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Long-Term Behaviour of Floating Pile Foundations in Soft Gothenburg Clay

Advanced Numerical Modelling of Creep Settlements and Soil-Pile Interaction

Master's thesis in Infrastructure and Environmental Engineering

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CHALMERS UNIVERSITY OF TECHNOLOGY
Gothenburg, Sweden 2026
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Master's thesis 2026

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Master's Thesis 2026
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Cover: Photography of the building Läppstiftet in Gothenburg at sunset [29].

Typeset in LaTeX
Printed by Chalmers Reproservice
Gothenburg, Sweden 2026

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Abstract

Urban development in the city of Gothenburg has become increasingly dependent on complex geotechnical solutions thus the high extent of deep deposits of soft clay and the ongoing densification of the urban environment. In areas subjected to long-term creep settlements, the utilisation of floating piles have become significant. Reliable predictions is therefore essential for safe foundation design.

The aim of the study was to evaluate the long-term behaviour of floating pile foundations in deep deposits of soft clay subjected to creep settlements and asses how these effects can be represented in advanced numerical analyses. The study focused on the area of Lilla Bommen in Gothenburg and was carried out in collaboration with Skanska.

Finite element analyses were carried out using both two-dimensional and three-dimensional numerical models in Plaxis. The Creep-SCLAY1S constitutive model was utilised to simulate long-term consolidation and especially creep behaviour of the clay. The numerical results were later compared to measurements of settlements and axial force observed in instrumented piles. Additional investigations were also carried out on the influence of pile spacing and pile length.

The numerical results were compared with the measured settlement data and observed axial forces. It was shown that the numerical model was able to capture long-term settlement behaviour and partially the development of negative skin friction within the floating pile system. However, differences between measured and simulated axial force distribution indicated that the simplified numerical approach may not fully capture realistic pile-group interactions effects and time-dependent load redistribution.

The additional investigations on the influence of pile spacing and pile lengths made it possible to conclude that it has high affect on both settlement behaviour and axial load distribution. Smaller spacing resulted in increased settlements and reduced pile efficiency due to pile group effects. On the other hand, the increasing pile length showed to reduce settlements. Furthermore, substantial differences were observed in the comparison between 2D and 3D analyses, particularly load distribution and the behaviour of edge piles and their effect on piles closer to the centre.

To conclude, the study demonstrates that a simplified 2D unit cell are able to represent the average behaviour of centrally located piles which can provide valuable support during design phases. However, the results further indicate that advanced 3D analyses are needed for to capture local effects, edge pile behaviour and load distribution within pile groups subjected to long-term consolidation and creep settlements. Additionally, the findings highlight the importance of creep deformation in the long-term behaviour of floating piles foundations in Scandinavian soft clay.

Keywords: Floating pile foundations, soil-pile interaction, axial force, long-term settlements, creep settlements, negative skin friction, Creep-SCLAY1S, soft clay, finite element modelling, pile group behaviour.

Acknowledgements

First of all, we would like to extend a special thank you to our supervisor and examiner, Prof. Jelke Dijkstra, for his encouragement to think and work independently. While also providing valuable guidance and insightful feedback that helped develop the study further.

We would also like to thank our supervisors at Skanska Teknik, Anders Kullingsjö, Peter Cleasson and Anna Björklund, for giving us the opportunity to carry out this thesis in collaboration with the industry. Their expertise and our engaging discussions have provided us with valuable insight into practical geotechnical engineering and important design considerations. Furthermore, the access to documentation and project material has been essential for the completion of this study.

Lastly, we would like to thank the entire Department of Geology and Geotechnics at Chalmers University of Technology. Every course and lecture throughout our education have given us the right competence to being able to conduct this thesis. In addition, the always welcoming discussions during breaks, exercises and tutorials have created an inspiring workplace. Your interest in your students is one of the most contributing factors to our success.

Marcus Isaksson and Jenny Torell Paulsson, Gothenburg, June 2026

List of Acronyms

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

CRS	Constant Rate of Strain
CSS	Current Stress Surface
CTC	Centre-To-Centre
FE	Finite Element
ICS	Imaginary Compression Surface
InSAR	Interferometric Synthetic Aperture Radar
MCC	Modified Cam Clay
NCL	Normal Consolidation Line
NCS	Normal Compression Surface
OCR	Over-Consolidation Ratio
SBUF	Development Fund of the Swedish Construction Industry *(Sveriges Byggindustriers Utvecklingsfond)*
SLS	Serviceability Limit State
SSC	Soft Soil Creep
ULS	Ultimate Limit State

Nomenclature

Below is the nomenclature of indices, sets, parameters, and variables that have been used throughout this thesis.

Indices

b	Base
c	Creep component
d	Deviatoric component
e	Elastic component
i	Intrinsic property
mi	Intrinsic isotropic preconsolidation
m	Isotropic preconsolidation (NCS)
eq	Equivalent mean stress (CSS)
s	Shaft
u	Ultimate / Undrained
v	Volumetric component
q	Deviatoric component
0	Initial value

Sets

CSS	Current Stress Surface
ICS	Imaginary Compression Surface
NCL	Normal Consolidation Line
NCS	Normal Compression Surface

Parameters

a	Destructuration parameter controlling isotropic degradation [-]
b	Destructuration parameter controlling deviatoric degradation [-]
A_b	Cross-sectional area at pile base [m^2]
A_s	Pile shaft surface area [m^2]
C_α	Creep index [-]
C_c	Compression index [-]
C_r	Recompression index [-]
C_s	Swelling index [-]
D	Pile diameter or equivalent pile width [m]
E'	Drained Young's modulus [kPa]
e_0	Initial void ratio [-]
k	Creep coefficient [-]
k_∞	Long-term creep coefficient [-]
k_x, k_y	Hydraulic conductivity in horizontal and vertical direction [m/s]
κ^*	Modified swelling index [-]
L	Pile length [m]
L_p	Embedded length of pile group [m]
λ_i^*	Modified intrinsic compression index [-]
M	Stress ratio at critical state [-]
μ_i^*	Intrinsic creep index [-]
N_c, N_{cp}	Bearing capacity factors [-]
ν'	Drained Poisson's ratio [-]
S_1, S_2	Plan dimensions of pile group [m]
τ	Reference time [s]
w_l	Liquid limit [-]
w_n	Natural water content [-]
X	Amount of bonding [-]
α	Adhesion factor; inclination of yield surface [-]
β	Ratio of compression to creep indices [-]
γ	Unit weight [kN/m^3]
ω	Rotational hardening parameter [-]
ω_d	Rotational hardening parameter (deviatoric) [-]

Variables

c_u	Undrained shear strength [kPa]
e	Void ratio [-]
$\dot{e}$	Rate of change of void ratio [-/s]
$\dot{e}^c$	Creep component of void ratio rate [-/s]
$\dot{e}^e$	Elastic component of void ratio rate [-/s]
$\dot{\varepsilon}$	Total strain rate [1/s]
$\dot{\varepsilon}^c$	Creep strain rate [1/s]
$\dot{\varepsilon}^e$	Elastic strain rate [1/s]
$\dot{\varepsilon}_v^c$	Volumetric creep strain rate [1/s]
$\dot{\varepsilon}_q^c$	Deviatoric creep strain rate [1/s]
$\dot{\Lambda}$	Viscoplastic multiplier [1/s]
p'	Mean effective stress [kPa]
p'_{eq}	Equivalent mean effective stress [kPa]
p'_m	Isotropic preconsolidation stress [kPa]
p'_{mi}	Intrinsic isotropic preconsolidation stress [kPa]
p'_{size}	Generic size parameter for yield surfaces [kPa]
q	Deviatoric stress [kPa]
Q_b	Base resistance [kN]
Q_s	Shaft resistance [kN]
Q_u	Ultimate bearing capacity [kN]
R_i	Individual pile capacity [kN]
W	Self-weight of pile [kN]
α_d	Deviatoric fabric anisotropy scalar [-]
χ	Bonding variable [-]
$\dot{\chi}$	Rate of change of bonding variable [-/s]
σ'	Effective stress [kPa]
σ'_p	Preconsolidation stress [kPa]
σ'_d	Deviatoric effective stress [kPa]
σ'_v	Effective vertical stress [kPa]

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1 Introduction

1.1 Background

In the city of Gothenburg, located on the west coast of Sweden, there is an ongoing expansion of new infrastructure and residential buildings within an already highly urbanised and densely developed environment. This development involves complex geotechnical constructions that may affect the surrounding ground and nearby buildings. Much of the city is founded on deep deposits of high compressible soft soil, where the use of floating piles has become common, both for economic reasons and to enable settlements compatible with the surrounding ground conditions [34].

The long-term performance of floating pile foundations is strongly influenced by the time-dependant behaviour of the soft clay. Following the installation, settlements develop through primary consolidation thus excess pore water pressure dissipates. However, soft clays also exhibit secondary compression known as creep. These creep deformations affect the interaction between the soil an the pile, influencing the time-dependent mobilization of shaft resistance along the pile and distributions of axial load within the pile [23]. Thereby, influencing the overall development of long-term settlements.

To account for these effects, several empirical and numerical design approaches have been developed. Nevertheless, the incorporation of creep effects in geotechnical design remains a challenge and in many cases the predicted settlements exceed those observed in-situ, leading to increased constructions costs and unnecessary material use.

In report 100 published by Pålkommissionen, there are recommendation on how to design floating pile foundations considering creep [24]. Based on these recommendations, Skanska has developed a simplified empirical calculation progress that estimates long-term settlements rather than the development and time-dependent settlements in floating pile foundation. This method is based on the assumption that creep settlements can be neglected for OCR > 1.25 and therefore, a reduction of the preconsolidation stress to around 80% is made to account for creep during design of floating piles in soft soil. Although, at large clay thickness, there is a reason to suspect that this method is overly conservative throughout the whole thickness.

Advanced numerical analyses based on the Finite Element Method (FEM) could provide a better estimate of long-term settlement development for floating pile foundations. This could be done by utilising a constitutive model that incorporates creep deformation where the soil-pile interaction can be studied further. As a result, these analyses may offer an opportunity to develop the simplified design method utilised at Skanska.

Moreover, the building Läppstiftet, located in the Lilla Bommen area of Gothenburg, is used as a reference case in this study, where the settlement development have been monitored for 14 plus years. During this period, the building has settled significantly less than the surrounding soil, resulting in negative skin friction and the structural capacity in the piles being reached. The measurement data, combined with an advanced FE model, enables an assessment of the long-term behaviour of floating pile foundations in soft Gothenburg clay. Moreover, providing a basis for optimising Skanska's assumption on a reduced preconsolidation pressure which is applied in the in-house FEM3DYN structural design tool [18].

1.2 Aim

The aim of the project is to evaluate the long-term behaviour of floating pile foundations in deep deposits of Scandinavian clay subjected to creep settlements. Furthermore, the incorporation of creep effects in simplified design methodologies will be evaluated with the focus on providing a basis for further improvements of the method currently in use at Skanska.

1.3 Objectives

The objectives of the study are:

- Establish a theoretical foundation by reviewing existing literature, reports and standards relevant to the study.
- Characterise the geotechnical site conditions in the Lilla Bommen area.
- Develop a numerical FE model for a floating pile in Plaxis using the Creep-SCLAY1S constitutive model for the soil.
- Compute a benchmark for the long-term settlements, development of axial pile loads and the soil-pile interaction of floating pile foundations.
- Investigate the influence of pile spacing and pile length on settlement behaviour.
- Provide a basis for potential improvements of Skanska's simplified design tool.

1.4 Limitations

The limitations presented in this section describe the anticipated constraints related to data, methodology and available resources. These factors may affect the precision and general applicability of the results.

- The FE model will be simplified into an axisymmetric 2D unit cell.
- The analysis will geographically be restricted to Gothenburg and the location of Lilla Bommen. The characterization of the site may affect the result and cannot necessarily be directly implemented at other locations.
- Various pile lengths will be assessed for their relevance to the area; however, the analysis must not necessarily be performed as a function of pile length.
- Installation effects are not considered in the numerical analyses.
- Primary consolidation is not further mentioned thus it is not within the scope of this study.

1.5 Methodology

This study combines literature research, site characterisation and numerical modelling to evaluate the long-term behaviour of floating pile foundations in soft Gothenburg clay.

Relevant literature were systematically reviewed to establish a theoretical foundation for the study. Geotechnical information such as field, laboratory and observational data were collected from the Lilla Bommen area. To being able to characterise the area which was also further implemented in the numerical model.

The long-term behaviour of floating pile foundations was investigated using FE modelling in Plaxis 2D and 3D [27]. The analyses were performed using the Creep-SCLAY1S constitutive model which can be utilized to simulate background creep deformations. The model is based on the Modified Cam Clay (MCC) model but includes features such as anisotropy and evolution of anisotropy, bonding and destruction [16]. Moreover, Creep-SCLAY1S is a viscoplastic model that integrates the rate-dependant attributes of soft soils [11].

Moreover, parametric studies were performed to evaluate the influence of pile spacing and length on the settlement development and axial force distribution.

1.6 Societal, ethical and ecological aspects

The project has a general goal of improving the design of deep foundations for larger infrastructure, thus benefiting the development of cities and broader societies while minimizing costs. Moreover, an improved design method for deep foundations could result in increased efficiency in construction. This would lead to fewer resources being used, as well as a reduction in the carbon footprint of the future infrastructure industry. Ethical considerations fall outside the scope of this thesis, as it is limited to a technical evaluation.

2
Theory

The following chapter presents the theoretical background relevant to the study. The focus is on time-dependant deformation in Scandinavian soft clays, the geotechnical mechanisms governing long-term development of settlements as well as its implication for deep foundations. Furthermore, the constitutive modelling approach adopted in this study is introduced, with emphasis on the Creep-SCLAY1S model.

2.1 Time dependent deformation in soft soils

Settlements results from the vertical strains that develop within the soil mass, and can be divided into elastic and inelastic components. Elastic strain occurs immediately upon loading and will recompress during fast unloading. Inelastic strain, on the other hand, is irreversible and includes time-dependent compression [20]. This irreversible compression refers to a viscous behaviour known as creep, which rapidly increases in softer soils. The total strain rate can be expressed as the sum of elastic and creep components, as shown in Equation 2.1.The same theory can be applied to the evolution of total void ratio, as shown in Equation 2.2.

$$\dot{\epsilon} = \dot{\epsilon}^e + \dot{\epsilon}^c \tag{2.1}$$

$$\dot{e} = \dot{e}^e + \dot{e}^c \tag{2.2}$$

In both formulations, the exponent e stands for elastic and c for creep, while time derivatives are denoted by a dot over the variables [15]. The elastic and creep strain rates may further be expressed using swelling, compression and creep indices, as shown in Equation 2.3 and 2.4.

$$\dot{\varepsilon}^e = -\frac{C_s}{\ln 10}\,\frac{\dot{\sigma}'}{\sigma'} \tag{2.3}$$

$$\dot{\varepsilon}^c = -\frac{C_a}{\tau \ln 10}\left(\frac{\sigma'}{\sigma'_p}\right)^{\beta}, \quad \text{with } \beta = \frac{C_c - C_s}{C_a} \tag{2.4}$$

Where C_s, C_c and C_α represent the swelling index, compression index, and creep index, respectively.

2.1.1 Normal consolidation line and stress dependency of creep

The normal consolidation line (NCL) describes the relationship between void ratio and effective stress for normally consolidated soil, i.e., when OCR = 1. Creep effects the evolution of the soil state significantly, since time-dependant compression gradually increases the preconsolidation stress. Thus, the soil becomes compressed and stiffened over time. With increased preconsolidation stress, OCR also increases, which makes the soil less prone to further creeping [20]. This relationship is illustrated in Figure 2.1.

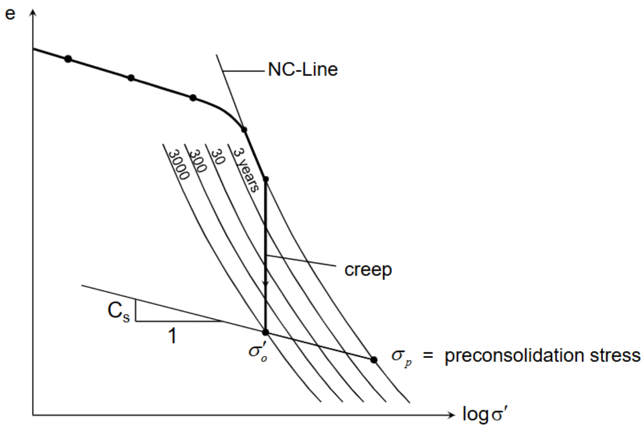

Figure 2.1: Indicating the relation between creeping and the increase of preconsolidation stress and OCR [20].

Another relationship between OCR and creep rate is shown in Figure 2.2, indicating that the creep rate decreases when increasing OCR. In fact, for soils with OCR greater than approximately 1.3, the creep rate ($\dot{e}^c$) is believed to be 1/1000 of OCR = 1 and therefore almost negligible [24]. This stress dependency is essential in long-term settlement predictions.

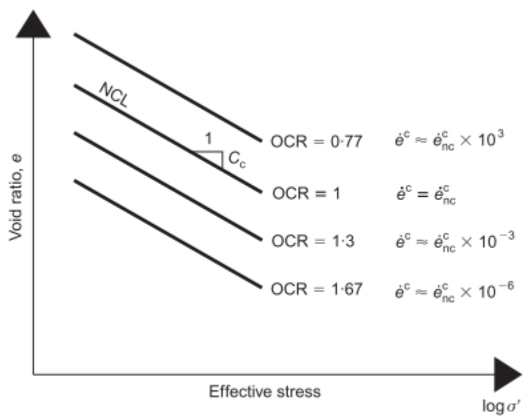

Figure 2.2: Variation of creep rate with OCR [20].

2.2 Deformation behaviour of Scandinavian soft soils relevant to creep and settlement

Scandinavian soft soils are typically of marine origin and were deposited during glacial and post-glacial periods. Different geological processes such as erosion, groundwater changes, ageing and cementation have resulted in soil deposits that are normally consolidated to slightly over-consolidated [4].

2.2.1 Stress history and preconsolidation stress

The stress history of Scandinavian soft soils plays a critical role in the determination of mechanical behaviour and deformation characteristics. The preconsolidation stress σ'_p, represents the maximum effective stress previously experiences by the soil. The over-consolidation ratio (OCR), defined as the ratio between the preconsolidation stress and the current vertical effective stress, provides an indication on the stress history of the soil.

When additional stresses are applied to the soil and the effective stress approaches the preconsolidation stress, the soil response transitions from elastic to plastic behaviour. This results in increased compressibility and time-dependant deformation. Oedometer testing has shown that creep rates increase significantly when the effective stress approaches the preconsolidation stress [2]. Consequently, an accurate assessment of the preconsolidation stress is critical for predicting long-term settlements in Scandinavian soft soils.

2.2.2 Compressibility and time-dependant deformation (creep)

The magnitude and rate of settlements are strongly linked to the compressibility of Scandinavian soft soils when subjected to loading. These soils are characterised by high water content and low effective stresses, which together explain their high compressibility. Primary compressibility can be described using the compression index C_c, while recompression behaviour is commonly represented by the recompression index C_r, or swelling index C_s, which are typically determined through Oedometer testing [6].

Due to the normally consolidated to lightly over-consolidated nature of Scandinavian soft soils, relatively large primary settlements may develop when subjected to loading. However, these soils also exhibits significant time-dependent deformation after excess pore water pressures have largely dissipated [2]. This behaviour is refereed to as creep or secondary compression and can be described using the creep index C_α [6]. Laboratory testing has shown that creep strains may accumulate over longer periods even during constant effective stresses. Consequently, settlements in Scandinavian soft soils are often governed not only by primary consolidation but often also by secondary compression.

2.2.3 Natural structure, bonding and destructuration

Scandinavian soft soils are natural marine deposits characterised by a distinct soil structure. This structure consist of a soil fabric, which is associated with inter-particle bonding [35], and provides the soil with an apparent strength and stiffness exceeding those of an equivalent reconstituted material at the same void ratio [5]. Furthermore, the fabric is often anisotropic, influencing both elastic and plastic soil behaviour.

When the soil is subjected to loading, plastic strains starts to modify the anisotropy and degrade the apparent bonding, a process referred to as destructuration [22]. This degradation leads to reduced stiffness and shear strength, as well as increased compressibility and time-dependant deformation. Although, at sufficiently high vertical effective stresses, the behaviour of fully destructured soil tends to converge towards that of the initially bonded soil[21].

The sensitivity of many Scandinavian soft soils further highlights the importance of natural structure. Large reductions in strength upon remoulding indicate that the mechanical behaviour is strongly influenced by both bonding degradation and intrinsic soil properties [5]. Sensitivity is defined as the the ratio between undisturbed and remoulded undrained shear strength and provides an indication of the potential for strain softening.

