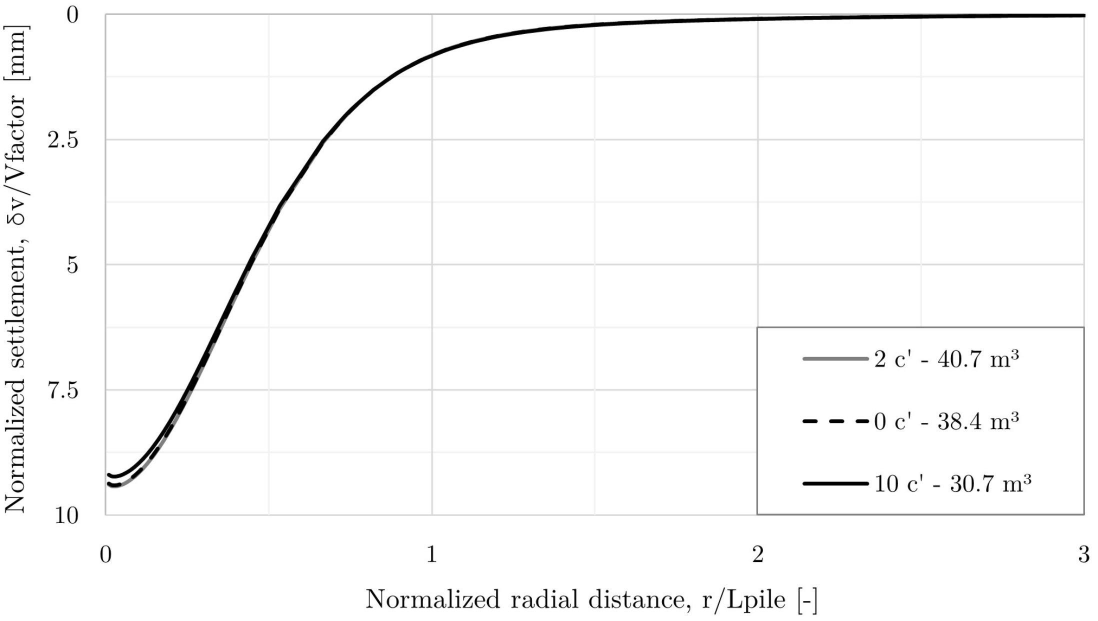


Figure 5.32: Normalized surface settlement for alternating apparent cohesion values in the till layer in SS FEM.

5.2.3 Settlements of a Pile Groups

To calculate the settlements resulting from the installation of a pile group, the method previously presented in 5.1.3 was used. The analysis was based on the following assumptions:

- LE soil model with baseline geometry
- $E = 250c_u$ [kPa]
- $L_{pile} = 60$ [m]
- $\Delta V_{\text{pile}} = 10$ [m^3]

The volume loss was assumed based on the responses from the interview study, where the correct volume loss was achieved by manually calibrating the volumetric strain until the target condition was achieved. The pile group was assumed to support an eight-story residential building, with a corresponding piling area of 25 x 25 meters, with a c.t.c of 5 m. The value of c.t.c was based on an estimated surface load of 10 kN/m^2 and an estimated design load of 2000 kN per pile, resulting in a total of 36 piles.

Figure 5.33 shows the uniform spatial distribution of surface settlements caused by the installation of the pile group, and in Figure 5.34 a cross-section of its distribution is illustrated through the Y = 0 axis of the system. Based on the figures, the influence area is seen to cover up to 150 meters and a settlement of approximately 10 mm is observed at a distance of 50 meters away from the pile group center.

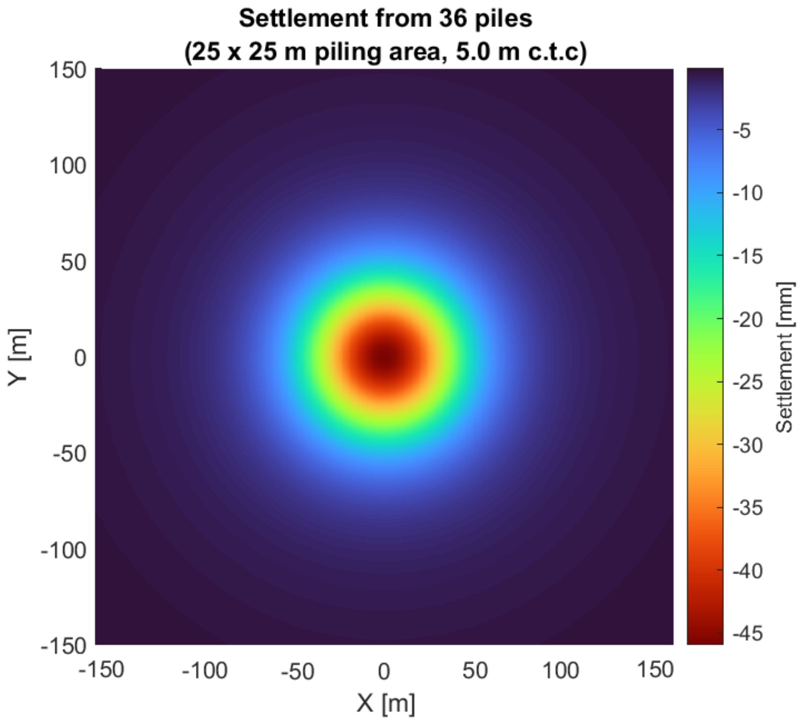


Figure 5.33: Spatial distribution of surface settlements generated by the installation of 36 drilled steel tube piles.

