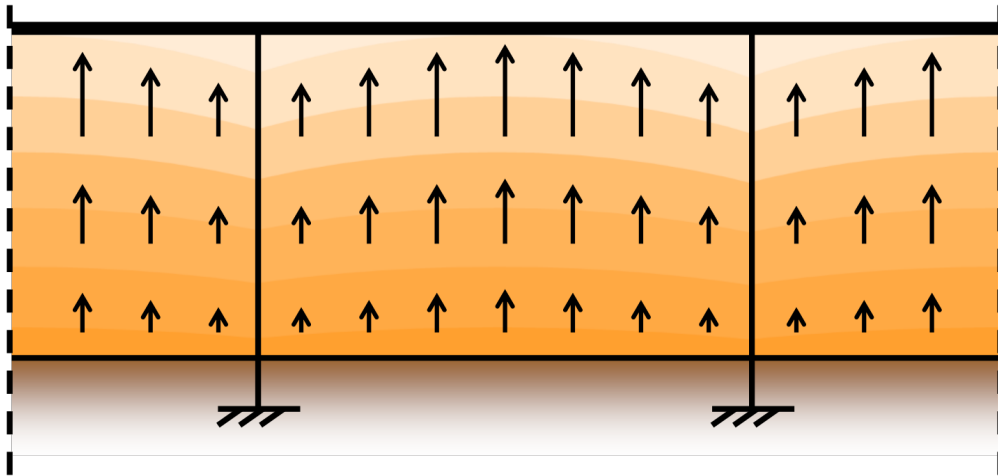




## Pressure Caused by Restrained Heave

A Study Regarding the Time-Dependent Unloading Behavior of Soft Soils

Master's thesis in Infrastructure and Environmental Engineering & Structural Engineering and Building Technology

CAROLINE BJÖRK TOCAJ  
ERIK TOLLER



MASTER'S THESIS 2017:47

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Soils

CAROLINE BJÖRK TOCAJ & ERIK TOLLER



Department of Civil and Environmental Engineering  
*Geology and Geotechnics*  
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Gothenburg, Sweden 2017

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Cover: Conceptualization of heave pressure caused by unloading, using an infinitely stiff system with cohesion along the piles.

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## Abstract

When excavating, the ground is exposed to unloading which results in heave of the soil. If the heave is restrained, due to e.g. a piled raft foundation, it may create a pressure that risks damaging structures. There are currently no design principles in Sweden taking the heave pressures into account. The aim of this study is therefore to investigate the time-dependency of heave caused by unloading and the magnitude of the heave pressures. The study is an introduction to the phenomenon rather than an actual design document.

The analyses have been carried out with the use of numerical modelling. The model has been calibrated against field measurements of pore pressure to verify the material parameters. The lock-in mechanism have been made infinitely stiff to obtain maximum heave pressures. The model have thereafter been adjusted to investigate how different parameters affect the heave pressures.

The results are obtained as the development of heave with time, the maximum heave pressures and as a pressure-unloading ratio. The maximum heave pressures are dependent on the time before the system is locked down, the stiffness of the system and the cohesion factors of the piles. The highest values of the pressure-unloading ratio are around 65% when the slab is casted immediately after excavation, decreasing to around 25% if the excavation is left open for 90 days.

There are no field measurements of the heave pressure, instead the model is calibrated against one of the most influential parameters, the pore pressure, but not the actual phenomenon. When applying the results on a more realistic case, with a less stiff system, the pressures are likely to be lower than what this study shows.

The study have concluded that the heave due to unloading is strongly time-dependent. It has been shown that the pressure-unloading ratio is decreasing with increasing excavation depth, indicating that far from all of the unloading is resulting in an upward lifting force when the system is locked down. The most influential construction parameters are the consolidation time before the slab is casted and the stiffness of the system, i.e. how locked the system is and how much of the heave that is locked in.

Keywords: Heave pressure, heave, unloading, numerical modelling, soft soil, clay



# Preface

When we first started this thesis, the confusion was real, but as the time passed we heaved up our anchor and sailed away towards softer soils. A lot of time went to trying to grasp the scope, and to separate the two phenomena of swelling and heave. When we understood the difference we felt a great unloading due to finally making the thesis more concrete. An excess pressure, which was negative, was created as the complexity of the analysis was swelling, making it essential to go back to solid foundation. When the pressure started to dissipate we encountered a new issue of restraint. We felt locked in our way of modelling, and contained within our limitations. Finally, as the report was finished, our internal pressure-unloading ratio reached zero, and all was well in the world again.