2.2.4 Shear strength and deformation characteristics

The shear strength of Scandinavian soft soils is one of the controlling parameters governing the soil's ability to resist deformation under loading. Undrained shear strength is commonly applied in soft soil analyses due to the low hydraulic conductivity, which prevents rapid dissipation of excess pore water pressure during loading. Therefore, leading to prolonged consolidation processes and significant time-dependant settlement. Typical undrained shear strength values in Scandinavian soft soils increase with depth and over-consolidation ratios [34, 35, 31].

The shear strength and deformation characteristics are "dependent on effective stresses, stress history and the rate of deformation" [19]. The stress-strain response may be determined by a Constant Rate of Strain test (CRS). Using such test, Sällfors showed that an increase in strain-rate lead to higher effective vertical stresses [26], which illustrates the rate-dependant behaviour typical for Scandinavian soft soils.

2.3 Constitutive modelling of creep behaviour

Accurate prediction of long-term settlements in soft clay deposits requires numerical analyses capable of reproducing the coupled stress-dependant and time-dependant soil behaviour. This places high demands on the constitutive model, as it must provide a realistic representation of creep mechanisms and the evolving soil structure.

Simplified elasto-plastic perfectly plastic models, such as Mohr-Columb, are not suitable for describing the stress-strain behaviour of normally consolidated to slightly over-consolidated soft clays, as emphasized by Karstunen [16]. Instead, more advanced elasto-plastic hardening or models are required. Furthermore, for long term settlement predictions, a rate-dependent constitutive formulation is necessary. Models such as Soft Soil Creep (SSC) or Creep-SCLAY1S may be used for this purpose. However, studies have shown that SSC may over predict creep settlements [16]. Therefore, the Creep-SCLAY1S is adopted in this study.

2.3.1 Creep-SCLAY1S model formulation

Creep-SCLAY1S is a critical state viscoplastic model based on Modified Cam Clay (MCC), but includes additional features such as anisotropy and evolution of anisotropy, bonding and destruction [16]. The model adopts a hierarchical framework analogous to the original elastoplastic model S-CLAY1S. The details of S-CLAY1S are not discussed further, see Wheeler et al. [33].

The Creep-SCLAY1S model is formulated such that total strains can be divided into elastic (e) and creep (c) components, consistent with Equation 2.1. Furthermore, the model is defined using three surfaces describing the soil state [12]. These are the Normal Compression Surface (NCS), that forms the boundary between small and large creep strains, the Current Stress Surface (CSS), that represents the current stress state of the soil, and the Imaginary Compression Surface (ICS), that accounts for bonding effects as proposed by Gens and Nova[9]. The three surfaces hold the same shape and orientation but differ in size.

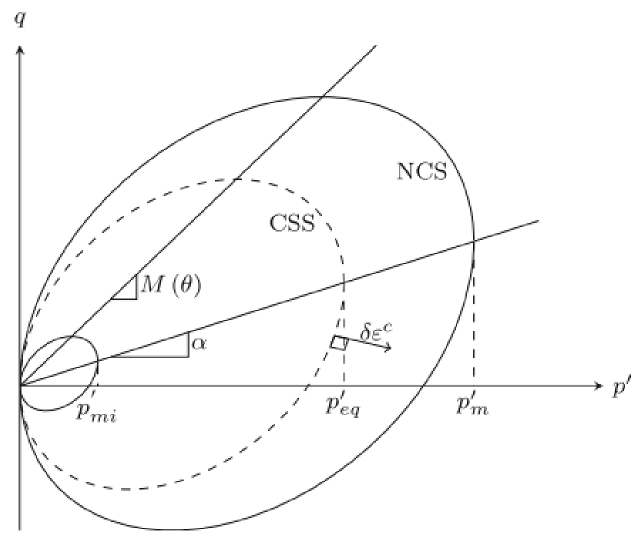

Figure 2.3: Yield surfaces introduces in the Creep-SCLAY1S model [16].

As illustrated in Figure 2.3, the intersection between the vertical tangent to the NCS ellipse and p'-axis defines the isotropic preconsolidation stress p'_m. The corresponding parameter for the CSS ellipse is refereed to as the equivalent mean stress p'_{eq}, while the ICS ellipse is defined by the intrinsic isotropic preconsolidation stress p'_{mi}. The size of the surfaces are defined by the following Equation 2.5.

$$p'_{\text{size}} = p' + \frac{(q - \alpha p')^2}{(M^2(\theta_\alpha) - \alpha^2)\, p'} \tag{2.5}$$

p'_{size} represent p'_m, p'_{eq} or $p'_m i$, respectively. The difference in size between the NCS and ICS line is further defined by the following equation $p'_m = p'_{mi} * (1 + X)$ where X is related to the current amount of bonding. Furthermore, α is a scalar that describes the inclination of the yield surface and M is the stress ratio at critical state that depends on the Load angle θ.

The model assumes an associated flow rule to keep the model user-friendly and numerically robust [16]. As a result, the creep strains are represented as in Equation 2.7.

$$\dot{\varepsilon}^c_v = \dot{\Lambda}\, \frac{\partial p'_{eq}}{\partial p'} \qquad \text{and} \qquad \dot{\varepsilon}^c_q = \dot{\Lambda}\, \frac{\partial p'_{eq}}{\partial q}, \tag{2.6}$$

$$\text{where} \quad \dot{\Lambda} = \frac{\mu^*_i}{\tau} \left(\frac{p'_{eq}}{p'_m} \right)^{\beta} \left(\frac{M_c^2 - \alpha^2_{K_0}}{M_c^2 - \eta^2_{K_0}} \right) \tag{2.7}$$

2.3.2 Hardening laws and destructuration

Since the Creep-SCLAY1S model is formulated using three yield surfaces, three corresponding hardening laws are introduced [13]. These include:

(i) a volumetric hardening law controlling the evolution of the intrinsic preconsolidation pressure (Equation 2.8),

(ii) a rotational hardening law describing the evolution of the anisotropy (Equation 2.9), and

(iii) a destructuration law governing the degradation of bonding (Equation 2.10).

$$\dot{p}'_{mi} = \frac{p'_{mi}}{\lambda^*_i - \kappa^*} \dot{\varepsilon}^c_v \tag{2.8}$$

$$\dot{\alpha}_d = \omega \left(\left[\frac{3\sigma'_d}{4p'} - \alpha_d \right] \langle \dot{\varepsilon}^c_v \rangle + \omega_d \left[\frac{\sigma'_d}{3p'} - \alpha_d \right] |\dot{\varepsilon}^c_d| \right) \tag{2.9}$$

$$\dot{\chi} = -a\chi \left[|\dot{\varepsilon}^c_v| + b\dot{\varepsilon}^c_q \right] \tag{2.10}$$

Through these coupled mechanisms, the model combines viscoelastic creep with rotational and volumetric hardening as well as destructuration. Unlike simpler constitutive models, Creep-SCLAY1S captures both stress-dependency and evolving anisotropy and bonding. As a result, enabling a more accurate representation of key mechanism governing long-term behaviour in soft clays.

2.3.3 Comparison between FEM3DYN and Creep-SCLAY1S

Per Kettil associate professor at Chalmers University of Technology, has together with Skanska developed an FEM program using MATLAB named FEM3DYN, which adapt a simplified representation of the creep process in the pile foundation design phase [18]. This simplification is based on the assumption that creep settlements are negligible for OCR values above 1.3. The method introduces a creep coefficient k defined as the inverse of OCR, Equation 2.11.

$$k = \frac{1}{\text{OCR}}x \tag{2.11}$$

For fully consolidated conditions, the coefficient becomes $k_\infty = \frac{1}{1.3} = 0.77$, which is used as a reduction factor of the preconsolidation stress. A reduction to 0.77 is for $t = \infty$ and for each project individually, this coefficient is evaluated based on design life, hydraulic conductivity and the length of drainage path. For a bridge ramp to Hisingsbron in Gothenburg, Skanska used FEM3DYM and a creeping coefficient of 0.81 [17]. In this project FEM3DYN was used to calculate the final settlements, and the creep coefficient was calibrated and verified with the help of Geosuite [32].

2.4 Deep foundations in Scandinavian soft soil

The need for deep foundations has become increasingly significant in densely populated areas where construction often takes place on soft clay deposits. The main purpose of a foundation is to transmit structural loads to the underlying soil, which culminates in a soil-structure interaction. The selection of an appropriate foundation solution depends on both site conditions and construction characteristics.

In areas with thick deposits of soft soil such as clay, pile foundations have historically been widely used. In such cases, loads are transferred to deeper and older soil layers, which generally holds both higher strength and stiffness due to their stress history [1].

2.4.1 Pile foundations and interactions with soil

Pile foundations can be designed in several different ways depending on the characteristics of the project. Piles may be categorized based on material type, installation method, soil conditions and loading type [1]. A common classification is based on the load transfer mechanism, where axial forces are resisted through shaft resistance, base resistance, or a combination of both [8].

2.4.2 Loading-bearing capacity of pile foundations

The ultimate bearing capacity of a pile can be expressed as the sum of the ultimate shaft resistance and ultimate base resistance, reduces by the self weight of the pile, as described in Equation 2.12 [8].

$$Q_u = Q_s + Q_b - W \tag{2.12}$$

Here the subscripts s and b denote for ultimate shaft resistance and ultimate base resistance respectively. W refers to the weight of the pile. If the bearing capacity of a foundation is insufficient, either the resistance must be increased or the axial force reduced. The latter may be achieved by increasing the number of piles in the foundation and thereby reducing the centre-to-centre (CTC) spacing. This distributes the total applied shear force among a larger number of piles, resulting in a lower shear force acting on each individual pile.

2.4.3 Base resistance and end bearing piles

Base resistance refers to the resisting force that the end of a pile supports, as described in Equation 2.13.

$$Q_b = A_b(N_c c_u + \gamma L) \tag{2.13}$$

Where A_b refers to cross sectional area, N_c as a factor of the pile embedment in the stratum between 6 and 9, and γL represent the total weight of the soil at the pile base level. Although γL is often assumed to be relatively equal to the weight of pile W. Consequently, both the weight of the soil and pile are often neglected in practice [8].

Piles for which the base resistance is significantly larger than the shaft resistance, $Q_b >> Q_s$, are referred to as end bearing piles. Such piles reach a firm soil layer or bedrock and mobilise the majority of axial forces through end resistance rather than shaft resistance. End-bearing piles are often used for thinner deposits of soil or for structures that require high bearing capacity.

2.4.4 Shaft resistance and floating piles

Skin friction refers to the mobilised axial force along the pile shaft. In undrained soft soils, this interaction may be described using an adhesion factor α and the undrained shear strength c_u along the pile shaft, as shown in Equation 2.14.

$$Q_s = A_s \alpha c_u \tag{2.14}$$

Floating piles, also referred to as shaft-bearing piles, does not reach a firm stratum but instead transfer loads primarily through shaft resistance. These piles are commonly installed in groups and are often used in deep soft soil deposits where end bearing piles would be economically unfavourable.

2.4.5 Negative skin friction and neutral plane

Background settlements may develop due to several mechanisms, such as groundwater lowering and creep deformation. When the surrounding soil settles more than the pile, downward shear stresses may develop along the the upper part of the pile.

This results in negative shaft resistance, where the soil drags the pile downward rather than providing resistance [1]. This additional load is referred to as negative skin friction and can have a substantial effect on pile settlements and the structural load carried by the pile. This negative skin friction is an important effect to take into considerations when designing piles structural capacity, both for end- and shaft bearing piles.

In floating piles, the lower part of the pile settles more then the surrounding soil, which results in shaft resistance mobilised along the lower part of the pile. The depth at which the pile and the surrounding soil experience equal settlements is referred to as the neutral plane. The determination process of the neutral place can be seen in Figure 2.4 were action effect represent the development of load, being the total load of forces at the pile head and negative skin friction. Meanwhile resistance represent the positive shaft resistance and bearing of the total load of the pile [1].

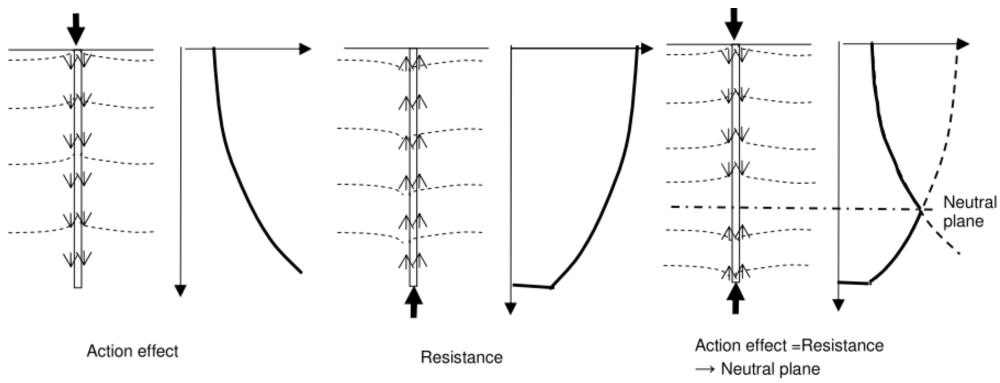

Figure 2.4: Determination of neutral plane using axial force and resistance [1].

In order to mobilise full friction or cohesion forces, a slip movement between the pile and ground of at least 2 - 5 *mm* is required. In some cases, the relative movement may be insufficient, for example if the surrounding soil settles at the same rate as the pile or if settlements stagnate at a certain depth due to stress history. Under such conditions, no additional drag load is generated nor any resistance. This results in a zone of equal strain as seen in Figure 2.5, where there is no relative movement between the pile and soil.

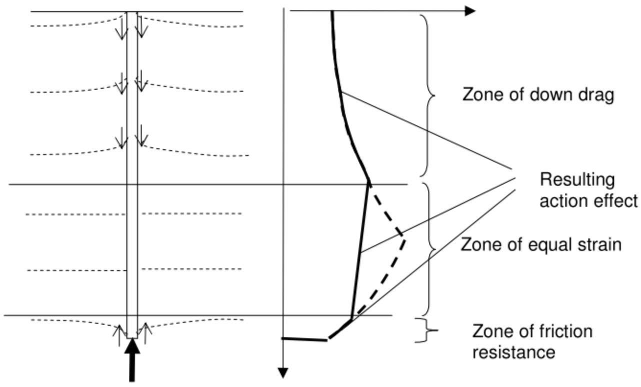

Figure 2.5: Condition of equal strain with not sufficient slip movement [1].

2.4.6 Single versus group behaviour of piles

The bearing capacity of a pile group is strongly influenced by the spacing between the piles [24]. If the spacing is sufficiently small, the group may behave as a block foundation rather than a set of individual piles. Thus, the soil located between the piles contributes less to the load distribution and will rather deform together with the whole structure.

The transition between single-pile behaviour and group behaviour is not always distinct, although it is of high importance since the governing failure mechanism differs. A pile group may fail either through individual pile failure thus the capacity is exceeded, or through a global block failure. The former is typically associated with larger pile spacing while the latter is more likely due to smaller spacing.

The presence of group effects may be assessed by comparing the ultimate bearing capacity of the pile group to the sum of the individual pile capacities [24].If the group capacity is lower than the sum of the single-pile capacities, failure will most likely occur alongside the periphery of the pile group. This mechanism may be evaluated using Equation 2.15.

$$2(S_1 + S_2)L_p c_u + N_{cp} S_1 S_2 c_u \leq \sum_{i=1}^{p} R_i \tag{2.15}$$

Where R_i is the sum of the single-pile capacities. Furthermore, S_1 and S_2 are the plan dimensions of the pile group which are specified in Figure 2.6 as well as the evolution of failure alongside the periphery.

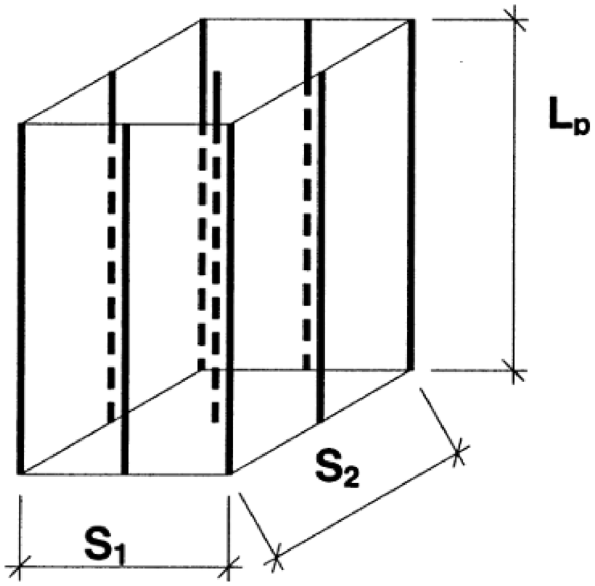

Figure 2.6: Failure mechanism alongside the periphery at small pile spacing [24].

3
Case Study Lilla Bommen

The case study area, Lilla Bommen, is located in the central part of Gothenburg adjacent to the Göta river. The surrounding area has expanded in recent decades, with both commercial buildings and major infrastructure projects. Such developments require careful geotechnical consideration, as the combination of high structural loads and soft soil conditions poses significant challenges for foundation design. Further development is planned as a part of the city's strategy to expand the city centre and connect the two sides of the river. Moreover, the deterioration of the old Göta river bridge have effectively created opportunities for new construction. Consequently, the findings of this study can provide a valuable basis for these future developments.

3.1 Geology of the city of Gothenburg

The fine grained soils in Gothenburg was formed during and after the Ice sheets from the Ice age started to melt in the area 13,000 years ago. During this time, the sea level was approximately 95-100 m higher than today [4]. Since then an isostatic postglacial rebound effects have risen the land, and for 10,000 years ago the shoreline was approximately 25 m above the current level.

During this period of melting of the ice sheet, large amounts of meltwater ran through the valley of the Göta river. This glacial meltwater also contained significant amounts of glaciofluvial material, both coarser and finer materials. Sediments of larger grain sizes were deposited close to the boundary of the ice sheet, while finer materials were further carried and deposited closer to the shoreline [4]. This is explained by the streams slowing down and mixing with saline water, leading to the settling of finer particles on the bottom of the river in the surrounding Gothenburg region.

3.2 Land reclamation of Lilla Bommen

Lilla Bommen is an important historic area located in the centre of Gothenburg city near the Göta river. The area originally consisted of meadow but was fill placed during dredging operations in the river at the beginning of the 19th century. These filling activities continued until approximately 1865 and was thereafter exploited as seen in Figure 3.1. Today the filling consist of a mix of waste and dredged material and is approximately 3.5-5 m in depth [7].

In recent years, the area has been heavily constructed with commercial buildings along the quay and Hisingsbron, which is one of the few connectors between the southern and northern part of the city and an important infrastructure corridor for the city of Gothenburg.

Figure 3.1: Historical map from 1790 with an overlay of present infrastructure [10].

3.3 Geotechnical conditions

Due to the geological conditions and history of land reclamation, the area of Lilla Bommen consists of deep deposits of mostly homogeneous soft clay, reaching up to around 100 m in depth. This poses significant geotechnical challenges when constructing foundations for tall and heavy buildings. The following geotechnical basis is primarily collected from Skanska's archive and summarisation of Claesson's work [7]. The presented boreholes are located in Lilla Bommen as well as a comparison with Lundbystrand.

3.3.1 Soil profile and index properties

In addition to the dredged fill material, the subsoil within the area mostly consist of a relatively homogenous clay deposit extending to depths from 80 to 100 m. The uppermost layers contain more organic-rich material and can locally be described as organic mud. At around 20 m depth, a siltier and stiffer clay layer is encountered. Below this level, they clay is of mostly postglacial origin. A typical section of the soil profile is presented in Figure 3.2.

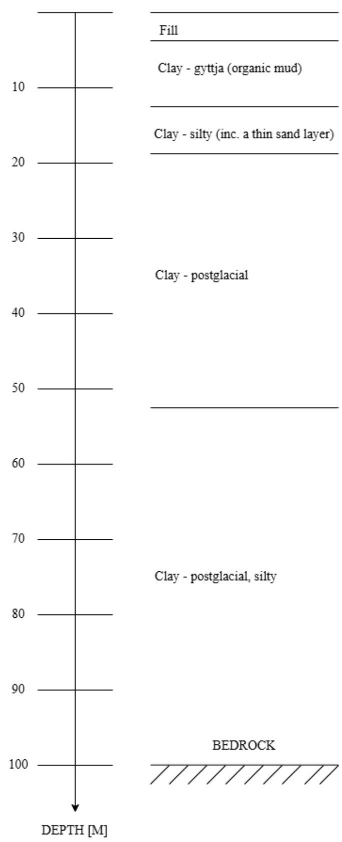

Figure 3.2: Typical section of the soil profile found at Lilla Bommen.

The profiles of water content w_n, liquid limit w_l and unit weight γ is illustrated in Figure 3.3. The results indicate that the water content varies significantly with depth, generally ranging between 55 and 80 percent. At several depths, the measured water content approaches the liquid limit. This suggest that the soil exhibits a relatively high liquidity and may be sensitive to disturbance. The unit weight remain relatively constant, typically between 16 and 17 kN/m^3, which is a characteristic of saturated clay deposits and indicates limited variation in bulk density.