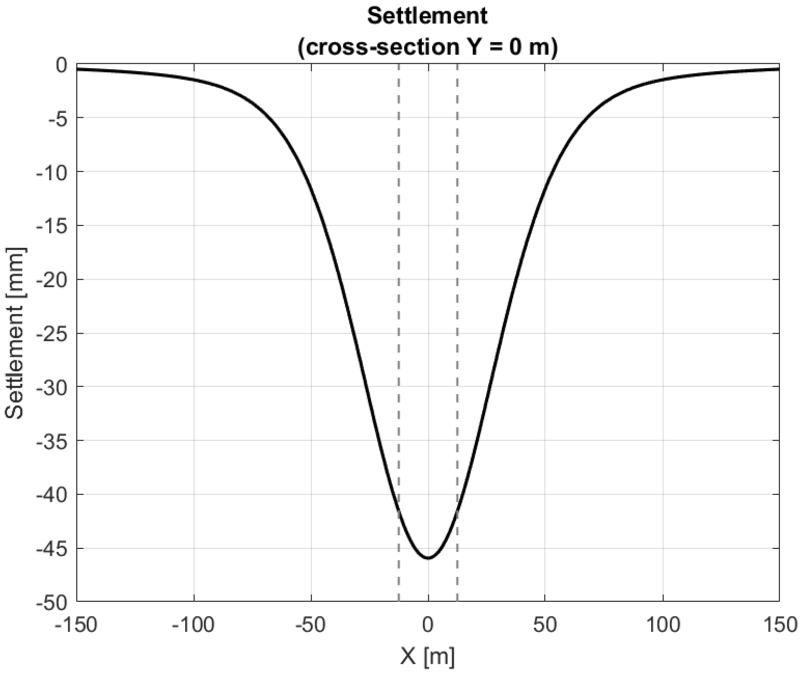


Figure 5.34: Cross-section through Y = 0 as seen in Figure 5.33, illustrating the settlement at a distance away from the center. The piling area, 25 meters across, is indicated by the dashed lines.

5.2.4 Summary of Results

Despite limitations of the model described previously, the results proved valuable in identifying trends and facilitating comparisons between the LE and SS models by normalizing the data. In the following list of points the results are summarized for the drilled steel tube pile.

Geometric variation

- Both models showcased a correlation between settlements and volume loss.
- The spatial distribution of the settlements was found to depend on the geometric shape of the soil cluster exposed to a washout. As the washout becomes more widespread, the magnitude of settlement decreases in the zone adjacent to the pile, whereas the rate of settlement decreases less with the radial distance. Where greater settlements can be observed closer to the boundary of the influence area.
- Of the geometries tested in Table 5.1, all settlement curves are seen to converge at approximately twice the normalized radial distance, which corresponds to 120 meters for a 60 meter pile.

Variation of soil stiffness

- The apparent cohesion and $E - c_u$ had a negligible impact on the results, due to a strain-driven response.
- Variations in OCR proved influential on the settlements generated. As the OCR increased, the preconsolidation pressure increased, resulting in a larger yield surface. This led to lower plastic deformations that reduced settlements.

Pile group

- Simulations of an installed pile group demonstrated considerable surface settlements, despite the applied volume loss being within the range deemed acceptable according to interview responses and small piling area.

6

Discussion

The following chapter presents a general discussion on the results obtained from the interview study and calculations. The discussion will cover the influence of pile groups, a comparison between analytical and numerical calculations, and a comparison of key findings of the interview study to findings from reviewed literature, primarily the BegrensSkade project.

It is important to recognize that the numerical calculations performed are not intended to represent precise values but rather serve as indicative results of the predicted behavior of the drilled steel tube and the cast-in-place piles. This is due to model simplifications and the lack of reference material, resulting in uncertainties. As described by Sandene et al. (2023), there is limited available research related to the installation effects of drilled steel tube piles, with regard to excessive washout.

6.1 Comparison With Other Drilled Steel Tube Pile Studies

While the interview study presented in Section 4 provided knowledge and insight into the Swedish practices regarding drilled steel tube piles, this section aims to compare the findings and recommendations presented in other (mainly Norwegian) studies with current Swedish practices. Rather than reiterating the interview findings in detail, a comparison of the recommendations outlined could provide valuable insights and explore how the established recommendations can inform and enhance the execution of drilled steel tube piles. Although the study focuses mainly on the local conditions and practice in Gothenburg, a comparison is still of interest.