Putting our puns aside, the study have been very rewarding. Even though it sometimes have been hard, we have laughed every day and learned that when you are writing about clay, you have to make jokes. We hope that our work will be of use and that it provides a good basis for further studies. Perhaps it can enlighten the building industry to regard, and in projects where it can pose problems, account for heave pressure during design.

The subject of the master thesis was proposed by *Skanska Teknik*. We want to express our gratitude to Johannes Tornborg, our supervisor at *Skanska*, who have helped us every time we got stuck and guided us through both confusion and doubt. We also want to thank Torbjörn Edstam, who undoubtedly have more experience and knowledge in the software *PLAXIS 2D* than we do, and have helped us with everything from boundary conditions to groundwater settings. Our supervisor at *Chalmers*, Minna Karstunen, has helped us to maintain the academic character of the thesis and explained theoretical concepts. Lastly, we would like to thank Lars Fagergren and Baltzar Linde, for their thoughts regarding our work and their company during the process.

CAROLINE BJÖRK TOCAJ & ERIK TOLLER, Gothenburg, June 2017



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# List of Variables

## Greek Characters

|                  |                             |
|------------------|-----------------------------|
| $\gamma$         | = unit weight               |
| $\gamma_w$       | = unit weight of water      |
| $\delta$         | = soil element thickness    |
| $\varepsilon$    | = strain                    |
| $\varepsilon_v$  | = vertical strain           |
| $\kappa^*$       | = swelling index            |
| $\lambda^*$      | = compression index         |
| $\nu$            | = Poisson's ratio           |
| $\sigma$         | = total stress              |
| $\sigma_v$       | = total vertical stress     |
| $\sigma'$        | = effective stress          |
| $\sigma'_0$      | = in-situ effective stress  |
| $\sigma'_c$      | = preconsolidation pressure |
| $\sigma'_{mean}$ | = mean effective stress     |
| $\sigma'_v$      | = effective vertical stress |

$\Delta\sigma_{exc}$  = vertical stress distribution of the unloading

$\Delta\sigma_{fill}$  = vertical stress distribution of the fill

## Roman Characters

|             |                                         |
|-------------|-----------------------------------------|
| $a_s$       | = swelling index                        |
| $c_v$       | = coefficient of vertical consolidation |
| $h$         | = heave                                 |
| $k$         | = permeability                          |
| $l$         | = length of the slab                    |
| $p'$        | = mean effective stress                 |
| $q_{heave}$ | = heave pressure                        |
| $q_{slab}$  | = load of the slab acting on the piles  |
| $q'_s$      | = effective in-situ overburden pressure |
| $t$         | = time                                  |
| $u$         | = pore pressure                         |
| $u_e$       | = excess pore water pressure            |
| $u_s$       | = static pore water pressure            |
| $y_{ref}$   | = reference depth                       |
| $z$         | = depth                                 |
| $z_d$       | = initial total clay depth              |
| $z_{exc}$   | = excavation depth                      |
| $z_w$       | = groundwater level                     |

|            |                                     |
|------------|-------------------------------------|
| $C_0$      | = integration constant              |
| $D$        | = soil layer thickness              |
| $E$        | = elastic modulus                   |
| $E_{inc}$  | = increase of stiffness with depth  |
| $E_{ref}$  | = reference modulus                 |
| $M$        | = oedometer modulus                 |
| $M_{ul}$   | = oedometer unloading modulus       |
| OCR        | = over consolidation ratio          |
| $P_s$      | = heave pressure                    |
| $Q_{pile}$ | = axial load at the top of the pile |
| $T_v$      | = time factor                       |
| $U$        | = degree of consolidation           |

# 1

## Introduction

When excavating, the soil is exposed to unloading which results in heave of the soil. If the heave is restrained, it may create a pressure that risks damaging structures. This chapter presents a background of this problem in combination with an explanation of the scope and aim of the study. Previous studies that treats unloading and heave of soft clay are summarized in order to provide a more extensive understanding of the phenomenon.