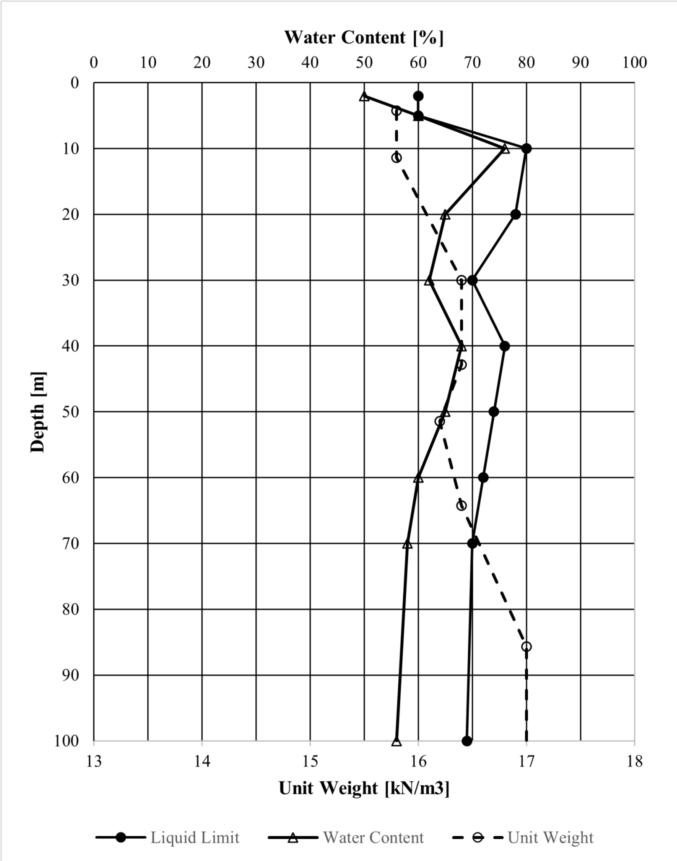

Figure 3.3: Evaluated index properties of Lilla Bommen.

3.3.2 Pre-consolidation stress and OCR

Constant Rate of Strain (CRS) tests from the site investigation was evaluated, and the in-situ effective vertical stress and were plotted together with the evaluated preconsolidation stress. The CRS tests results indicate on a relatively low pre-consolidation stresses, which may be imply sample disturbance, particularly in the upper metres of the soil profile. This can also be seen in the CRS curves, as typical indications of a disturbed samples are observed, being gradual increase in strain up to the pre-consolidation stress and a poorly defined σ'_p [25]. These aspects were taken into consideration in the interpretation of the test results and in the establishment of the preconsolidation stress trend seen, together with the variation of OCR with depth, in Figure 3.4.

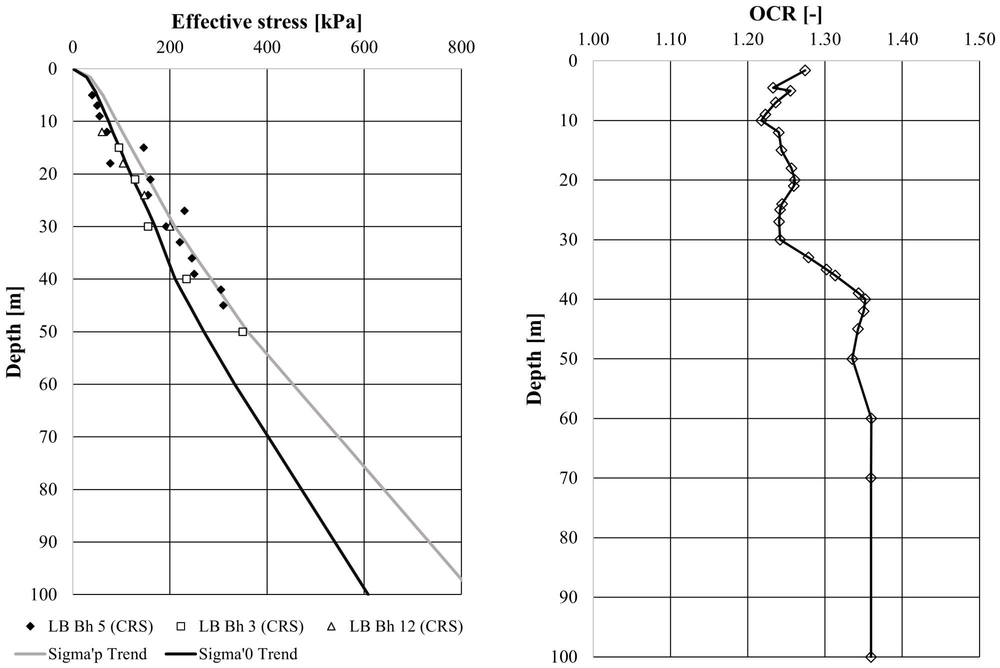

Figure 3.4: Effective Stress vs preconsolidation stress from CRS with an evaluated trend and further calculated OCR.

The analyses of the effective vertical stress and over-consolidation ratio (OCR) shows that the preconsolidation pressure exceeds the current effective vertical stress only marginally. This results in an OCR between 1.2 and 1.4 which is an indication on that the clay is generally slightly over-consolidated. The observed shear strength is also closely linked to the consolidation state of the clay. This is presented in Figure 3.5.

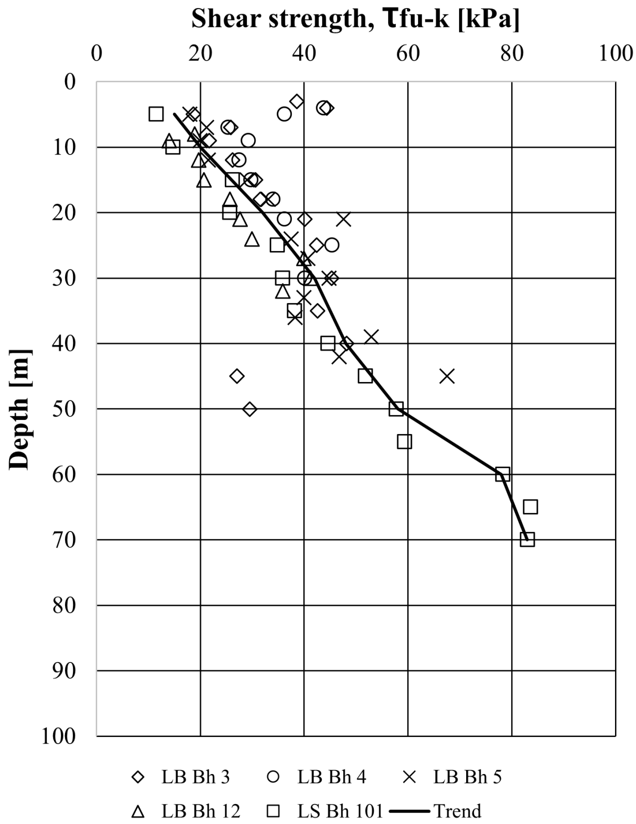

Figure 3.5: Characteristic shear strength of the soil.

There is a slight decrease in shear strength with depth in the upper most layers which can be interpreted as a consequence of diminishing structural and bonding effects within the clay as the overburden stress increases. Below these layers, the OCR stabilizes as well as the undrained shear strength increases.

3.3.3 Hydrogeological conditions

The case study area is hydraulically connected to the Göta river, where the groundwater table is located at an elevation of + 10.1, or slightly higher in direct proximity to the river [7]. Pore water pressure measurements was carried out in 1986 at Lilla Bommen. These measurements are presented together with additional pore pressure data from Lundbystrand, as well as a theoretical hydrostatic pore water pressure distribution, in Figure 3.6.

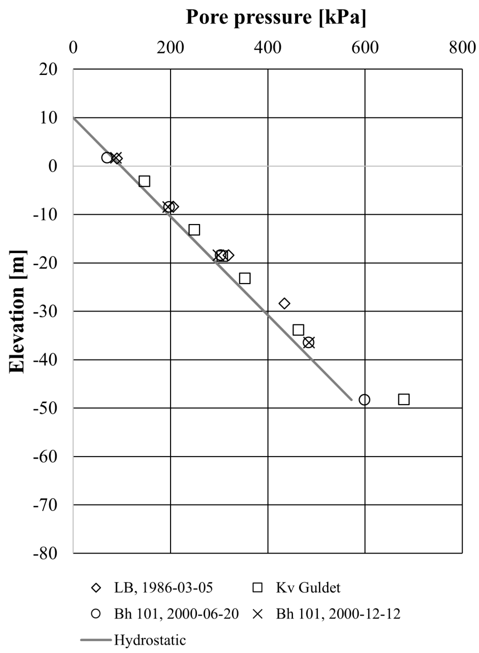

Figure 3.6: Pore pressure measurements from sites in central Gothenburg in comparison to the hydrostatic.

As shown in the figure, the measured pore water pressure profile for Lilla Bommen follows a near linear increase with depth and is in general consistent with the hydrostatic distribution. The additional measurements from the area exhibit a similar trend, although minor deviations from the hydrostatic line can be observed at greater depths. Overall, the results indicate that the groundwater is highly governed by hydrostatic conditions and somewhat influenced by the hydraulic connection to the river.

The hydraulic conductivity profile for borehole 101, from Lundbystrand, is presented in Figure 3.7. The results show generally low hydraulic conductivity values throughout the investigated soil profile, ranging from 10^{-10} to 10^{-9}. The upper part of the profile exhibits slightly higher hydraulic conductivity compared to the deeper sections. With increasing depth, the conductivity generally deceases. These low values are consistent for fine grained soil such as clay, and indicate limited groundwater flow throughout the deposit.

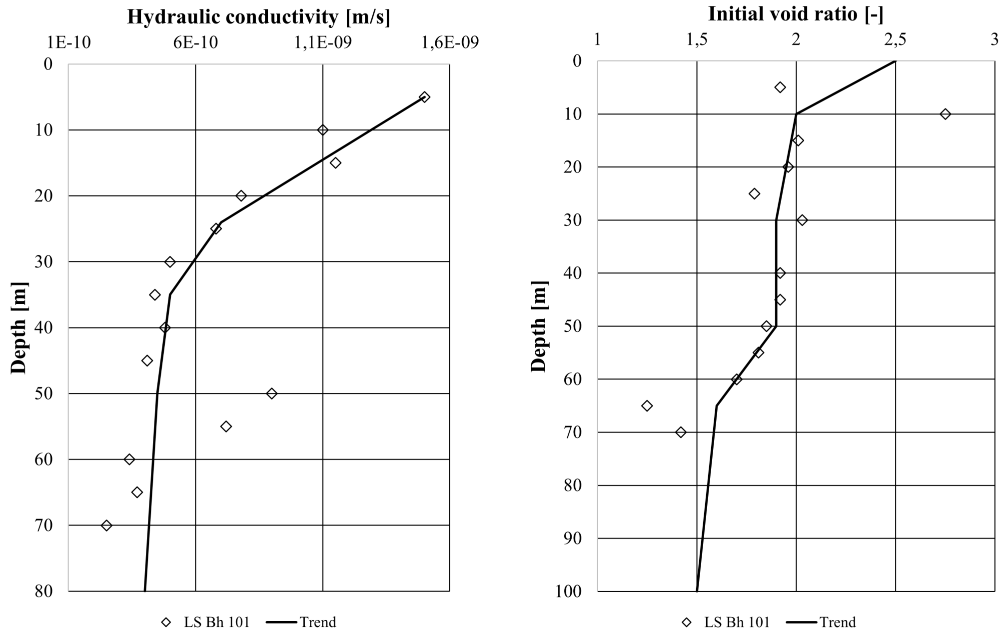

Figure 3.7: Hydraulic Conductivity and initial void ratio properties at site.

The overall low hydraulic conductivity supports the previously described pore pressure conditions, where the groundwater regime appears largely hydrostatic and is somewhat controlled by the hydraulic connection to the Göta river. The limited hydraulic conductivity suggest that excess pore water pressure will dissipate slowly, which is of high importance for consolidation and long-term settlements predictions.

3.3.4 Bellow hose measurements

In the adjacent Göta Tunnel project, Skanska installed a bellow hose in 1996 to monitor soil behaviour over several years. A bellow hose is an instrument used to measure ground movements as a function of depth over time. The measurements were collected approximately 300 m from the Göta river and are assumed to be representative of conditions within the case study area. The bellow hose measurements were performed from 24 May 1996 and are presented in Figure 3.8.

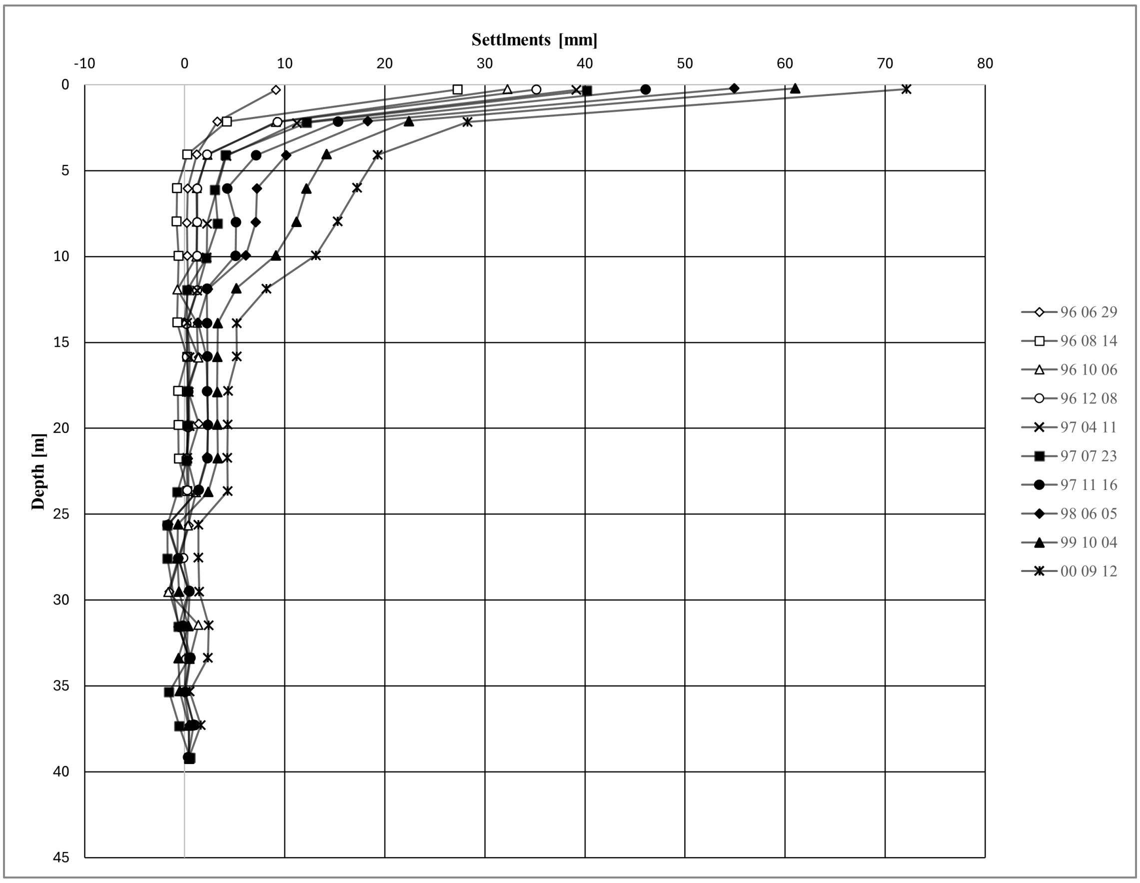

Figure 3.8: Bellow hose measurement from Göta Tunneln, located approximately 300 m from the Göta river.

3.4 Läppstiftet

One of the commercial buildings in the area is the 82 m (22-storey) office tower called Lilla Bommen or Läppstiftet, constructed between 1987 and 1989 with Skanska as one of the main contractors. The building is located adjacent to the former Göta River Bridge, which has since then been replaced by Hisingsbron. The structure is founded on prefabricated concrete floating piles installed in deep soft clay deposits. At the time of the construction, there was limited experience in the Gothenburg area regarding the long-term performance of such foundation solutions subjected to significant loading. Thus, a project funded by the Development Fund of the Swedish Construction Industry (SBUF) was initiated to investigate the floating piles performance and behaviour under load [7]. Measurements were conducted over a 14 year period which includes analyses of the deformation within the piles, the extent of settlements for the building as well as surrounding ground surfaces. The findings in this project lay as background for further investigation within this case study.

3.4.1 Permanent structure

The building complex is supported by a total of 571 prefabricated concrete piles, all with a standard cross-section of 275 $mm \times 275\ mm$. The pile lengths vary across the foundation, reflecting the geometry of the structure and the distribution of loads. The building complex consists of a 22 storey high-tower connected to a lower adjacent structure of approximately 5-6 storeys. The piles supporting the lower structure (337 units) have an average length of 44 m while the piles founded underneath the high-tower (234 units) reaches lengths up to approximately 85 m. Here, the perimeter piles are generally shorter than the central piles, with lengths ranging from approximately 65-78 m. The total footprint area of the entire building is approximately 4100 m^2, of which the high-tower accounts for around 1050 m^2. Approximately 4 m of fill was excavated during the construction of the high tower for the placement of the basement. Resulting in a placement of the pile head and concrete slab at the first clay layer.

3.4.2 Instrumentation

Bolts where installed evenly across the buildings footprint and in connection to the surrounding ground. Analyses have primarily been conducted on the bolts in the building while the others have been neglected. Other internal and external investigation have been conducted on the surrounding area as the development has progressed in the area.

To investigate the load development along the pile length, eight piles were equipped with a telltale measurement system. Four pipes of different lengths were installed along the outer surface of each pile. The pipes were arranged such that each pipe terminates at a distinct depth corresponding to approximately 1/4, 2/4, 3/4 and full pile length. As a result, measurement points A-D represent an incremental increase of one quarter of the total pile length. However, after installation only 3 piles had all intact measurement points.

4

Numerical Modelling of Lilla Bommen

The Plaxis 2D model is constructed such that it should capture realistic soil behaviour, particularly the rate of background creep settlements, while also simulating the introduction of a pile that is loaded and left to consolidate. The reference of Lilla Bommen and Läppstiftet was used as a baseline for the construction of the numerical model. Moreover, have several sensitivity analyses been performed to confirm its adaptation to reality. The constitutive model utilised for each clay layer is the Creep-SCLAY1S. The adopted model parameters are collected from Tornborg's [31] previous work that is believed to be highly representative for the area. Although, an extra sensitivity analysis of the creep index have been performed to establish its impact on the results. This is presented in Appendix A. Additional model parameters are concluded by calibration against the site specific parameters, as presented in Chapter 3. The site specific model parameters for each clay layer is presented in Table 4.1.

Table 4.1: Input parameters for Creep-SCLAY1S clay layers.

Layer	Elevation $[m]$	γ $[kN/m^3]$	OCR [-]	e_0	$k_x = k_y$ $[m/s]$
1	+7.7 to +3.7	15.8	1.15	2.34	1.30×10^{-9}
2	+3.7 to -1.3	15.8	1.25	2.34	1.30×10^{-9}
3	-1.3 to -6.3	16.1	1.3	1.99	9.65×10^{-10}
4	-6.3 to -21.3	16.3	1.35	1.92	5.90×10^{-10}
5	-21.3 to -31.3	16.4	1.4	1.92	4.59×10^{-10}
6	-31.3 to -41.3	16.5	1.4	1.89	6.55×10^{-10}
7	-41.3 and below	17.0	1.45	1.55	4.20×10^{-10}

Firstly, it is necessary to determine whether the problem should be analysed using a plane strain or an axisymmetric formulation. The plane strain assumption implies that the geometry extends infinitely in the out-of-plane direction [28]. In contrast, the axisymmetric formulation enables the analysis of a three-dimensional problem using a two-dimensional representation, provided that the geometry and loading conditions are rotationally symmetric around a central axis. Since this can reasonably be assumed in this case study, an axisymmetric approach is chosen. The discretisation of the soil cluster requires the selection of an appropriate element type, specifically whether 6-noded or 15-noded triangular elements should be applied. 6-noded elements are generally suitable for relatively simple deformation analyses, where a moderate level of accuracy is sufficient. However, they are less appropriate for axisymmetric analyses or problems involving failure development, as they tend to over-predict failure loads and safety factors [28]. For this reason, 15-noded elements are adopted in this study which most often is the default option. The use of 15-noded elements improves the accuracy of the analysis for more complex problems.

4.1 Modelling background creep effects

To initialise the development of creep within the model and clay deposit, an external load in form of a fill is applied at the top of the model. This represents the dredged material from the Göta river that was used for land reclamation in 1860. The dredged clay layer is modelled using the Mohr-Coulomb constitutive model to avoid numerical instabilities that may arise when using the more advanced creep model Creep-SCLAY1S. Furthermore, creep behaviour within the dredged clay is not of interest, as the majority of this material is likely to have been excavated during construction. Consequently, strains and displacements are not evaluated in the dredged material, the layer in included only to allow the underlying clay layers to consolidate realistically and to reproduce a representative stress history of the numerical model. The soil parameters for the fill are listed in Table 4.2.