Following the recommendations of the BegrensSkade study, both studies align on the fact that each project requires an individual assessment and the importance that a skilled drill rig operator has on minimizing negative impacts and damages. In particular, as the drill rig operator controls the rate of penetration (ROP) and return flow, which is emphasized in both studies as critical factors to control. However, under the current design of technical specifications, the drilling procedure is rarely regulated in the Swedish or Norwegian context, and thus the responsibility lies with the contractor. According to Baardvik et al. (2016) the contractor thus tends to prioritize methods that provide a faster production rate, which usually increases the risk of settlements and damages. This point of view was recognized throughout the interview study, considering both the choice of method and the drilling procedure. As an example, the importance of controlling the ROP was emphasized by several respondents, but no internal guidelines were in place, instead the drill rig operator is solely responsible. Compared to the previously mentioned guideline value of 0.5-1 m/min from the Norwegian study, this was clearly exceeded by visual observation during a site visit. However, in larger Swedish cities, such as Gothenburg, requirements limiting allowable ground movements are becoming increasingly common. These requirements appear to better regulate the formerly mentioned tendency and do not require several guideline values of specific drilling parameters to be in place, which are difficult to control. Despite this, the interview findings suggest that the responsibility for ground movements may be uncertain and that current tender specifications and risk models ought to be revised. This is believed to improve the industry both in terms of fairness and risk management.

Furthermore, when examining the recommendations related to improved documentation and drilling control, it became evident that many of the proposed measures align well with current Swedish practice, except for test drilling. This was only described by one interviewee, where test drilling was sometimes performed if it was deemed cost-effective and beneficial for the project outcome. However, Baardvik et al. (2016) have emphasized that this is a proven approach to limit damage, particularly in projects where sensitive soil conditions and structures are present. It provides valuable information in making the right choice of drilling method, equipment, and procedure. If performed, it is essential that test drilling is carried out in sequence with a carefully designed measurement program, to ensure accurate documentation, preferably commenced in the start-up of drilling. Based on project experiences in Norway, where it has been performed, desirable outcomes have been achieved.

Naturally, it would be of interest to control and quantify the volume of cuttings, as this would be a direct measurement of whether excessive washout or mass displacement has occurred. However, accurately measuring this is very challenging and requires several assumptions during the drilling process, as highlighted during the interviews. Lande et al. (2024) attempted to measure the volume balance during drilling, but this required several assumptions which provided uncertainty. Yet another approach was proposed, which was based on a non-dimensional framework, where a relationship between normalized cutting volume and normalized flow rate was developed. However, this requires accurate on-site measurements of key drilling parameters, which also makes it difficult to apply in practice.

As previously highlighted in this report, the BegrensSkade study strongly advocates the use of water-powered drilling methods, when it is technically viable. Apart from the cost difference to pneumatic drilling methods, which notably influences their application, the method still does not receive unanimous support based on the interviews. This is in part due to the practical challenges experienced due to the additional management of water and disposal of cuttings, which is further complicated during periods with low temperatures. Moreover, if the trend for larger pile diameters continues, this may become more complex. Evident is also that the cost and environmental impact of the water supply often appears to be overlooked prior to the selection of hydraulic drilling methods. In addition, severe deviations have been reported in which hydraulic drilling in sandy and silty conditions has generated large settlements. This is in contrast to the recommendations outlined by Baardvik et al. (2016), which state that water-assisted drilling should always be employed when drilling in soft clay, silt, and sand conditions. Although the cause of these deviations does not appear to be related to an excessive washout, since adequate return flow was not fully achieved, but rather to pore pressure differentials within the soil. The impact of such differences lies beyond the scope of this thesis, but it implies that hydraulic drilling may not always be the optimal method.

6.2 Comparison of Analytical and Numerical Methods for the Cast-In-Place Pile

A strict theoretical study of the soil displacements caused by cast-in-place piles have been performed in this thesis. Both analytical and numerical methods have been used, with the ultimate goal of comparing how results between the two comply. The one physical anchor-point used was the fact that a about 15-20% concrete over-consumption, relative to the theoretical pile volume, typically occurs when casting a pile. Of this over-consumption, about 6-7 percentage points is believed to be linked to a cavity expansion occurring due to the casting pressure of the fresh concrete. A simplification was made that the full hydrostatic concrete pressure would be exerted on the soil wall, a decision supported by a study into specifically casting pressure in cast-in-place piles, see Section 2.2.1.

Both the cavity expansion method and linear elastic finite element model produced the same results for the lateral displacement, and for the baseline 60 meter long pile a volume increase of about 6.1% was calculated using Gothenburg soil conditions. However it is important to note that the pile length will impact this number. Nonetheless, for cast-in-place piles installed in Gothenburg conditions, with deep deposits of low strength clay, the initial 6-7% prediction appears to be accurate according to the linear elastic models.

As the vertical displacements were also of interest, and the linear elastic finite element model resulted in a behavior that did not adhere to the assumed behavior, the more advanced Soft soil model was also used. The heave predicted with this model behaved more according to expectations, but the volume of laterally displaced soil was significantly larger then the linear elastic model, typically around 10 times the size. The difference can be linked to the plastic behavior introduced by the Soft soil model, effectively allowing for larger deformations at the same imposed pressures. Despite this, it should be noted that the size of the plastic deformations would not occur in reality, which can be shown by the linear elastic lateral displacement results being more realistic.

The analytical calculations related to heave used a variant of the shallow strain path method (SSPM). The maximum value of predicted heave was often within the same range comparing Soft soil FEM and SSPM, however the total volume of heaved soil differed by about 5 times the amount. If the results of the analytical calculations were scaled to match the volume of heaved soil by SS FEM the predicted heave by SSPM conformed very well to the shape of the SS FEM curve after a distance of roughly 20 meters.