### 1.1 Background

There are a lot of ongoing and upcoming investments of infrastructure and housing projects in the Gothenburg region. Infrastructure projects are getting larger and many foundations are situated below existing ground level. Apartment- and office buildings are becoming higher, with an increased number of basement floors in order to accommodate the growing need for housing. This will not only lead to an increased number of excavations, but also excavations being carried out to greater depths. In order to avoid damage of the foundation, and also of existing ground improvement, it is important to understand the unloading behavior of soft soils.

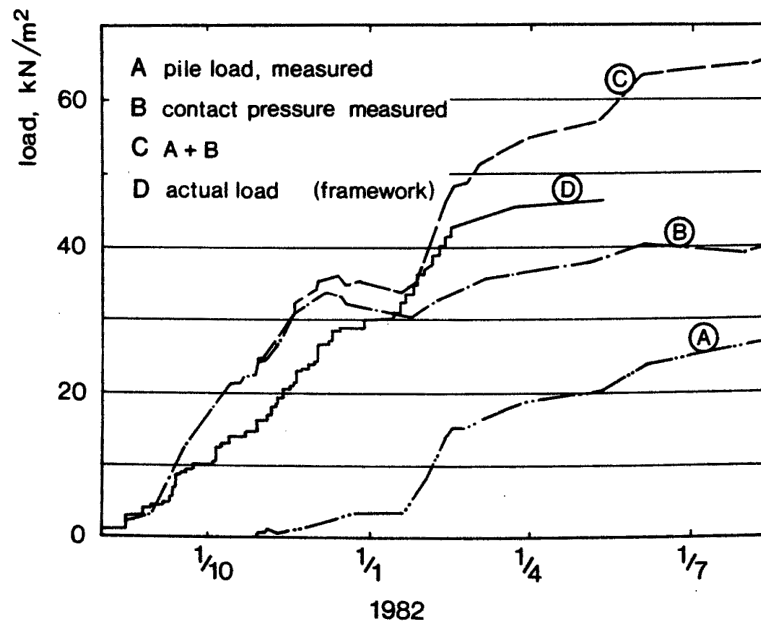
In combination with an increasing number of deep excavations, the construction times are getting shorter as a consequence of making the construction process more effective, which means that the problem of heave in clay might get more prominent. As the time shortens, the concrete foundations will be casted earlier, not leaving enough time for the clay to heave. It is, therefore, important to investigate the time-dependency of heave in order to establish guidelines for how soon after an excavation the foundation slab can be casted.

Today there are design principles considering the unloading modulus of clay, for larger infrastructural projects, such as tunnels. However, there are no design principles for taking the pressure caused by the restrained heave into account when designing underground constructions. There is an interest from the industry to develop design criteria taking these pressures and heave into account when necessary during design of underground constructions. In order to do this, there is a need of investigating the phenomena to determine: is there an issue, how does it affect the structures, and how can it be taken into account during design?

#### 1.1.1 Previous Studies

In a study regarding load transfer of friction piled foundations in soft clay it was discovered that a phenomenon referred to as locked-in heave might occur (Jendeby, 1986). During measurements of the load transfer between a raft foundation and

friction piles it was discovered that the obtained contact pressures were about 10 kPa higher than that obtained from the actual load, see Figure 1.1. This difference was assumed to be a result of the locked-in heave, occurring when the raft was casted shortly after the excavation was finished, not leaving enough time for the soil to heave. The raft prevented the soil from expanding thus resulting in a contact pressure, i.e. heave pressure, between the soil and the raft. This pressure had to be overcome before the load was transferred from the framework to the piles.



**Figure 1.1:** Actual and measured load during construction (Jendeby, 1986)

The phenomenon described by Jendeby (1986) is of greater importance today, since bottom slabs and rafts are becoming thinner in design. Earlier they were designed to carry load, whereas nowadays the pile foundations are more often assumed to carry all of the load. The heave pressures that might be obtained, as a result of the locked-in heave, may therefore govern the design and hence needs to be estimated.

The heave of excavations in the Gothenburg region have previously been studied during the construction of the garage beneath *Nya Ullevi* by Friis and Sandros (1994) and Alén and Jendeby (1996) and during the construction of *Götatunneln* by Kennedy (2003) and Persson (2004).