Table 4.2: Model parameters for the dredged material.

Soil model	γ $[kN/m^3]$	E' $[kN/m^2]$	ν' [-]	c' $[kPa]$	φ' $[\circ]$	$k_x = k_y$ $[m/s]$
Mohr-Coulomb	15.0	12×10^3	0.2	10	35	1.30×10^{-9}

4.1.1 Bellow hose calibration

To being able to verify the clay layers representation of reality, the development of settlements were compared to the bellow hose measurements collected from Göta Tunneln (see Figure 3.8). Since the bellow hose is not installed in the exact same position as the high-tower of Läppstiftet, the thickness of the dredged material can not be exactly confirmed. Both locations should share similar soil properties, though it is expected to have different creep rates and development of settlements thus the thickness varies in the area. The affect of this was investigated by letting different thicknesses of fill consolidate, without any surrounding installations, up until 1996-06-29 when the bellow hose was installed. Thereafter, several consolidation steps are introduced to directly compare the development of settlements. The results showed very good agreement with the measured data when assuming a fill thickness of 2.5 m, as illustrated in Figure 4.1. However, increasing the fill thickness up to 5 m within the model produced similar results, with slightly increased settlements in the upper most layers. The maximum deviation from the measured values was approximately 10 mm.

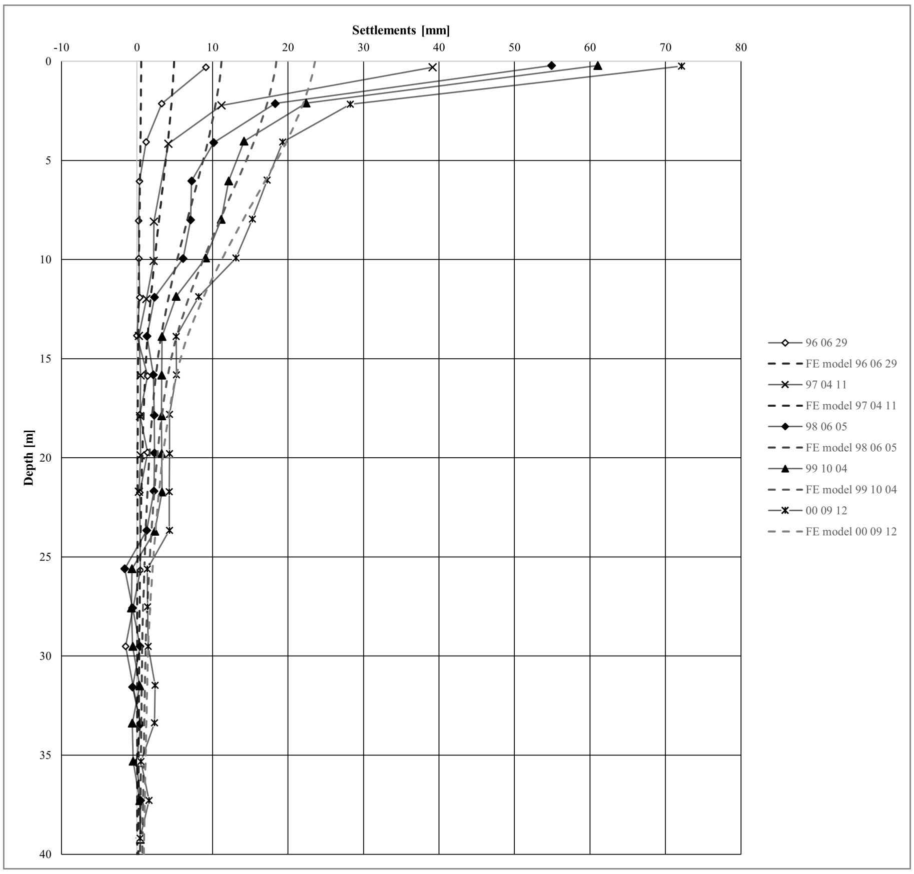

Figure 4.1: Developed settlements from bellow hose compared to FE model at different time stamps and 2.5 m applied fill.

4.1.2 Development of OCR

The development of the Creep-SCLAY1S ratio pp'_0/peq' was studied and at the time of construction compared to the measured OCR based on the CRS tests presented in 3.3.2. Figure 4.2 shows this correlation with an thickness of 3.5 m of the reclaimed fill material. It is known that OCR and p_{p0}/p_{eq0} is not directly comparable since the Creep-SCLAY1S ratio also considered anisotropy, but this difference is thought to be negligible for this study. Moreover, the development of OCR depending on fill thickness showed that the maximum derivation was 0.05 in OCR between the thinnest and thickest while maintaining the same trend development within the clay layers, see Figure B.1 in Appendix B.

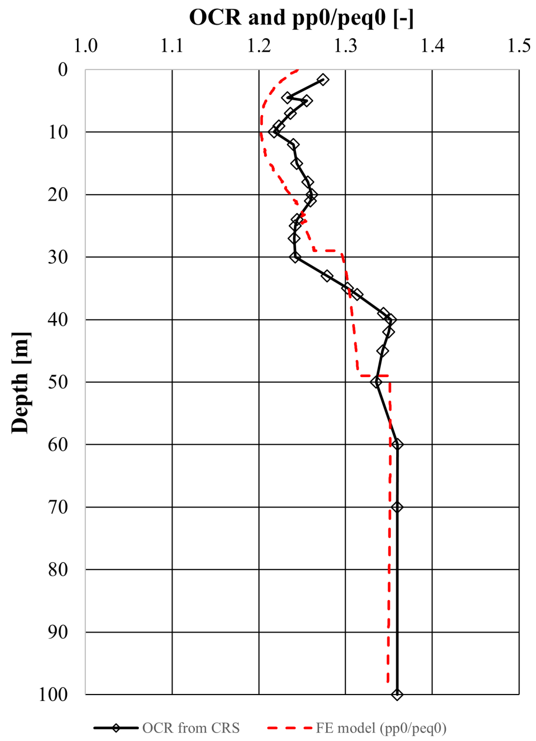

Figure 4.2: Comparison between the evaluated OCR trend compared to FE model p_{p0}/p_{eq0} cross section trend with 3.5 m fill.

4.1.3 Creep control against InSAR

A creep control was also conducted to enable verification of the rate of background creep settlements at the position of the high tower of Läppstiftet. With InSAR data [30], an estimation of the rate of background creep settlements can be made. Since the InSAR data is satellite measurements on surface settlements, the exact position of each pile can not be taken as a reference. InSAR measurements too close to current foundation should also be observed with caution since a side effect of down drag is a decreased settlement in the surrounding soil.

Background creep settlement in the surrounding area, which has experienced similar dredging and loading histories was therefore considered when evaluating the governing creep rate. Observed measurement data from 2015 to 2021 indicate creep rates ranging from approximately 4 $mm/year$ to over 10$mm/year$, with the majority of measurements clustering around 6 $mm/year$. Since all InSAR measurements are based on areas where surrounding structures are expected to have a small, yet non-negligible impact on the observed creep rate. A slightly higher creep settlement rate was considered appropriate for the numerical model, in which no such structural interference is present. A background creep settlement rate of 7 $mm/year$ in 2020 was therefore desired in the Plaxis model. This was achieved when a dredged clay layer of 4 m were loaded during the land reclamation process. Although, it could be concluded that the thickness of the dredged material had high impact of the rate of settlements which is presented in Table B.1 in Appendix B.

4.2 Pile modelling

To model pile behaviour in Plaxis, several modelling decisions must be made regarding the structural representation and the soil-structure interaction. Since the objective of this study is to develop a simplified numerical model capable of reproducing the generalised pile response, the modelling approach is based on this. Special regrades have been taken on the interaction between soil and structure, as it governs the development of shaft resistance for floating piles. Pile number 629 (78 m long) at Läppstiftet was selected as the reference pile [7].

4.2.1 Pile element

In Plaxis, piles can be modelled using different structural features, including embedded pile rows, plate elements, fixed-end anchor or volume elements. In this study, the pile is represented as a volume element in order to maintain a simplified model. The pile geometry is defined by a polygon corresponding to the pile cross-section and is assigned with material properties. The modelled pile dimensions corresponds to those used at Läppstiftet, with a cross-section of 275 mm × 275 mm. Although, since the model is axisymmetric a correction of the radius have been performed to achieve the same shaft surface area as for a quadratic geometry. The adopted material properties of the piles are presented in Table 4.3.

Table 4.3: Model parameters for the concrete pile.

Soil model	Drainage type	γ_{unsat} $[kN/m^3]$	E_{ref} $[kN/m^2]$	$\nu(nu)$ [-]
Linear Elastic	Non-porous	25.0	37×10^3	0.2

4.2.2 Plate element

The basement slab of the building is believed to approximately have a thickness of 2 m beneath the high tower. This is replicated within the numerical model as a plate element where the self weight is removed and applied as a surcharge load at a later stage. The concrete slab have the following material properties, as presented in Table 4.4.

Table 4.4: Model parameters for the concrete slab.

Material type	w $[kN/m/m]$	EA_1 $[kN/m]$	EI $[kN\ m^2/m]$	$\nu(nu)$ [-]
Elastic	0	60×10^6	20×10^6	0.2

4.2.3 Pile-soil interface

To accurately capture the interaction between pile and soil, the structural interface element can be utilised in Plaxis. The interface include user-defined properties such as material mode, permeability condition and virtual thickness factor. In this study, the interface is adapted as a model specific material that depends on the stiffness of the soil which is calibrated against the virtual thickness of the interface [3]. This is done to achieve full negative skin friction at a 2 mm slip movement between pile and soil, as previously mentioned. The finalized model parameters for the interface is presented in Table 4.5.

Table 4.5: Model parameters for the interface.

Soil model	Drainage type	γ $[kN/m^3]$	E'_{ref} $[kN/m^2]$	$\nu(nu)$ [-]	E'_{inc} $[kN/m^2/m]$
Mohr-Coulomb	Undrained B	16	3957	0.2	544

4.2.4 Boundary conditions

The deformation boundary conditions are defined to minimise boundary effects while ensuring numerical stability. Along the vertical model boundaries, displacements are fixed in the the horizontal (x) direction while vertical displacements (y) are allowed. The bottom boundary is fixed in both the horizontal and vertical direction.

Regarding groundwater conditions, vertical seepage is permitted while horizontal flow across the vertical boundaries are restricted. This allows for pore pressure redistribution in the vertical direction while maintaining controlled hydraulic boundary conditions.

4.2.5 Pile spacing and distributed load

Design documentation obtained from Skanska's archives indicates that a surcharge load of 200 kN/m^2 has previously been applied in pile and raft calculations for the high tower. However, the basis for this value is not clearly documented and it cannot be confirmed whether it represents a characteristic load or a design load including partial safety factors. Therefore, the applied surcharge load is associated with high uncertainty.

Similarly, the pile centre-to-centre spacing is believed to be varying throughout the building while the available documentation provides inconsistent information. Older design drawings indicate a spacing of approximately 1.1 m, whereas modelling assumptions suggested by Wood, propose a spacing of 2.5 m [34]. In order to establish a representative value for the numerical model, several independent sources were evaluated. The area of the raft was is known to be approximately 1050 m^2, and when distributed over the total amount of piles (234), each pile supports an area of approximately 4.5 m^2. This corresponds to an equivalent centre-to-centre spacing of around 2.1 m and a spacing-to-diameter ratio of around 8. From another point of view, when assuming a nearly full utilisation of structural capacity of the piles (1080 kN) it corresponds to a pile spacing of approximately 2.3 m. As most options corresponds to a spacing of 2.5 m (8D), this is believed to be accurate and is further applied in the model.

It was tried to convert the surcharge load into an equivalent load applied directly at the pile heads, while load transfer from the raft to the surrounding soil was neglected. This idealised approach reduced the total settlements, as a larger portion of the load is transferred directly into the piles and deeper soil layers which limits the stress increases in the upper clay layers. Although, since the objective of this study is to capture time-dependent settlement behaviour governed by consolidation and creep, the raft-soil interaction was retained by utilising an surcharge load in the final modelling approach. Neglecting the interaction was considered potentially non-representative.

Given the uncertainty in the applied surcharge load, several load magnitudes were evaluated. A surcharge of 150 kN/m^2 was found to provide more reasonable agreement with the observed settlement behaviour in serviceability limit state (SLS) analyses. To be precise, it reproduces the settlement rate reported by Claesson during the first years after construction [7], as well as the estimated long-term settlement trends reported in internal documentation. The overall analysis of load magnitude is presented in Figure 4.3 and 4.4.

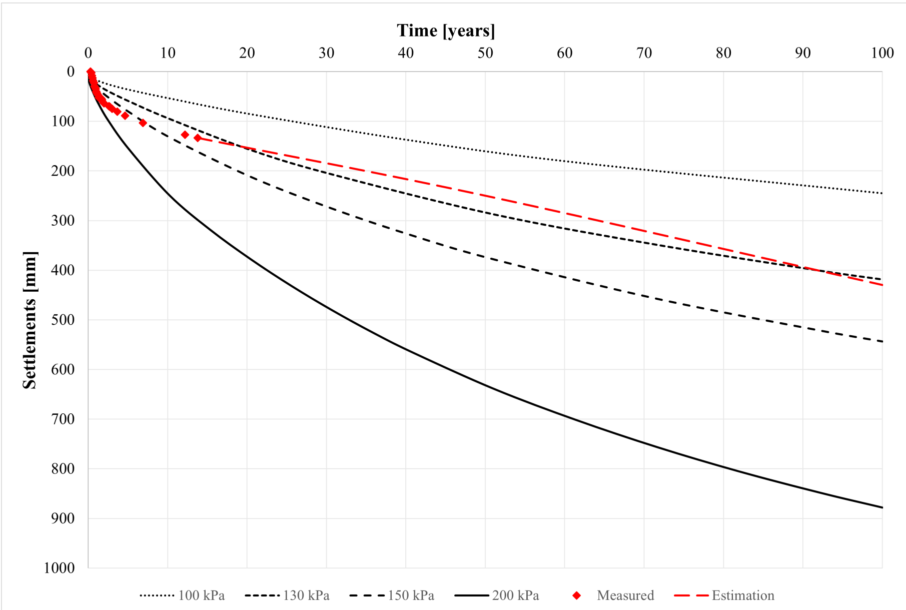

Figure 4.3: Development of settlements over a hundred year consolidation period at different load magnitudes.

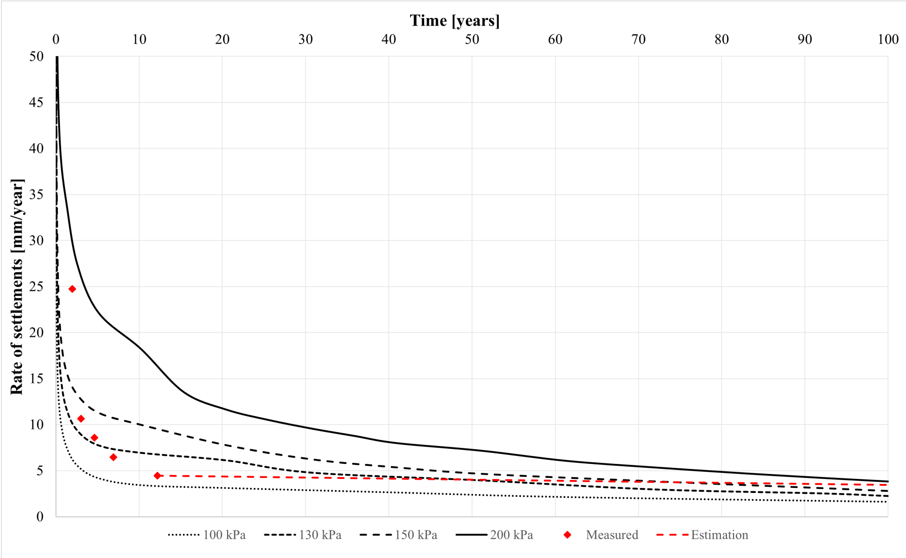

Figure 4.4: Rate of settlements over a hundred year consolidation period at different load magnitudes.

Overall, both the surcharge load and the pile spacing should be regarded as approximate values. The adopted parameters represents a best-estimate interpretation based on available archival material, geometric assessment, and calibration against measured settlement behaviour.

4.3 Plaxis 2D model

A visual representation of the developed 2D FE model of a floating pile is shown in Figure 4.5. Conceptually, the model represents a single centre pile within an idealised pile group, assumed a fixed centre-to-centre spacing and an equal load distribution between the piles. A detailed description of each calculation step is provided in Appendix C. Furthermore, a mesh sensitivity analysis to confirm that the result converges which is provided in Appendix D.

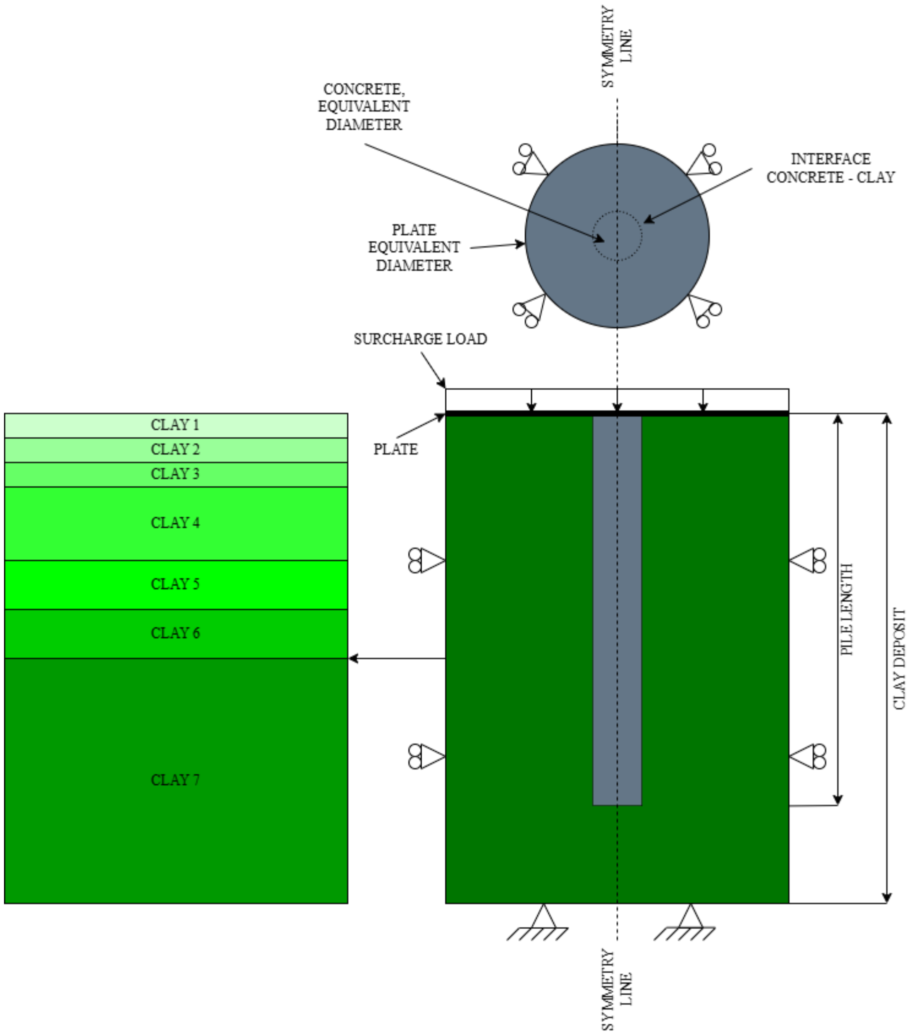

Figure 4.5: Visual representation of the 2D numerical floating pile model.

4.4 Plaxis 3D model

A 3D model was developed to evaluate how representative the simplified 2D unit cell approach remains when scaling the system to a larger pile group configuration including surrounding soil. The 3D analyses were therefore not only used for direct comparison with the 2D model, but also to investigate which structural and geotechnical mechanisms are not captured by the simplified axisymmetric approach.

The 3D model shares the same boundary conditions, soil profile, Creep SCLAY1S model parameters and construction stages as the 2D model. It was constructed as a 5×5 pile group with symmetry lines on both the x- and y-axes, representing one quarter of a full group of 100 piles. The model also include surrounding soil, as only the soil mass for the foundation is excavated. Although, no surrounding structures or interactions with the Göta River were considered in the 3D model.

The piles was modelled with a square cross section in contrast to the 2D model, but with the same centre-to-centre distance. The 3D model extends 75 m wide in both x and y direction and has a uniformly depth as the original 2D unit cell model. An overview of the PLAXIS 3D model is shown in Figure 4.6.