As a final remark, it was difficult to quantify the sensibility of the results related to the heave. For the linear elastic cavity expansion, full casting pressure can be used as a guideline to predict the lateral displacements and amount of concrete over-consumption to expect in Gothenburg clay when installing a cast-in-place pile.

6.3 Vertical Displacements Caused by Pile Groups

As piles normally are installed in groups, the collective displacements of the pile group are more interesting then that of a singular pile, especially to study the impact on the surroundings. So while the displacements caused by the single pile might not be significant, especially far away, if multiple piles are installed the displacements can add up to be critical. As such, a simplified approach of calculating the combined impact of a pile group was undertaken. This method used superposition to sum the contribution of all the piles within a group. This means that all the piles were identical, causing the same displacements, and no interaction between them occurred. One major point to highlight is that the displacements calculated in this thesis, for both piles, is that only one part of the total problem surrounding pile installations were studied and calculated. For the drilled steel tube pile, a Venturi effect can occur for one or more piles, leading to additional settlements from the pile group. Similarly, for cast-in-place piles, only about one-third of the observed concrete over-consumption is accounted for, though this likely captures the majority of associated displacements. These calculations however assumed that all the piles in the group were installed at the same time. An unfeasible achievement in reality, where about 1 pile is completed per day. Each completed pile would also stiffen the soil, reducing displacements caused by the subsequent pile.

7

Conclusion

The general aim of this thesis was to better understand and assess the installation effects related to cast-in-place piles and drilled steel tube piles, which are considered to have a negligible influence on surrounding soil. This has been partially achieved through an interview study, in which experiences and insights have been collected about drilled steel tube piles and compared to reviewed literature. Furthermore, numerical calculations were performed to illustrate the in practice often underestimated effects of excessive washout. For the cast-in-place pile, numerical and analytical calculations have been carried out to study the impact casting pressure has on soil displacements.

As a part of the numerical calculations, parameter studies were performed, where 60 meter long piles served as the representative length for both pile types (a length considered representative for the deep clay deposits in Gothenburg). These analyzes were carried out for single piles, but the obtained results were then used to calculate the combined effect that an entire pile group has on adjacent soil. Based on this, the following conclusions could be drawn:

- **Drilled steel tube pile**
  - Piling contractors in the Gothenburg region are generally aware that a washout effect often will occur in permeable and erodible soils, and most have experiences from projects where this have happened.
  - The experience of drill rig operators and their observations during installation serve a crucial role in avoiding excessive soil washout.
  - The high water usage required during hydraulic drilling is often not considered, even though it may have environmental and economic implications.
  - Hydraulic drilling is less disruptive to the surrounding soil, but it may not always be the most appropriate drilling technique.
  - The drilling method should always be selected on the basis of the project requirements and site conditions. Therefore, it cannot be concluded that neither pneumatic nor hydraulic drilling should be seen as the universally advisable option.

  - Risks related to pile installations, such as unexpected soil displacements, should be discussed beforehand. A clear outline of whom is responsible for what should be established prior to signing the contract.

  - The current culture within the industry and the design of the tenders promote a tendency among contractors to prioritize production rate, rather than endorsing precautionary measures, such as guideline values for rate of penetration or test drilling. These measures have proven to be effective in reducing soil disturbance and ground settlements.

  - Guidelines regarding the robustness of a foundation can be implemented. These could additionally be used to improve tender documents and ensure a fairer industry for both clients and contractors.

  - The soil displacements observed from an excessive washout is dependent on the geometric shape of the formed cavity and the depth at which it occurs.

  - While the displacements caused by a single pile are relatively small, a group of piles can cause significant displacements even at great distances from the piling area. This was showcased when the plausible volume loss suggested by an interviewee was studied. The magnitude of settlements was on the order of approximately 0.9 mm and 0.3 mm at distances of 25 and 50 meters, respectively, for a single pile. Compared to the estimated pile group of 36 piles, where the settlements were on the order of approximately 30 mm and 12 mm at the same distances from the piling area.

- **Cast-in-place pile**

  - The lateral displacement due to the casting pressure for the cast-in-place pile can be accurately predicted assuming a linear elastic behavior. The resulting values conform with results based on past experiences in Gothenburg regarding concrete over-consumption. Calculations of heave caused by the cavity expansion are hard to judge, but SSPM is deemed appropriate in most regards.

  - As for the drilled steel tube pile, when only a single pile is considered the displacements are relatively small. However, once a pile group is considered these displacements could add up to be critical for nearby structures. From the simplified calculations performed in this study, heave caused by a cast-in-place pile group reach magnitudes of 80 mm just outside the piling area. 50 meters away the heave is approximately 20 mm.