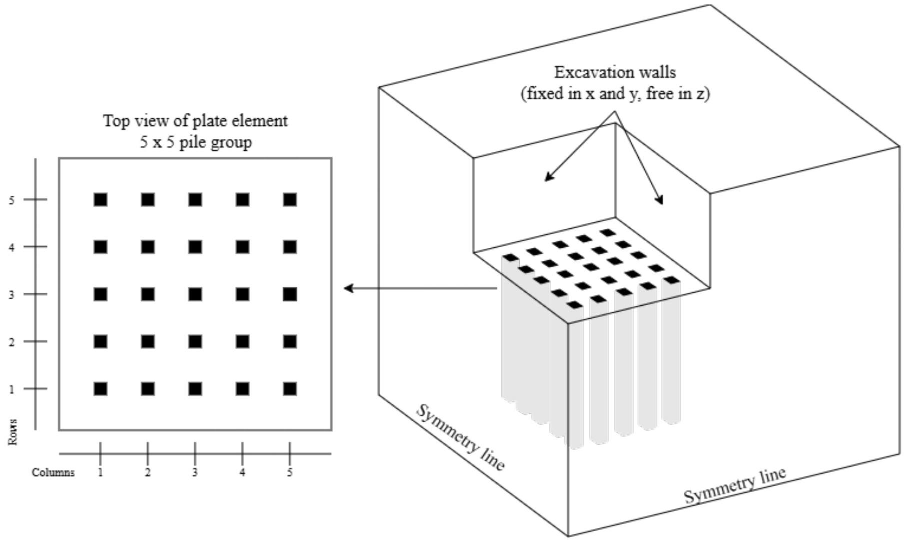

Figure 4.6: Visual representation of the 3D numerical floating pile model.

An alternative Plaxis 3D model with shorter edge piles were also constructed to study the impact of different pile lengths throughout the structure. The edge piles, being the last 2 columns and rows, were shorten to both 50 and 65 m instead of 78 m, as in the original model.

5

Results

The following chapter presents the results from the FE analyses of a floating pile unit cell. The numerical simulations were mainly performed using two-dimensional axisymmetric formulation but also compared to a three-dimensional. The presented results focus on the settlement development, axial force distribution, pile-soil interaction and the influence of pile spacing and length. Comparisons between numerical results and field measurements are also included.

5.1 Geotechnical ultimate limit state

The geotechnical ultimate limit state (ULS) was investigated by applying a static progressively increased axial load at the pile head, followed by unloading. The objective was to evaluate at which load level the pile-soil interaction exhibits a change in stiffness, i.e., indicating geotechnical failure. The results are presented in Figure 5.1.

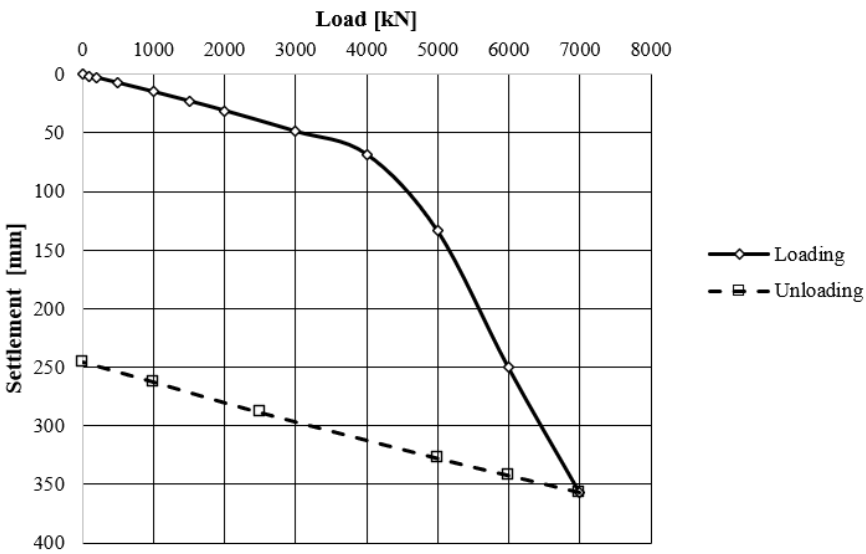

Figure 5.1: Results from the static pile load test for pile number 629 in the 2D floating pile model.

The load-settlement curve shows an initially stiff response at lower load levels, followed by gradually increasing the settlements as the applied load increases. A significant non-linear response is observed at approximately a load of 4000 kN, where a small increase in load results in larger settlements. During unloading, only limited recovery of settlements are observed. To ensure that the adopted pile stiffness does not govern the ULS a sensitivity analysis was carried out, which is presented in Appendix E.

5.2 Settlement development over time

Figure 5.2 presents the development of settlements over time for pile number 629 as obtained from the FE floating pile model. The figure further includes measured settlement data from monitoring point 146, as well as an extrapolated estimate of future settlements by internal representation. In addition, the modelled settlement behaviour of the surrounding soil before any imaginary construction had taken place (wished in place pile).

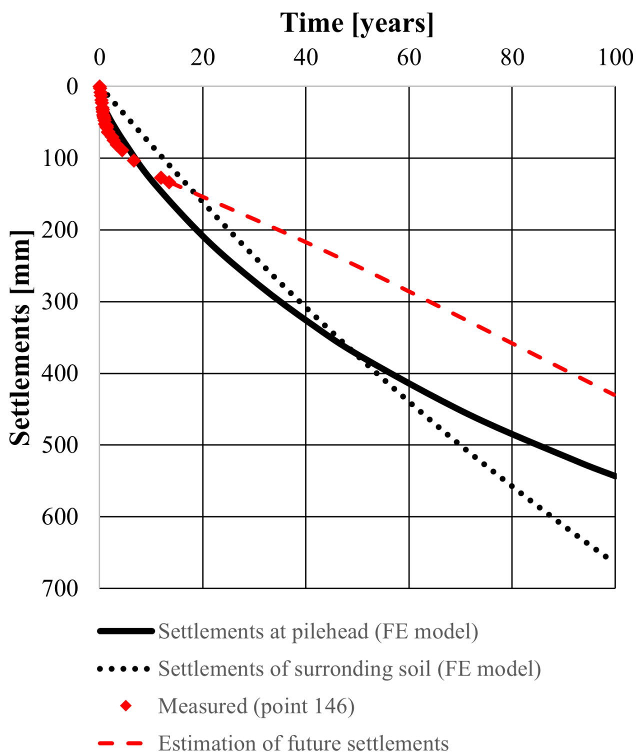

Figure 5.2: Comparison between settlements from FE analysis, measured data from point 146 and estimated future settlements.

The numerical results indicate that the settlements at the pile head increase more rapidly in the initial period compared to the following development where it flattens with time. The settlements of the surrounding soil predicted by the FE model are larger than those at the pile head after the investigation period. Although, following a similar rate response as the FE pile head which is initially more rapid then the following.

The measured settlements data initially follows a similar trend as the FE pile head results but deviates at later stages. While the numerical model predicts continued settlement development over the whole analysed period, the measured data indicate a reduced settlement rate during the monitoring period. The extrapolated future settlement curve suggest that additional settlements may continue to accumulate although this remains highly uncertain.

5.3 Axial force development

Figure 5.3 presents the measured axial forces reported by Claesson [7] together with the axial forces obtained from the 2D FE analysis at the same time period.

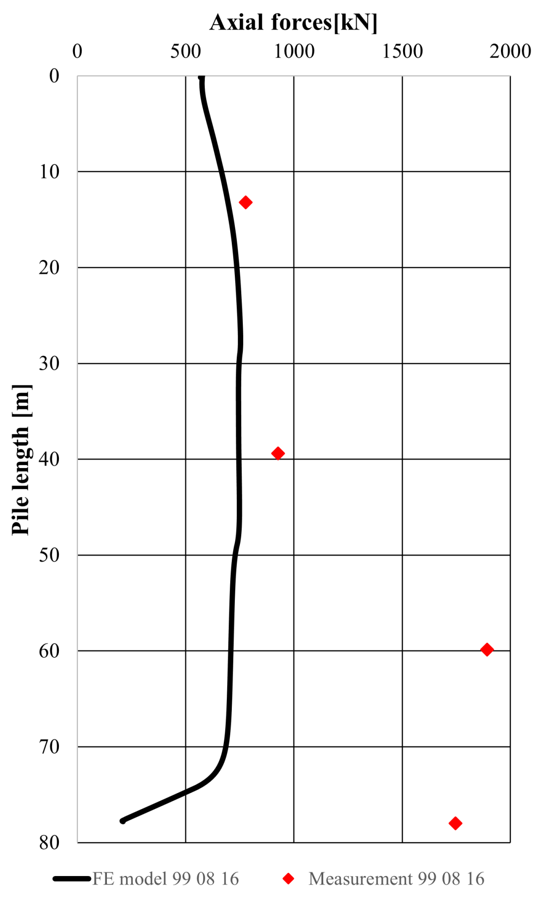

Figure 5.3: Recorded axial forces in pile number 629 by Claesson [7] compared to those measured in the FE analysis at the same time period.

The measured axial forces highly differ from those predicted by the numerical model. The measured data show higher maximum axial forces and a different force distribution along the pile length compared to the numerical model. The numerical model shows an almost constant axial force with depth.

5.4 Soil-pile interaction

As shown in Figure 5.4, the pile and the surrounding soil settle relatively uniformly along most of the pile length within the 2D model. In the first 20 *m* of the pile, the soil settles faster relative to the pile which results in increasing relative displacement between the pile and and surrounding soil in the upper clay layers.

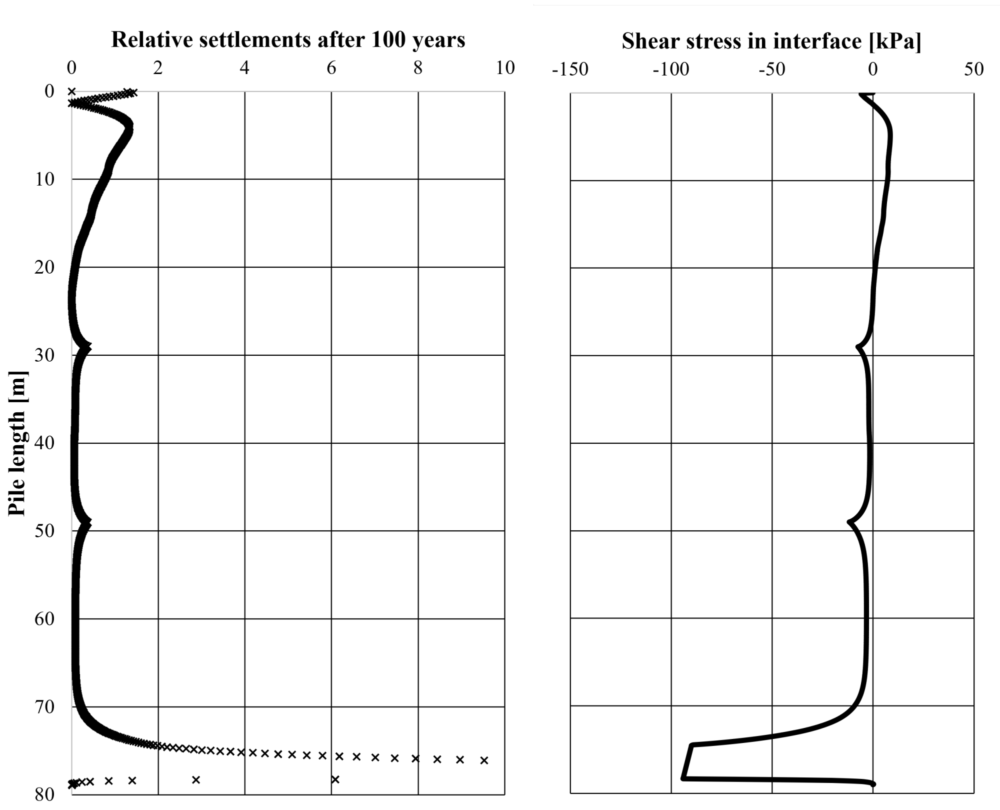

Figure 5.4: Absolute relative settlements and shear stresses in interface after 100 years.

The computed settlement profile at the models boundary is presented in Figure 5.5. The results show that the soil settles relatively uniformly down to 78 *m* depth, approximately between 520-550 *mm*. Below the pile tip level, the settlement curve becomes nearly horizontal which indicate that the majority of vertical deformation is concentrated in the upper portion of the clay layers.

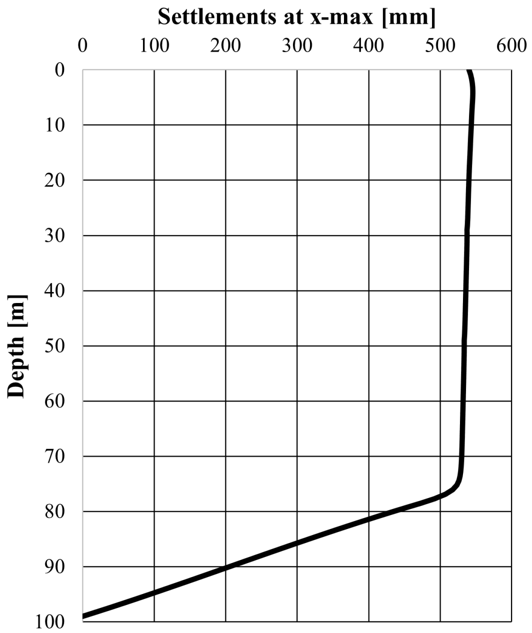

Figure 5.5: Settlements at end boundary (x-max) after 100 years.

5.5 Influence of centre-to-centre spacing

Different centre-to-centre (CTC) spacings have been studied and key units have been compared. The time-settlements curves, presented in Figure 5.6, show the development of vertical displacement over a 100 year consolidation period. The original FE pile number 629 results are included as a comparison. All CTC spacings results follow a similar trend although with different magnitudes. For CTC spacings below 8D, the settlements at the pile head increased as the distance between piles decreased. However, for CTC spacings of 8D and above, increasing the CTC distance had no significant effect on the total settlements at the pile head. The results therefore suggest that the settlement behaviour becomes less sensitive to pile spacing for spacings greater than 8D.

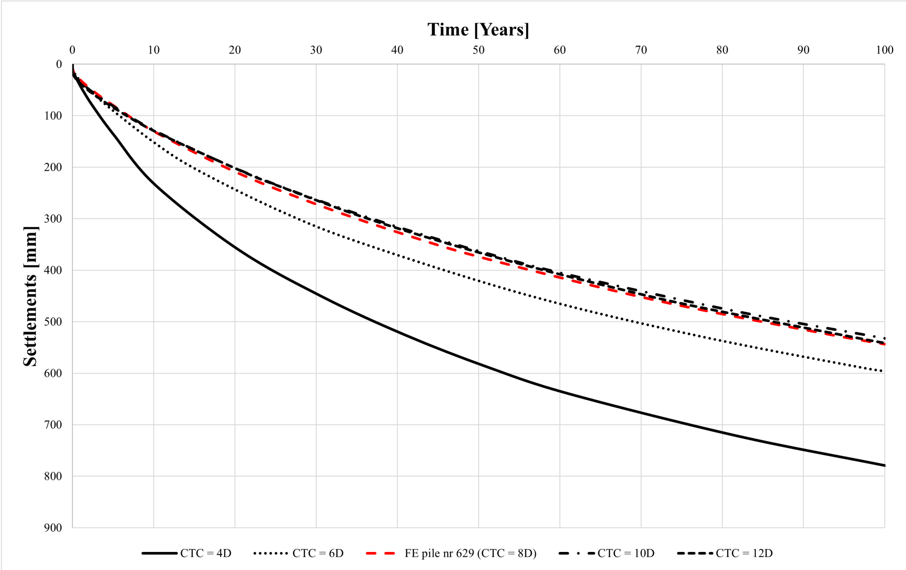

Figure 5.6: Settlements over time at different CTC spacings, including the FE pile number 629 results as comparison.

The axial force profiles, presented in Figure 5.7, shows how the axial load is developed along the pile length for different CTC spacings. The results demonstrate that the axial force distribution varies depending on pile spacing. As well as, that it increases with higher spacing. The largest differences can be observed in the upper sections of the pile shaft where negative skin friction develops and drag loads.

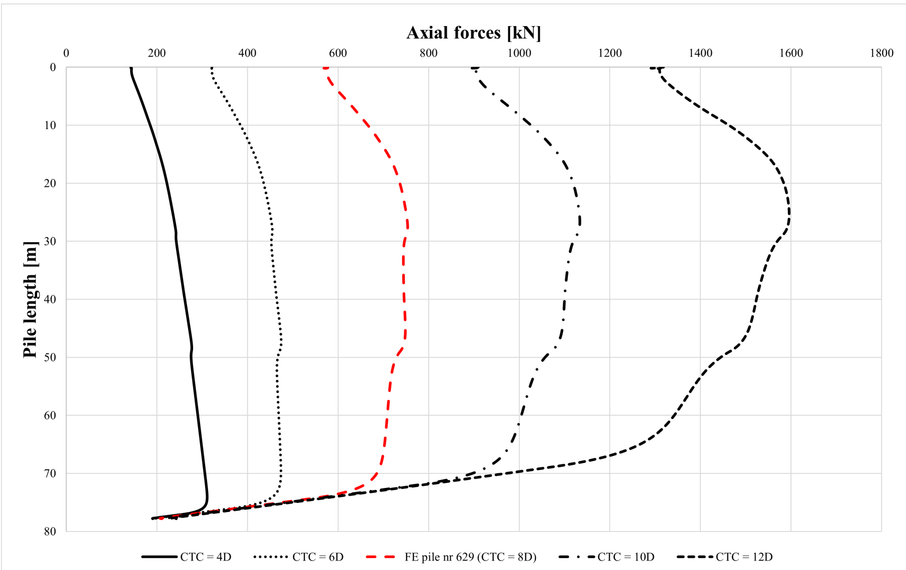

Figure 5.7: Developed axial forces after a 100 year consolidation period for different CTC spacings.

The settlement profiles at the model boundary, presented in Figure 5.8, show a consistent trend across all spacing ratios. Results show that with an increase in CTC spacing, soil settles less up to a spacing of 10D and then increases again when reaching 12D. This can be seen in both the pile head as well as in deeper deposits. The results especially show a major increase in settlements for CTC spacings of 6D and 4D.

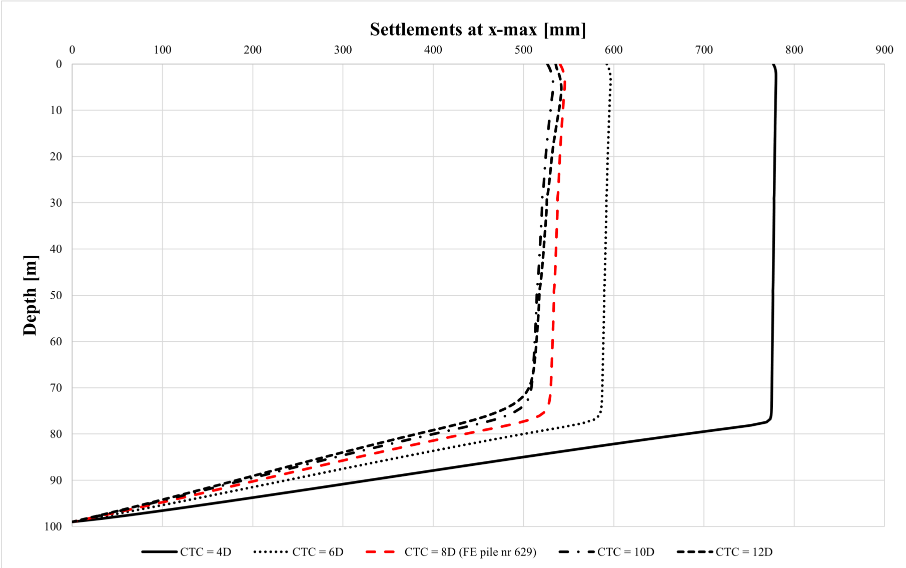

Figure 5.8: Settlements at end boundary (x-max) for different CTC spacings after a 100 year consolidation period.

5.6 Influence of pile length

The pile length-to-diameter ratio (L/D), has a significant influence on the magnitude and distribution of total settlements at the pile head. Figure 5.9 shows the development of settlements over time for different pile lengths. Increasing pile length resulted in lower settlements throughout the analysed period. The shortest pile configuration ($L/D = 100$) develop the largest settlements while the longest configuration ($L/D = 300$) produces the smallest settlements. All pile configurations showed an initially rapid settlement development, then followed by a lower settlement rate.

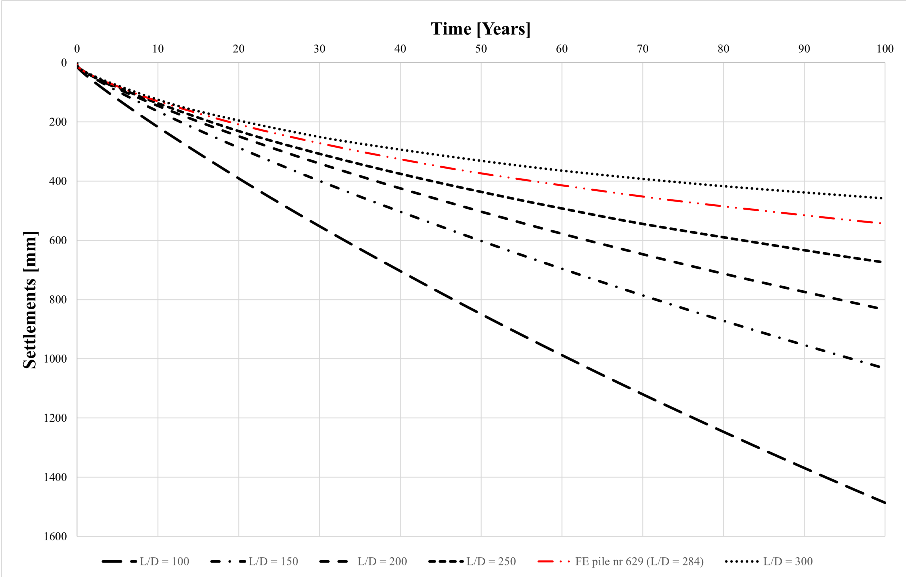

Figure 5.9: Development of settlements over time for different L/D ratios, including results from FE pile number 629 ($L/D = 284$).

The axial force distribution, presented in Figure 5.10, provide additional insight into the load-transfer mechanism. While the axial forces at the pile head remained relatively similar across all cases, the longer piles maintain higher axial forces over greater depths. As the shorter piles show a more rapid reduction in axial forces with depth.

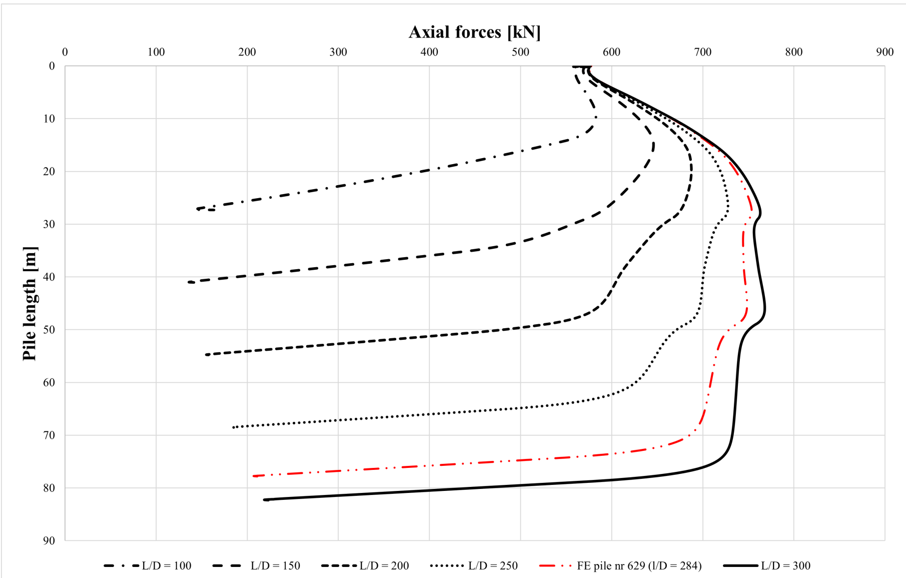

Figure 5.10: Developed axial forces after 100 year consolidation period for different L/D ratios.

The settlement profile at the model boundary presented in Figure 5.11 further show that the deformation behaviour varies depending on pile length. Shorter pile lengths exhibit larger settlements concentrated in the upper clay layers. In contrast, longer piles lengths show reduced settlements near the ground surface as well as a more linear reduction beneath the pile tip. The $L/D = 300$ case produced the lowest settlement over the majority of the analysed depth.

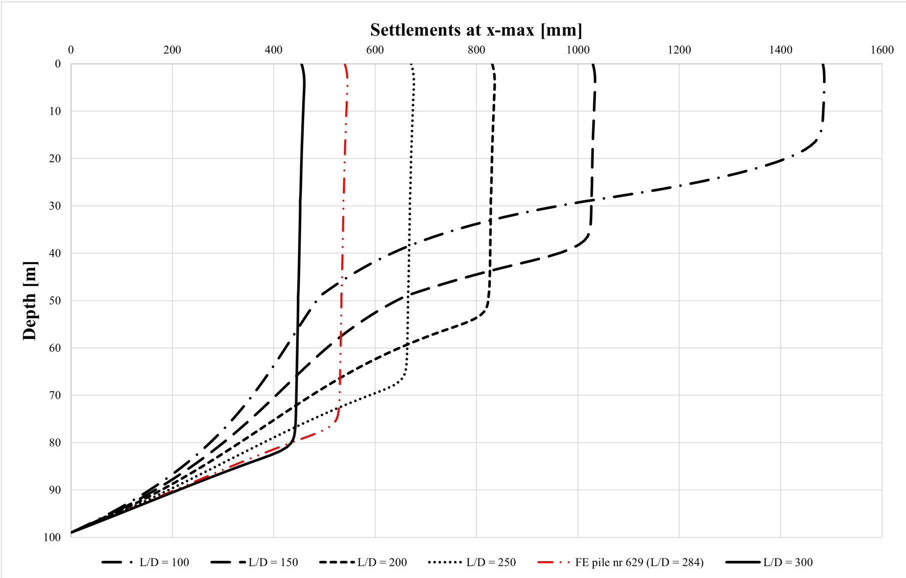

Figure 5.11: Settlements at end boundary (x-max) for different L/D ratios after a 100 year consolidation period.

5.7 2D versus 3D FE floating pile model

The 3D FE models generally produced lower settlements compared to the 2D unit cell model. Differences between the piles located in the centre and edge piles were observed in the axial force distribution. While the 2D model generated uniform axial forces, the 3D models showed more variations depending on the placement within the pile group. Therefore, the piles selected for analysis in all models were chosen to provide an varying representation.

5.7.1 Uniform pile lengths

As shown in Figure 5.12, which represents a 3D model with a 5 x 5 group of 78 m long piles installed in the same soil profile as the 2D reference model, much lower settlements are shown in the 3D model compared to the 2D unit cell model. With uniform pile lengths, no notably differences in settlements at each individual pile head can be observed.

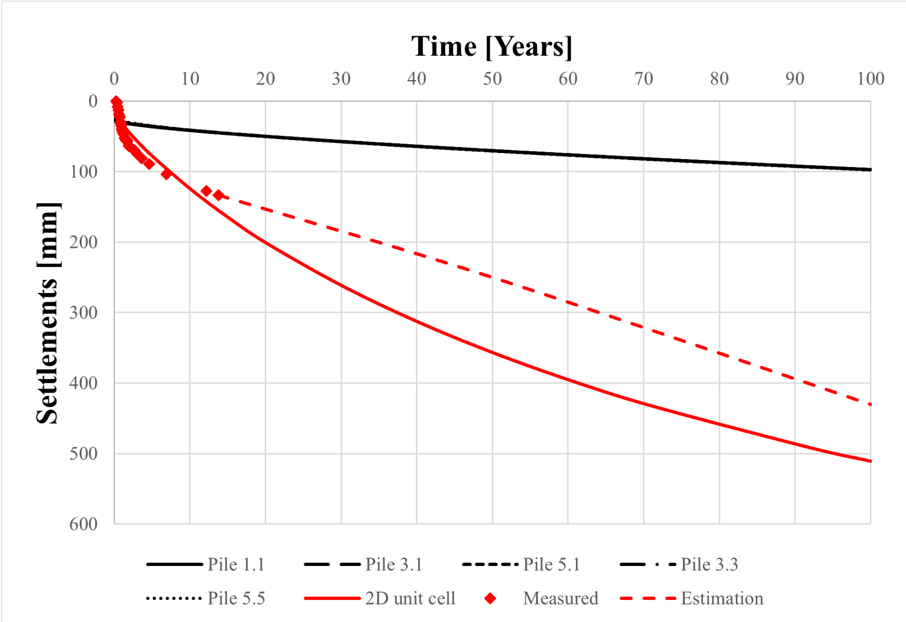

Figure 5.12: Settlements for selected piles in the 3D model with uniform pile lengths compared with the 2D FE model and measured as well as estimated settlements.

Although individual pile settlements are similar, results show major differences in axial load in piles as presented in Figure 5.13. The edge piles developed higher maximum axial forces compared to the centre piles. The measured response from pile number 629 was closer the axial force curve of the edge piles than the centre piles. The axial force profiles show that negative skin friction developed primarily in the upper clay layers while below the neutral plane, the axial force gradually decreases with depth.

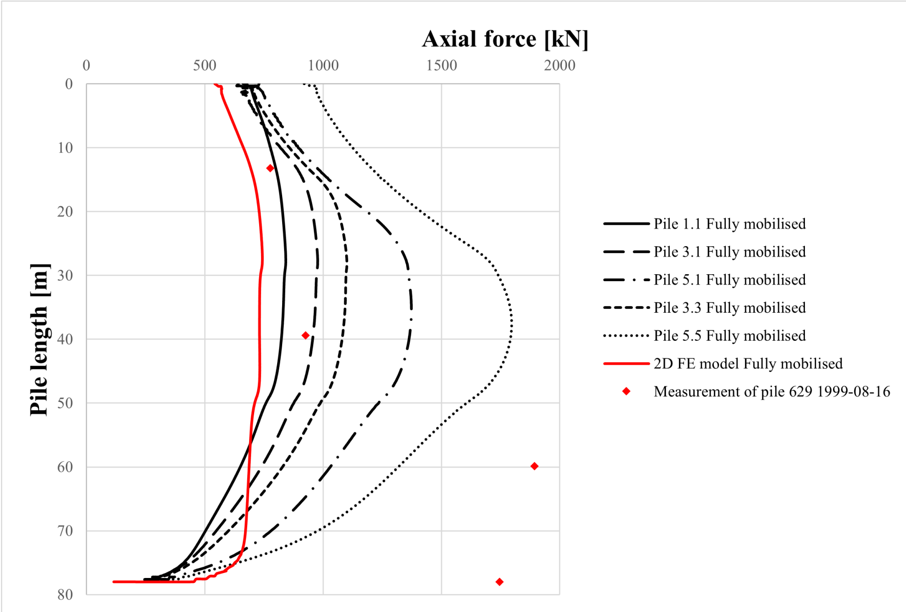

Figure 5.13: Axial force distribution for selected piles in the 3D FE model compared with the 2D FE model and measured axial forces from pile number 629 under fully mobilised conditions.

5.7.2 Varying pile lengths

Figure 5.14 shows the axial force distribution in a model with 65 m edge piles on the two outermost rows and columns while the rest are maintained as 78 m long. The edge piles developed higher axial forces compared to the interior piles, while the centrally located longer piles maintained axial forces over a greater depth.The largest variations occurred within the upper and middle sections of the pile shaft.

As the axial load increases down to the neutral plane, a redistribution occurs further down where the load is transferred to the longer piles located closer to the centre. The axial forces in Pile 1.1, the pile closest to the centre, are relatively similar to those in the 2D unit cell model in terms of their development, but remain considerably higher throughout most of the pile length. It can also be noted that the location of the neutral plane of Pile 1.1 is slightly lowered compared to the uniform pile length model.

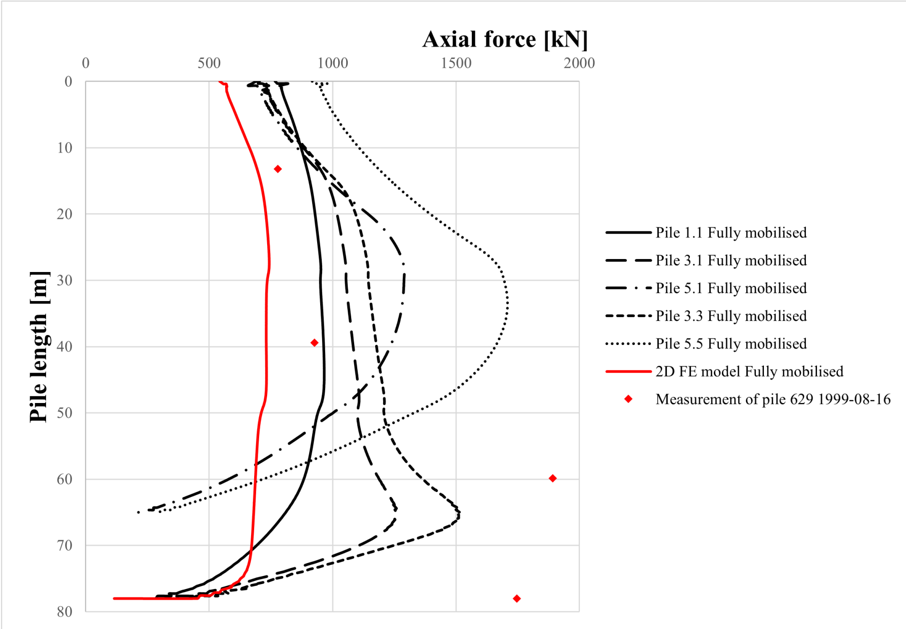

Figure 5.14: Axial forces for the 3D model with 65 m edge piles.

No noticeable difference in settlements can be observed between the model with uniform pile lengths and the model with 65 m edge piles, see Figure 5.15 for reference. The pile heads developed relatively similar settlements throughout the pile group despite the variation in pile lengths. The overall magnitudes of the settlements also remained close to those shown in the uniform pile length configuration.

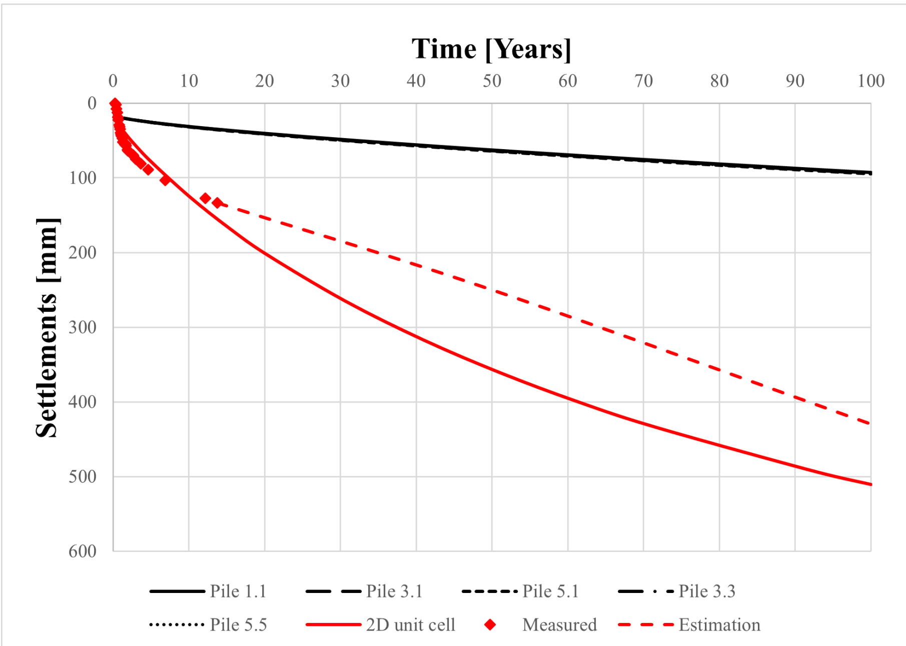

Figure 5.15: Settlements for the 3D model with 65 m edge piles.

5.7.3 Additional studies

Additional 3D analyses were initiated to further investigate the influence of some increased surcharge loads, one doubled CTC distance and changed boundary conditions which is presented in Appendix F.

Overall, the preliminary results showed only minor increases in settlements and slight increase in axial loads for the models with higher surcharge load. The model with double CTC spacing between piles exhibited similar trends, though the differences were more pronounced. Although, the additional studies were limited in scope, the results indicate that pile spacing and surcharge loading continue to influence the load transfer behaviour and settlement response within the pile group.

Additionally, an impact study on boundary effects from surrounding structures was conducted using the varying pile-length model. In this study, the symmetry lines in both the x- and y-directions were moved closer to the building to evaluate the influence of surrounding structures and additional soil on total settlements. As shown in Figure 5.16, the amount of surrounding soil in the x- and y-directions in the 3D model had a significant impact on the results. When soil was cut 4 m from the boundary of the structure, representing half a street the total settlements more then doubled then previous 3D results featuring 64 m of surrounding soil in x and y direction. When the soil was cut just at the boundary of the plate element the 3D model is shown to exceed the 2D unit cell prediction.

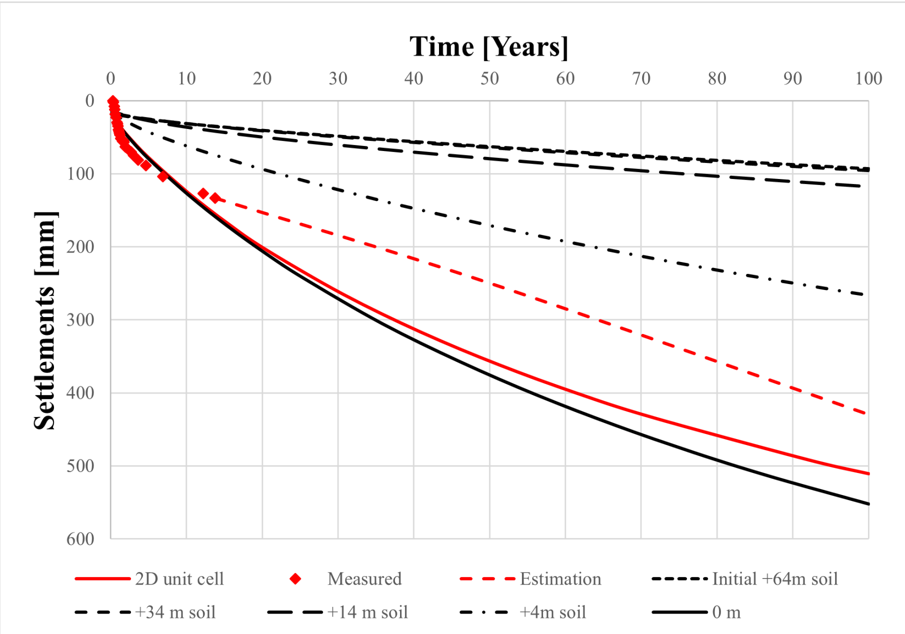

Figure 5.16: Illustration on how the total settlements varied with less surrounding soil in x and y direction before the 3D model assumed symmetry

6
Discussion

The following chapter discusses the results from the numerical models and evaluates them against the measured and estimated data, modelling assumptions and geotechnical pile mechanisms. Particular focus is placed on settlement behaviour, axial force distribution, pile group interaction and the difference between 2D and 3D results. Furthermore, limitations with the numerical model approaches and relevant suggested further studies is presented.

6.1 Geotechnical conditions and uncertainties

The creep rate of the clay deposit at the location of the bellow hose at the time of installation has been assumed thus there were limited available information. The exact thickness of the clay as well as the exact depth of the clay deposit remain uncertain due to limited data. The available soil investigations only extend to 40 m depth. Consequently, the deeper stratigraphy and its contribution to long-term settlements cannot be fully verified with the bellow hose calibration alone.

Verification of creep rates in greenfield conditions using todays InSAR measurements has some uncertainties due to disturbances associated with existing structures such as the Götatunneln. Therefore, locations nearby have been taken into account and compared. As the measured rates were generally within the same range, these uncertainties are not expected to have a significant influence on the results.

With both the bellow hose and InSAR calibrations being matched. The uncertainties with the lack of strain measurements below 40 meters seems negligible.

6.2 Behaviour of the 2D FE Floating pile model

The results from the geotechnical ultimate limit state analysis indicate that the pile-soil system approaches failure at 4000 kN. The unloading curve further shows that the recovery of settlements are limited and therefore indicate that a larger portion of the deformation is irreversible, i.e., plastic deformation. This is considered reasonable within the assumptions of the model and the defined properties of the pile and surrounding soil.

In design of pile foundation, both geotechnical capacity and structural capacity have to be considered. The geotechnical capacity is shown to be 4000 kN in the static pile load test. This is though to be reasonable with the total surface area and site specific geotechnical conditions. It should be noted however that the numerical model does not account for installation effects, potential buckling of the pile, or structural limitations associated with the joints between each pile element. Consequently, the numerical model does not account for the structural capacity of the pile which may be the design variable in this project, potentially due to the great length and slenderness of the pile. This is thought to be the reason behind the much lower design capacity of 1080 kN adopted by Skanska in this project. It can therefore be noted that the static pile load test does capture geotechnical capacity reasonably well but further studies have to be done to estimate the structural capacity.

The settlement development observed in the analyses is consistent with typical response of slightly overconsolidated soft clay deposits. The settlements increase rapidly during the initial stages and gradually reduce with time as excess pore pressures dissipate. Furthermore, the surrounding soil settles more than the pile itself within the upper part of the clay layers, resulting in negative skin friction and drag loads acting along the pile shaft. This behaviour is consistent with Claesson's findings[7].