7.1 Future Research

Currently little reference material is available on the topics covered in this thesis. Hopefully the information and findings in this thesis could serve as a basis for further research and discussion. Additional knowledge is required to fully understand the displacements caused during the installation of both drilled steel tube piles and the cast-in-place piles. Aspects that could be studied further include the following:

- **Drilled steel tube pile**
  - Under the model conditions adopted in this thesis, only the excessive washout is accounted for. Adopting a more complex model, in which the effect of pore pressure build-up and remolding of clay is considered, could prove valuable in understanding other installation effects that are currently not considered.
  - Considering the experienced model limitations, other modeling approaches could be evaluated to simulate volume loss, such as the usage of line contraction in a 3D analysis.
  - Explore the relationship that an alternating geometry of the till layer has on the spatial distribution and magnitude of settlements.
  - Current models have not been validated against field measurements but have only considered reasonable values of volume loss. In future work, it would be beneficial to compare and calibrate the models against full-scale field measurements of both a single pile and pile groups, where effects of excessive washout is considered in a short-term case.
  - In the interviews, it was expressed that under current practices disagreements often arise in relation to risk allocation and responsibility between client and contractor. Resulting in disputes, which often start once the problem has already occurred. This could be further explored and compared against the findings presented in BegrensSkade, where a comprehensive risk study was performed. Comparisons to the Norwegian industry and the findings regarding risk taking from this thesis for the Swedish industry have not been discussed, see Section 6.1.
- **Cast-in-place pile**
  - In the present study, it was assumed that the soil would be exposed to the full casting pressure during installation. Further investigations into what the fresh concrete pressure actually is when casting the pile could be warranted. Conflicting theories around a critical depth existing or not could also be covered.
  - A case-study with data for vertical displacements caused by the installation of cast-in-place piles would be needed to fully understand the mag-

nitude and behavior of displacements. Analytical calculations give reasonable results as shown in this study, but comparing the results against field measurements would give a better appreciation of how applicable they are.

- Additional installations effects from the casting stage of cast-in-place piles that was not covered in this thesis includes the fact that concrete can vacate from the pile shaft, for instance into a weak layer. This would cause concrete to flow into the soil and lift the overlaying clay, causing heave. Utilizing a finite element model to also include this aspect is also of interest.

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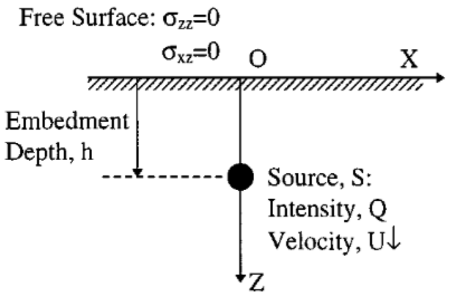
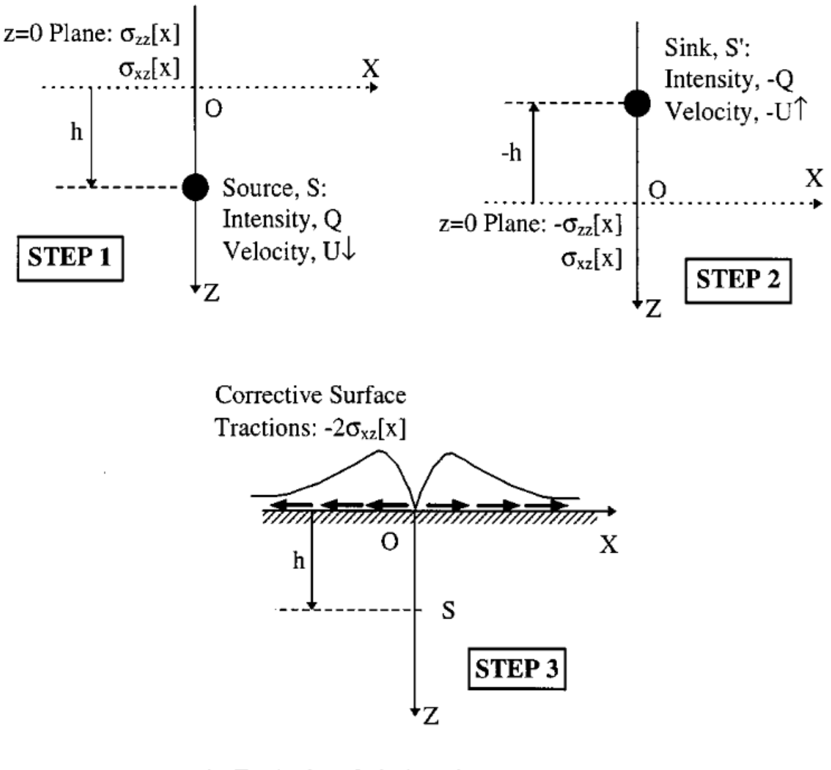
A

Shallow Strain Path Method



a) SSPM Representation of Shallow Penetration Problem



b) Equivalent Solution (Steps 1+2+3)

Figure A.1: The conceptual model of the shallow strain path method (SSPM) for the penetration of a pile. (From Sagaseta et al., 1997).

B

Cavity Expansion Method

By equating the soil volume of the original cavity, with the volume of the expanded cavity, see Figure B.1, the expression in Equation B.1 can be derived. Equation B.2 provides a ratio between the cavity expansion and pile radius. R_i is the radius of soil before pile installation, R_o is the pile radius and ξ is the cavity expansion.

$$R_i^2 = (R_i + \xi)^2 - R_o^2 \tag{B.1}$$

$$\frac{\xi}{R_o} = ([\frac{R_i}{R_o}]^2 + 1)^{\frac{1}{2}} - \frac{R_i}{R_o} \tag{B.2}$$

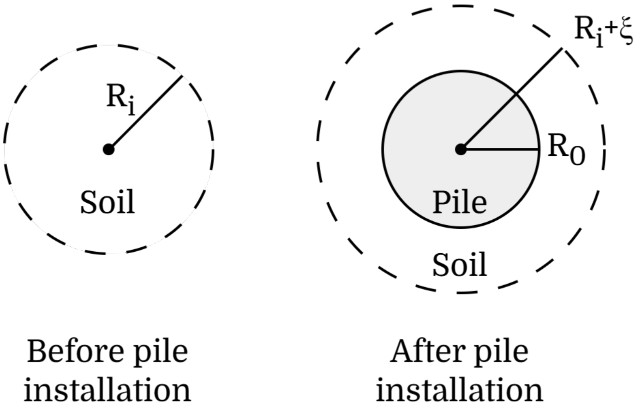


Figure B.1: Principle for the cavity expansion method of a single pile (Recreated from Randolph et al., 1979).