The settlement profile at the end boundary further indicates that the majority of long-term deformation occurs within the upper clay layers. Below the pile tip level, the settlements become significantly smaller which suggest that the deeper soil layers contribute less to the overall deformation behaviour. This is also consistent with the lower OCR values observed within the upper clay deposit, where the soil is more prone to consolidation and creep.

6.3 Axial force behaviour and load redistribution

The measured axial forces reported by Claesson differ significantly from those predicted by the 2D FE analysis. The measured results show higher axial forces and a different load distribution along the pile shaft compared to the numerical model.

Claesson suggest that the shorter edge piles were unable to sustain the same magnitude of drag loads as the longer piles. As a result, the load-bearing capacity below the neutral plane seem to be insufficient for the shorter piles, causing a redistribution of loads from the shorter piles to the longer piles, which would lead to elevated axial forces in the longer piles.

This behaviour is not captured in the 2D unit cell FE analysis, which instead showcases lower axial forces and a larger zone of equal strain along the pile shaft. These results are more consistent with a behaviour expected for a central pile within a pile group, where the edge piles contribute more effectively.

Furthermore, the axial forces predicted by the numerical model remained almost constant over the full consolidation period and appear to be fully mobilised at the start. The measured data capture a more complex time-dependent redistribution of forces. Consequently, the seen difference between the measured and simulated axial forces may partly be explained by the difference in boundary conditions and structural representation. Moreover, the constitutive assumptions and interface formulation adopted may be unable to fully capture the progressive time-dependent redistribution of shaft resistance which is often seen in reality.

Additionally, installation effects were neglected in this study. Pile driving generates excess pore pressures and remoulding which can influence the development of shaft resistance and negative skin friction. This may also partly explain the difference between the measured and simulated axial forces.

These observation highlights the importance of considering three-dimensional group effects when analysing floating pile foundations, particularly for pile groups containing varying pile lengths.

6.4 Influence of pile spacing

Looking at the impact study of the pile spacing, some interesting results can be found. The results clearly show an major increase in total settlements for pile spacings of 4D and 6D. This occurs despite the distributed load being identical in all cases, and despite the piles with 4D and 6D spacing experiencing significantly lower axial forces than those in simulations with larger pile spacing.

This behaviour might suggest that the total bearing capacity of the pile group is lower then the sum of each individual pile's bearing capacity, thus resulting in group effect. When the spacing becomes to small, overlapping stress zones develop within the surrounding clay which reduces the effectiveness of the individual piles and increases the overall settlements. The effective shaft resistance becomes smaller for every pile.

The results further indicate that a spacing ratio of 10D results in the least amount of settlements both at the pile head and model boundary. For 12D the settlements are slightly increased both at the pile head and boundary, indicating on that the optimised pile spacing to reduce settlements is already reached. Moreover, this indicate that the threshold for spacing where the interaction between piles becomes less significant, and the group effect impact is minimized.

Except for cases where the piles begin to behave in a group effect, it should be noted that the major difference does not lie in the settlements when changing the pile spacing. Instead, the primary difference is observed in the axial forces within the pile elements. The difference is a result of a larger portion of the raft element being carried by each pile as pile spacing increases. This leads to higher axial forces at the pile head as well as an increase in axial loads along the pile. This increase is most likely due to load transfer from the surrounding soil, which is also partly supporting the raft element and further causing down drag on the pile element along the shaft.

6.5 Influence of pile length

Analysing the results from study of varying pile lengths, it can be observed that pile length has a significant influence on the total settlements, both at the pile head and at the boundary between adjacent piles. However, the influence on axial load is less pronounced in the pile head. Along the pile shaft on the other hand, the development of axial forces increases with increasing pile length.

As the results of the study suggest, shorter piles do concentrate settlements in the upper soil layers. As piles get longer, settlements are concentrated in deeper soil deposits. This is reasonable since this is where each pile usually dissipates its load in the lower parts of the structural element. This results in higher settlements for piles dissipating their load in soil closer to the surface, since soil increases in stiffness with depth.

The study clearly confirms the theory that pile head settlements are heavily driven by the pile length, and that to reduce settlements, an increased pile length is required. However, for longer piles, it is crucial to account for background settlements during design, as the effect of negative skin friction increases with increasing relative displacement when pile settlements are reduced.

6.6 Comparison between 2D and 3D FE Models

The comparison between the 2D and 3D FE analyses demonstrates differences in predicted settlements and the distribution of axial forces. Overall, the 3D model predict lower settlements compared to the 2D unit cell model.

The larger settlements in the 2D model may be partly due to the assumption of an infinite structural element without surrounding soil interaction, as symmetry boundary conditions restrict horizontal movement at the model boundaries. This assumption may contribute to an overestimation of settlements thus the influence of lateral soil restraints are reduced.

In contrast, the footprint of the building is small relative to the extent of the surrounding soil domain in the 3D model. Consequently, the applied stresses can spread laterally into a comparatively large volume of surrounding soil, potentially resulting in an underestimation of settlements. The modelling approach adopted for the basement structure may also have introduced additional structural stiffening. Furthermore, horizontal displacements beneath the basement slab were observed, which may have increased the confinement of the soil beneath the structure, thereby further reducing the predicted settlements. The 3D model in general is also relatively small and boundary effects may interfere with the final results. This is not analysed within this study and could be further assessed.

The 3D analyses further demonstrate that pile location within the foundation significantly influences the axial force distribution. Centrally located piles generally behave similarly to the 2D unit cell model, whereas edge piles develop much more different axial force distributions. This is especially shown when the pile group contains of varying pile lengths where the shorter edge piles exhibit higher axial forces higher up compared to the longer piles. This behaviour is not captured in the 2D model and further highlights the importance of three-dimensional interaction effects within pile groups.

The 2D unit cell approach is shown to be effective in representing the axial forces in a pile located at the centre of a raft foundation with uniform pile lengths. However, it is shown to be ineffective in capturing the development of axial forces in piles located closer to the boundary of the foundation. It is also shown to be less representative for pile foundations with varying pile lengths. This is believed to be related to the varying load transfer to each pile, depending on its location.

In summary, the comparison between the 2D and 3D analyses shows significant differences in total settlement. However, within the 3D analyses, the difference in total settlement between models with uniform pile lengths and those with varying pile lengths is minimal. The differences between the two 3D models instead lies in the axial forces, where the uniform pile length model is much more similar to the 2D unit cell model. It is also worth noting that in the varying pile length model, the longer 78 m piles, located adjacent to the shorter 65 m edge piles appear to better match the measurements much better of Pile 629 obtained in 1999.

Despite these limitations, the 2D unit cell model still captures the general settlement development and axial force behaviour reasonably well for centre piles. The simplified approach therefore remains relevant and valuable for preliminary analyses. However, the study demonstrates that three-dimensional analyses are required when evaluating edge pile behaviour, varying pile lengths and spatial load redistribution within the foundation system.

6.7 Modelling limitations

Some modelling assumptions and simplifications will influence the reliability of the results presented in this study.

Installation effects were not included in either the 2D or 3D analyses. While in reality, installation may significantly influence the surrounding soil of the pile through remoulding and excess pore pressure generation etcetera. Neglecting these effects may therefore influence both the predicted settlements and the axial force distribution.

Moreover, the estimation of future settlements are based on linear interpolation, which implies an approximately constant settlement rate beyond the measurement period. Such assumption are often unlikely to accurately represent the long-term settlement behaviour. Especially, important time-dependant mechanisms governing creep and consolidation are not taken into account.

Additional uncertainty is associated with the measured settlements and axial forces due to the sensitivity of the instrumentation. The instruments may have been damaged during installation or influenced by variations in the structural elements. Consequently, the measured data should be interpreted with caution when compared to the numerical results.

Despite these limitations, the numerical analyses provide valuable insight into the behaviour of floating pile foundations in soft clay deposits and contribute to an improved understanding of settlement development, load redistribution and pile group interaction effects.

6.8 Additional studies and further research

To further elaborate these theories and statements discussed in the previous chapter, it is recommended to further continue this research with the following studies.

A limitation of the present study is that installation effects were not considered in the numerical models. These effects can have a significant impact on pile performance, particularly for displacement piles. The Läppstiftet project was constructed using driven displacement piles, a piling method widely used in Scandinavian soft soils. During installation, the pile displaces the surrounding soil both laterally and vertically, leading to stress redistribution and potential remoulding of the soil. This process may locally reduce the undrained shear strength. These are only a few examples of installation effects that may occur in reality but are not captured by the "wished-in-place" approach used in this study. For future development of this study, it is recommended that the impact of installation effects be further investigated.

The cut-3D model confirmed that the presence of surrounding soil does have a significant effect of reducing differential settlements. Therefore for further development and studies, to fully account for the full soil-structure interaction mechanisms, a complete 3D representation of the boundary conditions, such as surrounding structures, Göta River and bedrock topography is recommended. This was partly carried through in Isaksson's work [14] and can be further used for inspiration in future work.

7
Conclusion

This study investigated the long-term behaviour of floating pile foundations in a soft clay typical for Gothenburg. A combination of advanced numerical modelling and benchmarking against measured settlement data from Läppstiftet in Gothenburg was used. The main objective was to evaluate how the modelling approach influence settlement development, axial force distribution and soil-pile interaction within floating pile systems subjected to long-term consolidation and creep deformation.

The results showed that creep deformation significantly affects the settlement behaviour in deep soft clay deposits that are slightly overconsolidated. Both the numerical analyses and the measured settlements show that settlement evolves over long periods of time and continues even after 100 years, which represents the scope of the present study. The numerical results captured the measured settlement data the early time periods, as deviations occurred during later stages. These differences are likely due to uncertainties in field measurements, simplifications in the numerical model and the complexity of capturing the in-situ creep behaviour with high accuracy.

Furthermore, the study corroborated that pile spacing has a substantial effect on the behaviour of floating pile groups. Smaller centre-to-centre distances resulted in increased settlements and reduces pile efficiency due to pile group effects. On the other hand, the increasing spacing reduced these effects up until a threshold of approximately 10 pile diameters, where there was no impact on the settlements. The influence of pile length was also investigated through analyses with varying length-to-diameter ratios. Increasing the pile length resulted in reduced settlements and distributed axial forces over greater depths. While shorter piles exhibit a more rapid reduction in axial forces with depth. Moreover, in normally consolidated or slightly overconsolidated clay deposits where stiffness increases with depth, the study confirms that shorter piles result in larger settlements than longer piles.

The comparison between the 2D and 3D models showed that the 2D approach was capable of representing the general settlement behaviour and average axial force development of centrally located piles. However, the 3D analyses captured substantial variations between piles within the pile group that could not be represented in the simplified 2D model. Specifically, the edge piles developed higher axial forces and were strongly affected by negative skin friction and settlements in the surrounding soil. The results therefore demonstrate that the three-dimensional effects are important when evaluating load distribution and interaction between piles.

To conclude, the study demonstrates that the long-term behaviour of floating pile foundations in Scandinavian soft clay is governed by a complex interaction between deformations in the soil (including creep), soil-pile interaction and pile group effects. Simplified numerical approaches may provide reasonable estimates of average settlement behaviour, although advance three-dimensional modelling is necessary to capture the variability of axial forces and load distribution within large pile groups. The findings also highlight the importance of considering long-term creep effects when designing floating pile systems constructed in deep deposits of soft clay.

To conclude, the study demonstrates that the long-term behaviour of floating pile foundations in deep Scandinavian soft clay is governed by the interaction between creep-induced soil deformations, soil-pile interaction and pile group effects. The numerical analyses showed that a simplified 2D unit-cell model provides a reasonable representation of the average settlement behaviour and the response of centrally located piles. However, advanced three-dimensional modelling is required to capture the redistribution of axial forces and the spatial variability within large pile groups, particularly for edge piles.

The results further indicate that the current simplified treatment of creep may be conservative for deep clay deposits, suggesting that there is potential to improve practical design methodologies. Overall, the findings contribute to a better understanding of the long-term response of floating pile foundations and provide a basis for further development of simplified design approaches for soft Scandinavian clays.

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A

Creep Index Sensitivity Analysis

A study of the impact of the creep index was also conducted. As A.1 suggest, the intrinsic creep index μ_i^* has a large effect of the overall settlements for a floating pile in large clay deposits. When neglected (towards zero) the constitutive model does not capture creep effects. It can also be stated that the impact increases with magnitude up until the utilised value where it later stagnates. Furthermore, it can be concluded that it is a sensitive parameter that has large impact on the developed settlements over time.

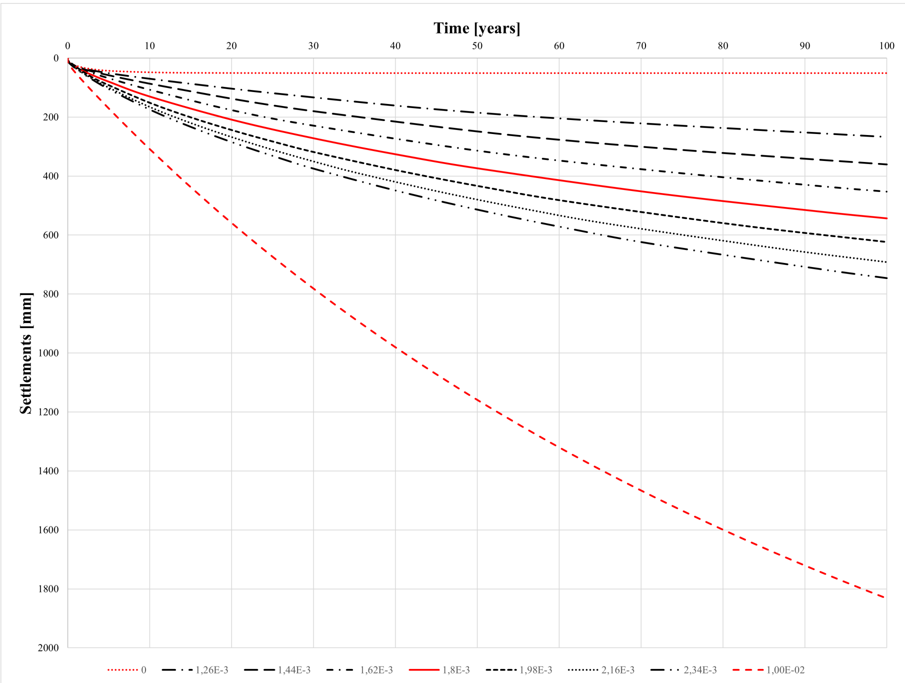

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Figure A.1: Variation of settlements at pile head for varying creep indexes.

B

Thickness of Fill Sensitivity Analysis

The analyses presented in Figure B illustrates the influence of varying assumes dredged material thickness have on the development of p_{p0} / p_{eq0}. Moreover compared to the with OCR values evaluated from Claesson's work. This is done in order to evaluate the sensitivity of the assumed historical loading conditions.

The results show that when increasing the material thickness it generally result in lower values of p_{p0} / p_{eq0}. It is more the same in the upper clay layers but varies almost with the same interval beneath 20 m depth.

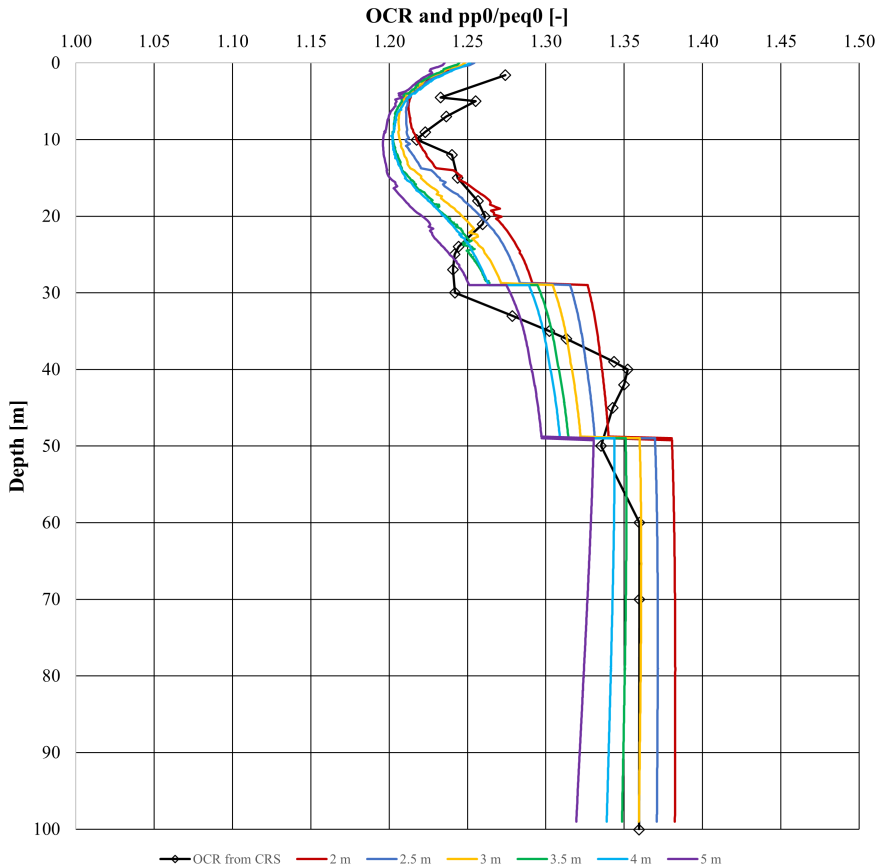

Figure B.1: Variation of p_{p0} / p_{eq0} depending on dredged material thickness and a comparison with evaluated OCR values.

The settlement rate was also evaluated for different assumed material thicknesses to investigate which thickness correspond to those observed in reality. The results show that the found rates between 3 and 5 m are mostly a representation of reality.

Table B.1: Rate of settlements depending on dredged material thickness.

Thickness [m]	2	2.5	3	3.5	4	5
Rate of settlements [mm/year]	3	4	5	6	7	9

C

FE Calculation Phases

Table C.1: Each calculation step in the Plaxis 2D model that each calculation and control models is based on and kept running on parallel.

Stage	Calculation type	Comment
Initial model	K0 Procedure	Start of model in 1860
Dredging	Plastic	Activate volume of dredged clay
Consolidation to 1990	Consolidation	130 years of consolidation
Pile calculation		Parallel simulation, see below
Creep control		Parallel simulation, see below
Bellow hose control		Parallel simulation, see below

Table C.2: Each calculation step in the following parallel pile calculation model.

Stage	Calculation type	Comment
Pile installation	Plastic	The dredged clay is deactivated and pile is wished in place as a volume element
Load on pile	Plastic	Pile is loaded with a line load and plate element
Consolidation 100 years	Consolidation	Loaded pile is consolidated in 100 years to represent end of service life

Table C.3: Each calculation step in the following parallel creep control model.

Stage	Calculation type	Comment
Parallel creep control	Consolidation	1 year of consolidation to check creep rate at start of construction
Consolidation to 2020	Consolidation	30 years of consolidation
Parallel creep control	Consolidation	1 year of consolidation to check creep rate at the time of INSAR measurements
Consolidation to 2090	Consolidation	70 years of consolidation
Parallel creep control	Consolidation	1 year of consolidation to check creep rate at the end of service life of the construction

Table C.4: Each calculation step in the following parallel bellow hose control model.

Stage	Calculation type	Comment
1996-05-24	Consolidation	Bellow hose is installed and displacements are reset
1996-06-29	Consolidation	Interval to compare with bellow hose
1996-08-14	Consolidation	Interval to compare with bellow hose
1996-10-06	Consolidation	Interval to compare with bellow hose
1996-12-08	Consolidation	Interval to compare with bellow hose
1997-04-11	Consolidation	Interval to compare with bellow hose
1997-07-23	Consolidation	Interval to compare with bellow hose
1997-11-16	Consolidation	Interval to compare with bellow hose
1998-06-05	Consolidation	Interval to compare with bellow hose
1999-10-04	Consolidation	Interval to compare with bellow hose
2000-09-12	Consolidation	Interval to compare with bellow hose

D

Mesh Sensitivity Analysis

A mesh sensitivity analysis was performed to enable accountable results, presented in Figure D.1. The general mesh setting of "very fine" was utilised and was further refined until the result converged. The final mesh is presented in Figure D.2, were a total amount of 3426 elements and 30106 nodes were generated.

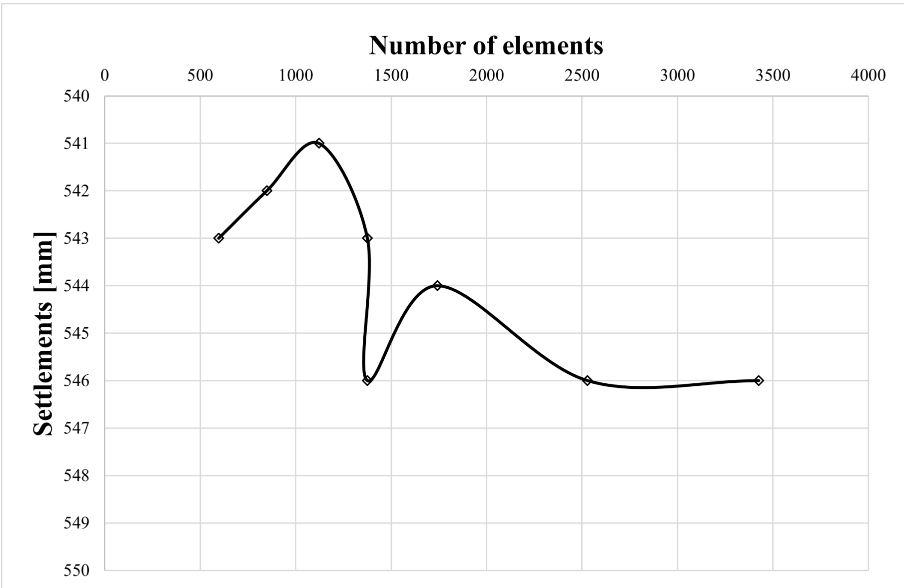

Figure D.1: Total displacement convergence depending on the number of elements.

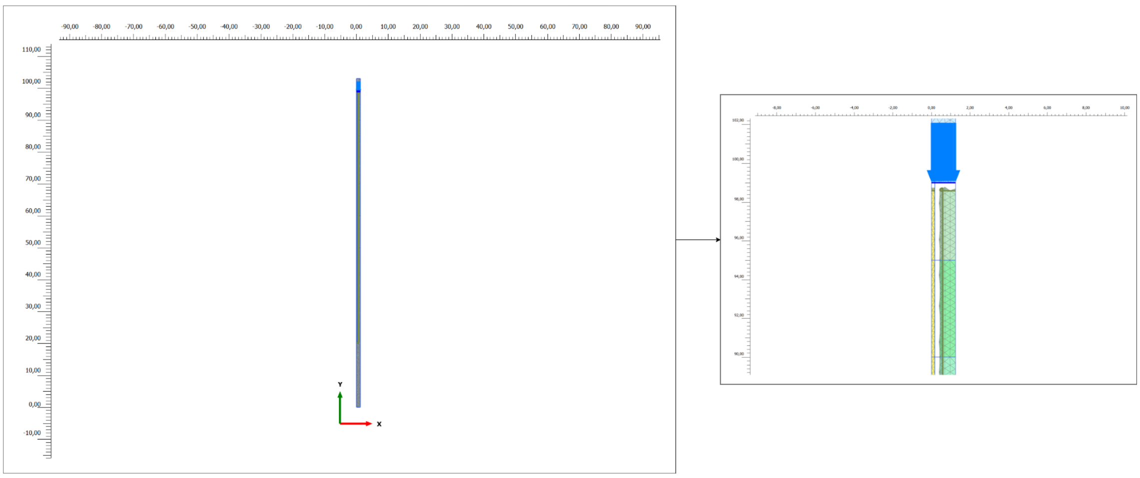

Figure D.2: Finalised mesh generation with 3426 elements and 30106 nodes.

E

Stiffness Sensitivity Analysis

Due to the high pile slenderness (L/D = 300), the stiffness of the pile can become increasingly important for the overall geotechnical ultimate limit state. The pile starts to deform progressively along its length, influencing the shaft resistance and distribution of axial forces. The analysis, as presented in Figure E.1, show that the the increases E modulus reduces the settlements and increase the initial stiffness response. However, all stiffnesses exhibited a reduction in system stiffness at almost the same load. Which indicate that the behaviour is primary governed by the deformation and strength characteristics of the surrounding clay.

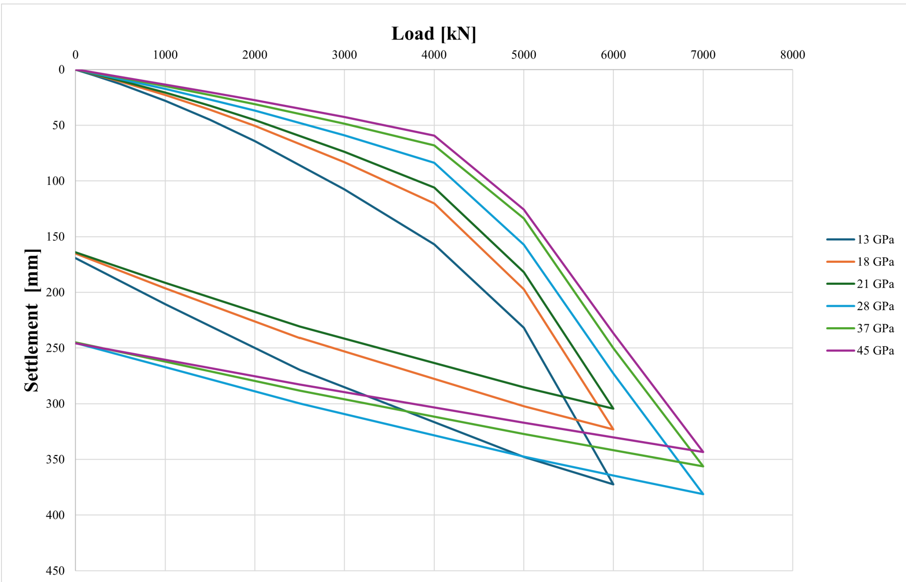

Figure E.1: Load-settlement curves obtained from the stiffness sensitivity analysis showing the influence of pile E modulus on the response of the pile-soil system.

F
Additional 3D Studies

Additional 3D parametric studies were performed to evaluate the influence of varying model conditions and geometry in a 3D floating pile group numerical model. The results from these studies, including settlements and axial force development for the different investigated cases, are presented in following appendix.

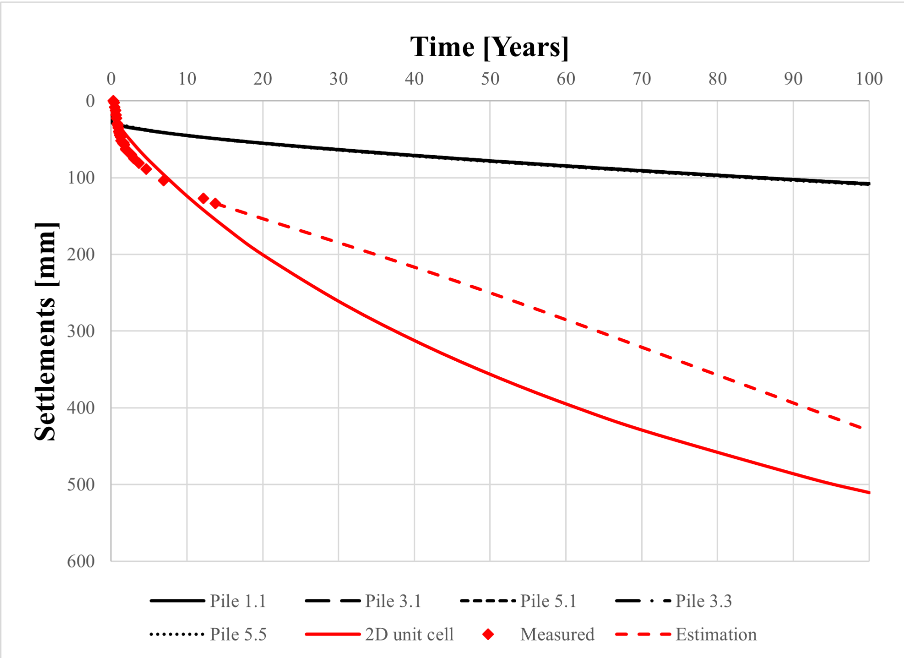

Figure F.1: Settlements for 3D study with increased distributed load to 200 kPa.

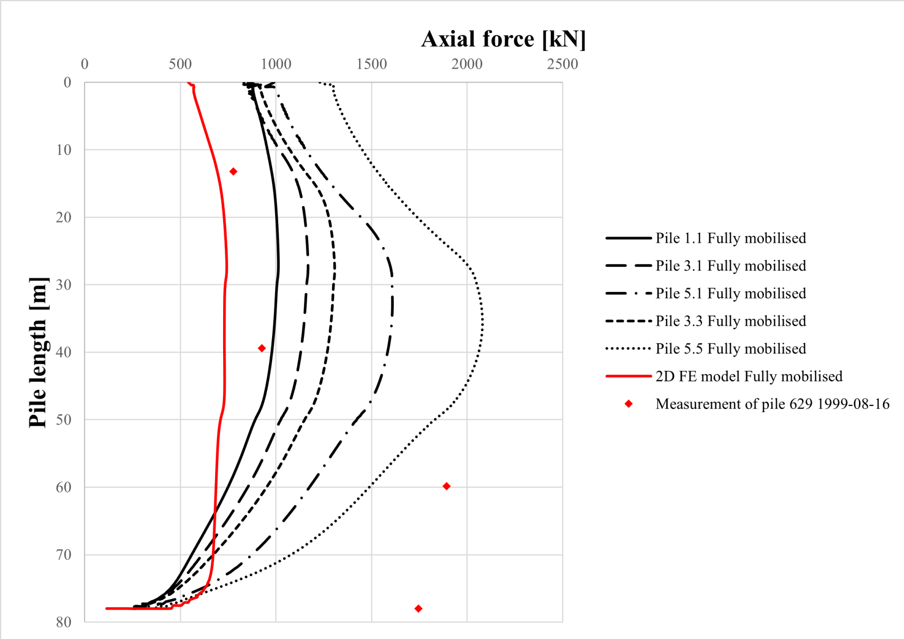

Figure F.2: Axial force for 3D study with increased distributed load to 200 kPa.

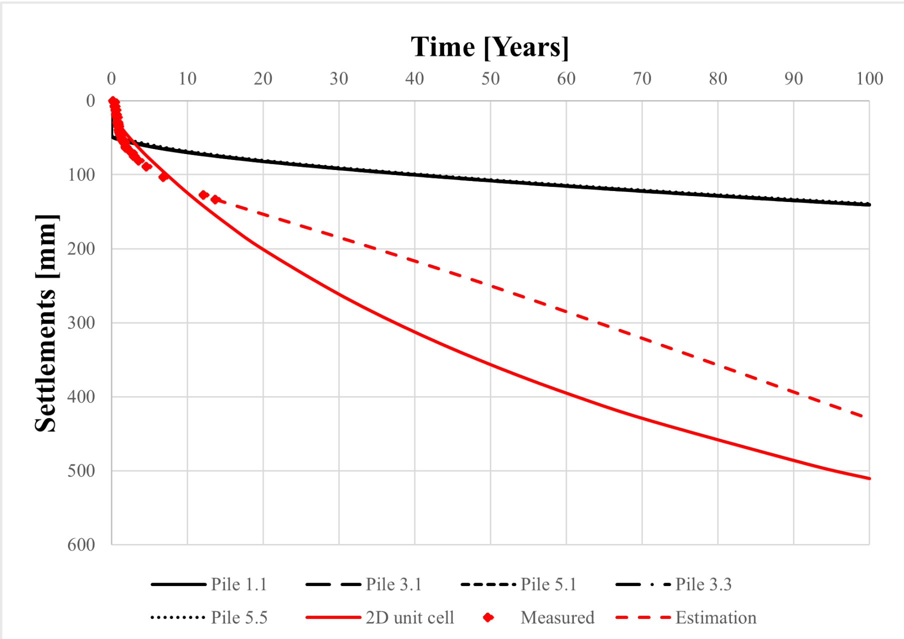

Figure F.3: Settlements for 3D study with increased CTC distance to 16D.

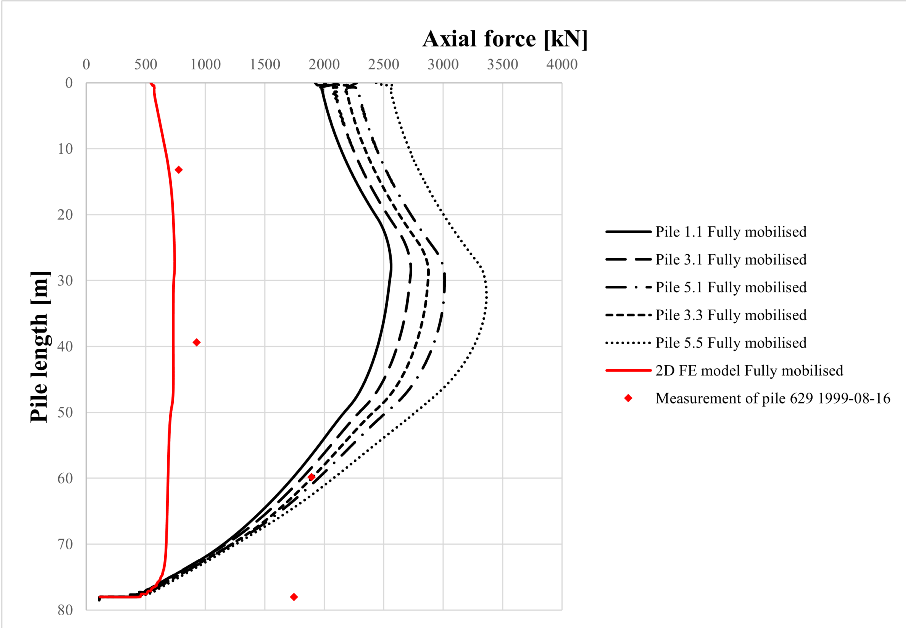

Figure F.4: Axial force for 3D study with increased CTC distance to 16D.

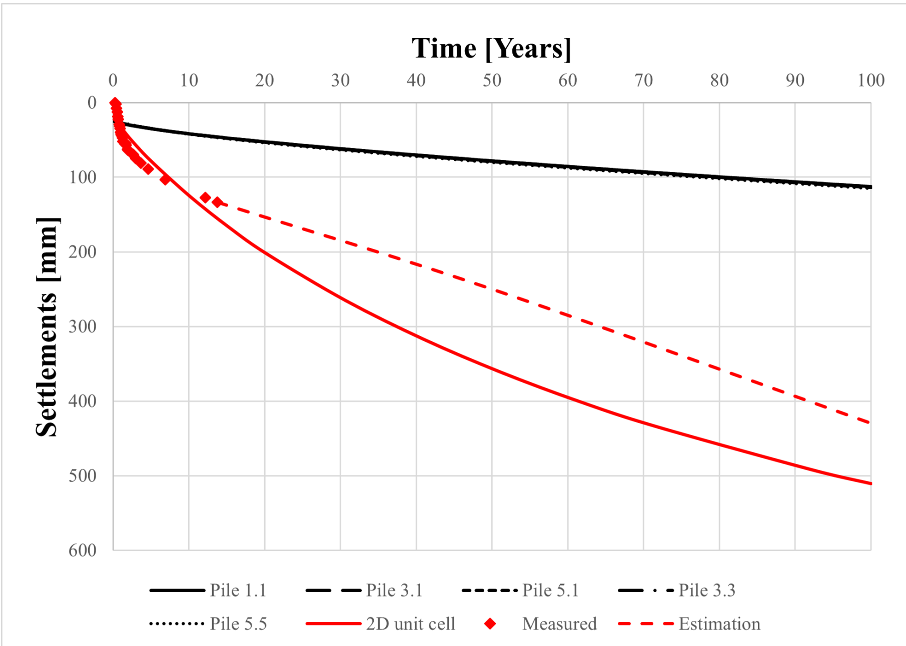

Figure F.5: Settlements for 3D study with increased distributed load to 200 kPa and 65 m edge piles.

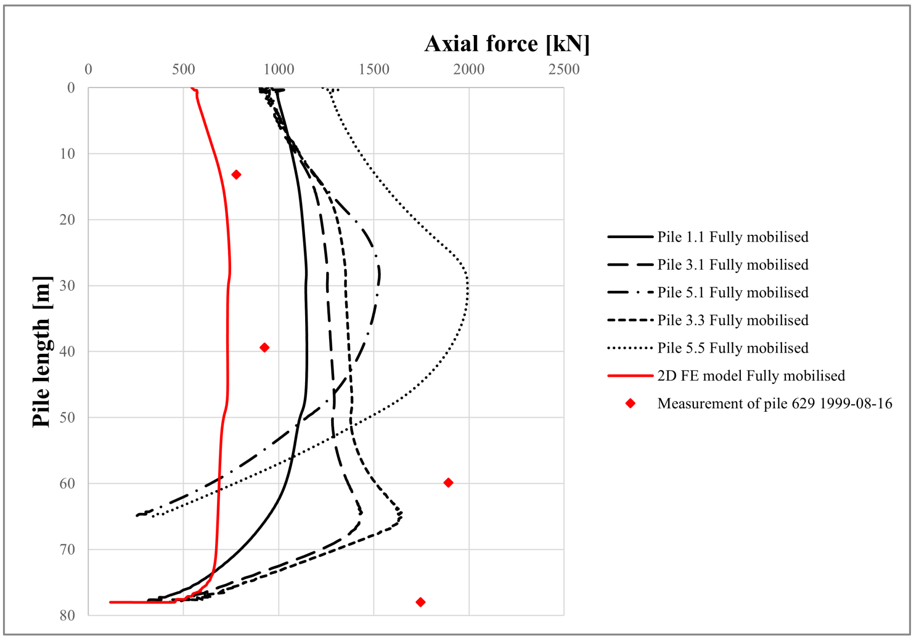

Figure F.6: Axial force for 3D study with increased distributed load to 200 kPa and 65 m edge piles.

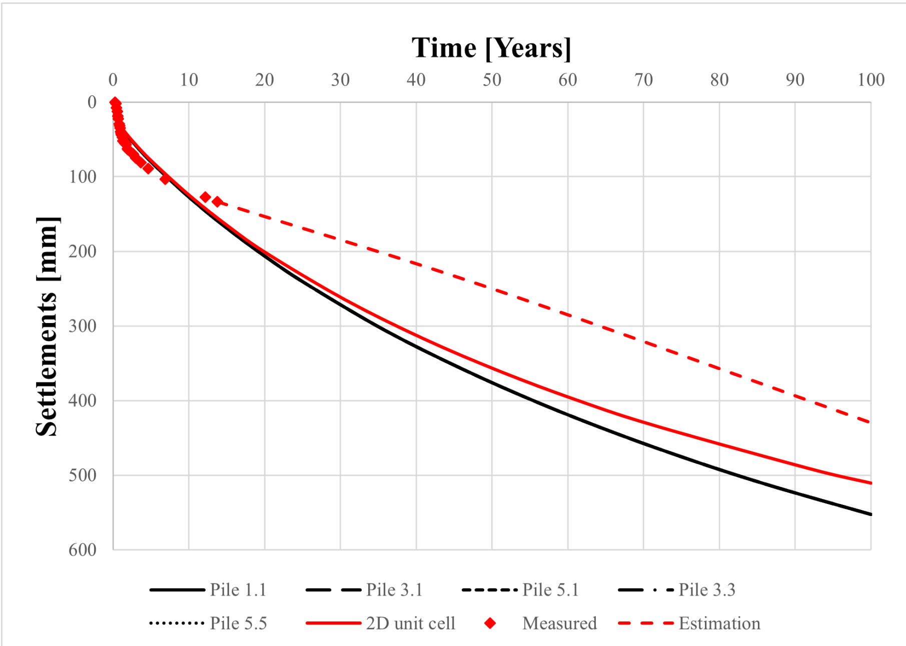

Figure F.7: Settlements for 3D study with no surrounding soil.

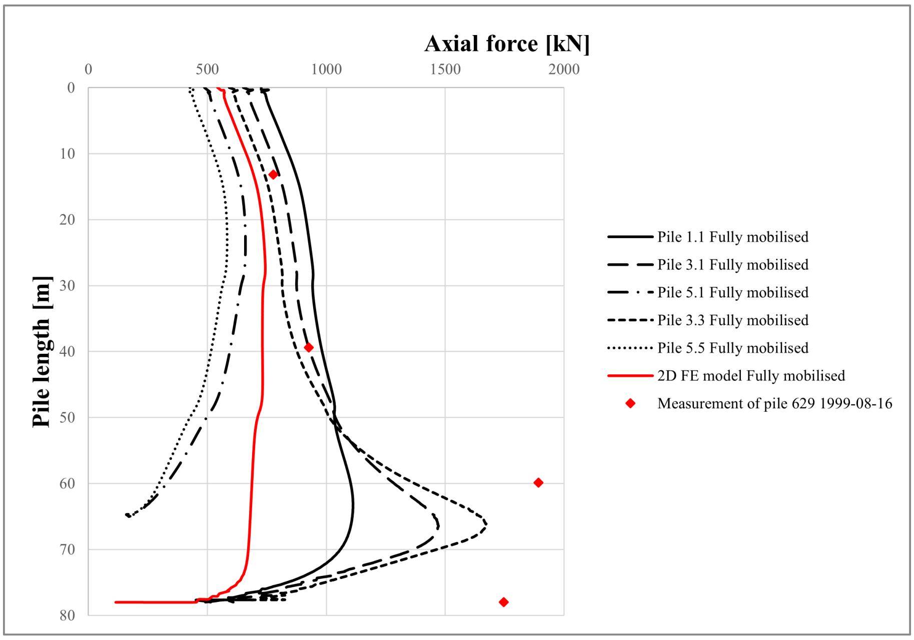

Figure F.8: Axial force for 3D study with no surrounding soil.

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