C

Interview Questionnaire

General Information

- How common is it that you as a contractor select the pile type for your projects?
- How common is it that you select the method of installation?
- When do you use bored steel tube piles, if you as a contractor get to choose?
- What advantages and disadvantages do you find with this pile type compared to other pile types, such as driven concrete piles?
- How big is the demand for this type of pile on the market currently? Is the demand for bored piles increasing? Are they more common for certain types of constructions?
- What dimensions are commonly used? What differs in the usage of drilling equipment etc. with the different dimensions?
- How do you verify the bearing capacity of the pile?

Selection of Method

- Drilling method

- What drilling technique is most adopted for the installation of bored steel tube piles, and what governs the selection of method?
- When do you opt for pneumatic drilling?
- When do you opt for hydraulic drilling?

- Drill bit

- What types of drill bits do you use (wing drive, concentric, eccentric)?
- What benefits and drawbacks do you see with these, and in what usage scenarios?
- RC – reversed circulation?

Deviations

- What are the most common deviations during installation?
- How do you manage any potential deviations?
- What installation effects do you find to be the most critical when using bored steel tube piles? Can these vary depending on method adopted, drill bit etc?
- Have you been involved in projects where there has been significant impact on the surroundings during installation of bored steel tube piles?
  - If yes, what type, and how did you learn from the problem? What were the applied changes?

Control Measurements During Installation

- What parameters are monitored during installation of bored steel tube piles? Which of these are documented in the piling protocol?
- Based on these parameters, which do you find to be the most critical to monitor during drilling, when assessing bearing capacity and impact on the surroundings?
- How do you generally control the volume of drill cuttings for pneumatic and hydraulic drilling?
- What is the typical procedure when casting the pile and how do you measure the volume of concrete used?
- Do you use a standardized control plan for each drilling method, or does it differ depending on project requirements? Who establishes the control plan, you as a subcontractor, the main contractor, the client or consultants?

Improvements

- In what way would you like to improve the tender documents, to achieve a more "just industry" and to promote "improved quality"?
- How should the risk distribution look like, who should be responsible for what?
- How would you like the industry to work in a "proactive" way to reduce the risk of unanticipated installations effects that generate impact on its surroundings?

D

Additional Figures for the Cast-in-place Pile

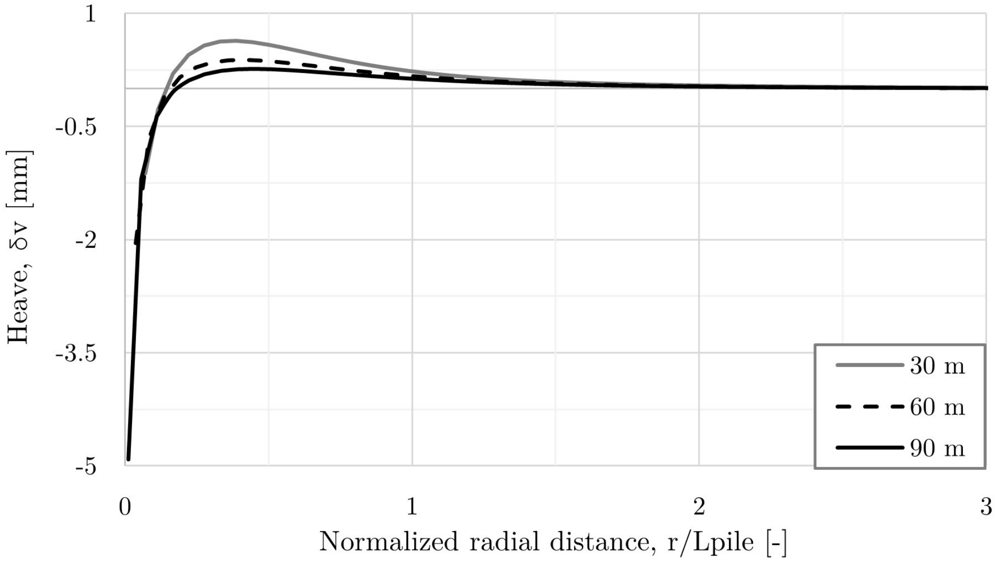


Figure D.1: The impact of pile length on the heave in LE FEM.

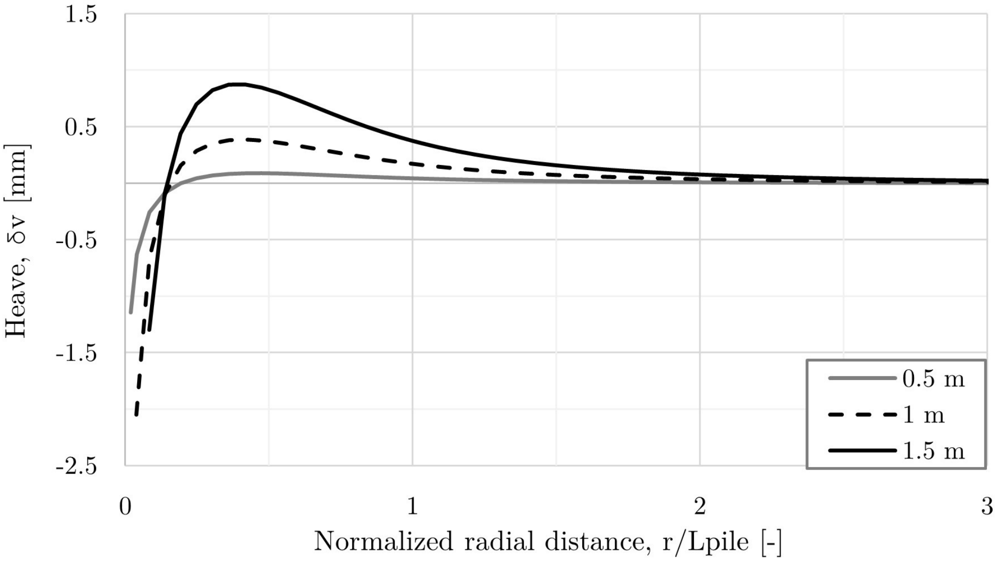


Figure D.2: The impact of pile radius on the heave in LE FEM.

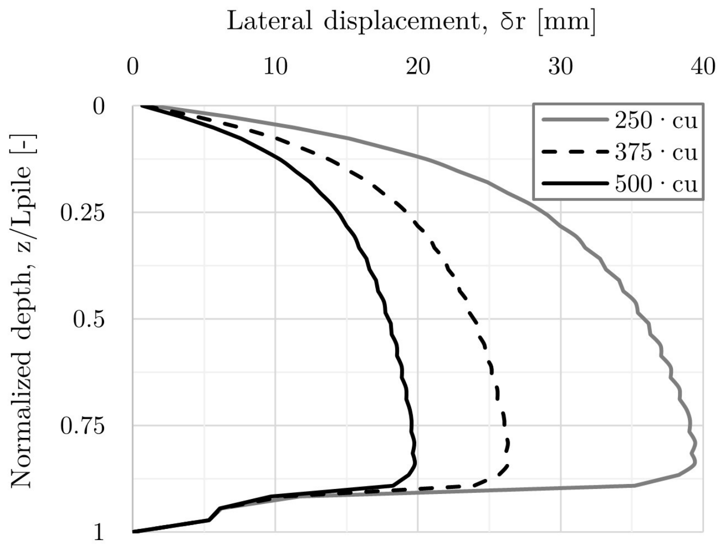


Figure D.3: The impact of elasticity modulus on the lateral displacement in LE FEM.

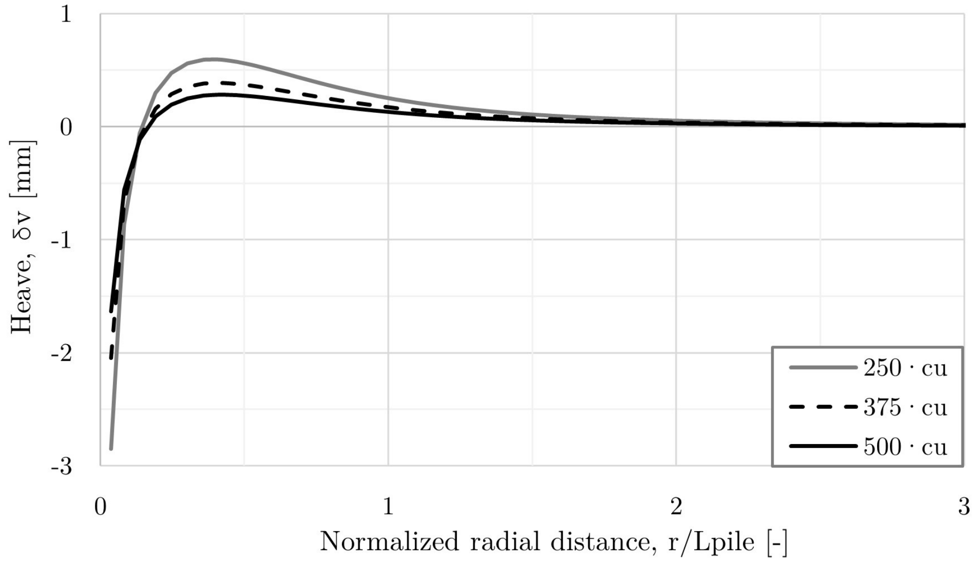


Figure D.4: The impact of elasticity modulus on the heave in LE FEM.

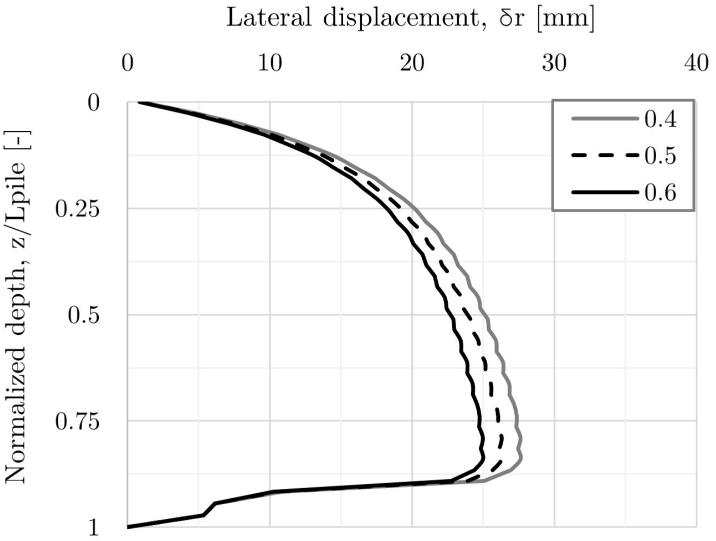


Figure D.5: The impact of K_0 on the lateral displacement in LE FEM.

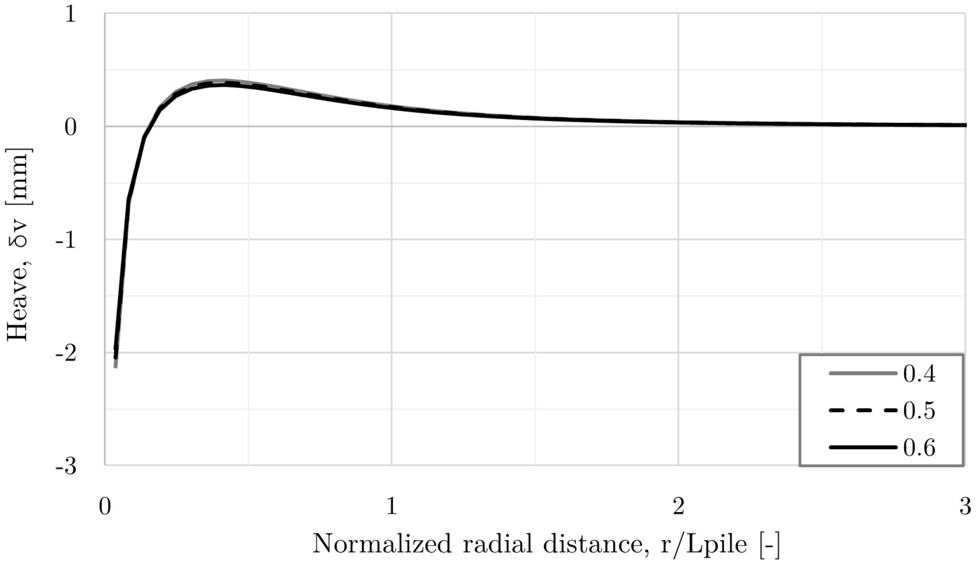


Figure D.6: The impact of K_0 on the heave in LE FEM.

E

Volume Loss Analysis

Table E.1: Calculated volume loss and corresponding V_{factor} for different drilled steel tube pile model runs during the parameter study. Here, LE stands for Linear Elastic, SS stands for Soft Soil, and G stands for geometric variation. Geometric variations of alternatives 1-4 are listed in Table 5.1 and EXP means that a volumetric expansion was applied instead of a contraction (also indicated by a negative volumetric loss).

Model	**Run**	**Pile Length [m]**	**Prescribed V_{loss} [m^3]**	**Realized V_{loss} [m^3]**	**V_{factor} [-]**
LE	LE_250c_u	60	48.25	29.96	0.621
Parameter	LE_375c_u	60	48.25	29.96	0.621
	LE_500c_u	60	48.25	29.96	0.621
SS	SS_250c_u	60	48.25	44.85	0.929
Parameter	SS_375c_u	60	48.25	38.41	0.796
	SS_500c_u	60	48.25	33.58	0.696
	SS_0c'	60	48.25	40.74	0.844
	SS_2c'	60	48.25	38.41	0.796
	SS_10c'	60	48.25	30.69	0.636
	SS_1OCR	60	48.25	45.46	0.942
	SS_1.2OCR	60	48.25	38.41	0.796
	SS_1.4OCR	60	48.25	35.75	0.741
LE	LE_G1	30	48.25	29.06	0.602
Geometric	LE_G2	60	48.25	29.96	0.621
	LE_G3	90	48.25	30.23	0.627
	LE_G4 (Alt1)	60	48.25	29.97	0.621
	LE_G5 (Alt2)	60	48.25	29.97	0.621
	LE_G6 (Alt3)	60	48.25	29.95	0.621
	LE_G7 (Alt4)	60	48.25	31.97	0.663
	LE_EXP	60	-48.25	-29.95	0.621
SS	SS_G1	30	48.25	48.09	0.997
Geometric	SS_G2	60	48.25	38.42	0.796
	SS_G3	90	48.25	39.06	0.809
	SS_G4 (Alt1)	60	48.25	47.46	0.984
	SS_G5 (Alt2)	60	48.25	48.10	0.997
	SS_G6 (Alt3)	60	48.25	42.88	0.889
	SS_G7 (Alt4)	60	48.25	32.09	0.665
	SS_EXP	60	-48.25	-32.98	0.684

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