The scope of the study by Friis and Sandros (1994) was to determine the unloading modulus and the magnitude of the heave for deep excavations. The studied excavation had a depth of four and a half meters and a total volume of 18 000 m<sup>3</sup>. The study was performed by measuring the heave of the excavation and the variation in pore water pressures during the first part of the construction period (3 months). The unloading modulus was thereafter calculated for field conditions, by laboratory experiments and by empirical formulas. The study concluded that the unloading modulus obtained from empirical formulas were very low, while the modulus obtained from laboratory experiments and the unloading modulus obtained from field

measurements were much greater. The survey was thereafter continued by Alén and Jendebý (1996) for two more years, where they investigated and monitored the pore pressure change and heave.

Kennedy (2003) conducted a comparison between the vertical deformations measured during construction of a part of *Götatunneln* with those calculated by numerical modelling. The numerical models were set up in *PLAXIS 2D* and calculated by the use of the Mohr-Coulomb and the Hardening Soil material models. A parametric study was also conducted in order to determine the effect of variations in different material parameters. The study concluded that there are difficulties with determining the unloading modulus of clay, since empirical formulas, laboratory testing and field measurements give different results. Furthermore, the different material models in *PLAXIS 2D* gave similar results, but the Hardening Soil material model needed much higher accuracy of the input parameters.

The study by Persson (2004) aimed to investigate the unloading behavior of soft clay in order to reduce existing uncertainties. The field measurements were conducted during the excavation of a part of *Götatunneln* and extensive measurements regarding the heave and pore pressures were obtained during a year of construction. The unloading modulus was estimated from the obtained data and compared with laboratory experiments. Curves for the unloading modulus for normally to slightly overconsolidated clays were presented based on the comparison of field and laboratory data. It was also shown that the heave process, for the specific excavation, only was ongoing for three months after the finishing of the excavation and terminated at the same time as steady-state pore pressures were obtained. Nevertheless, the short heave process could be a result of the ongoing piling at the construction site.

### 1.1.2 Current Practice

There are no Swedish guidelines, and normally no criteria from the client to account for heave pressures during house constructions. However, during large infrastructural projects in Sweden, such as tunnels, there have been examples of guidelines regarding heave pressure presented in the tender documents. Heave pressures were regarded during the design and construction of *Götatunneln* according to the tender documents provided by the Swedish Road Administration (Vägverket, 2001). The following equation was used

$$P_s = \left(0.25 + 0.15 \frac{D}{30}\right) q'_s, \quad < 0.4q'_s \quad (1.1)$$

where  $q'_s$  is the effective in-situ overburden pressure on the foundation level and  $D$  is the clay layer thickness below the foundation level.

Since there are normally no guidelines provided, different contractors have different ways of approaching the issue. For example, the contractor *Skanska Sverige*, has experience from earlier projects where the heave caused by unloading have resulted in problems, as well as projects where it has not posed a problem. *Skanska Sverige* has therefore created internal guidelines on how to account for the pressures that might develop, in order to ensure that this phenomenon does not pose a problem for both infrastructure- and housing projects. *Skanska Sverige* has created a course for their structural engineers called *Geoteknik för huskonstruktörer*,

i.e. Geotechnics for Structural Engineers, where the phenomena of heave and pressures from restrained heave are described. The course presents simplified guidelines, regarding heave pressure and the time-dependency of the heave. It is stated that the heave pressure that should be regarded for design is equivalent to the remaining effective in-situ vertical stress at the foundation level which has not had the time to heave. As a rough estimation the pressure can be assumed to 40% of the effective in-situ stress at the foundation level. This pressure is then accounted for in design in combination with the mean water pressure against the slab. The heave, and thus heave pressure, is dependent on:

- The time,  $t$ , that the excavation is open before the foundation slab is casted together with the piles
- The permeability,  $k$
- The oedometer unloading modulus,  $M_{ul}$
- The coefficient of consolidation,  $c_v$
- Previous construction work, e.g. piling
- The excavation depth
- The clay layer thickness,  $D$

where

$$c_v = \frac{kM_{ul}}{\gamma_w} \quad (1.2)$$

and the time factor is

$$T_v = t \frac{c_v}{(D/2)^2} \quad (1.3)$$

Based on these parameters, the degree of consolidation,  $U$ , can be determined from the graph of the relationship between the average degree of consolidation and the time factor, which is a known graph based on Terzhagi's theory of one-dimensional consolidation (Knappett and Craig, 2012). This gives that the pressure can be determined as the remaining consolidation potential  $(1 - U)$  after the foundation slab has been casted. If the excavation is open for an extensive time period,  $U$  will be equal to one and the heave pressure will be zero.

## 1.2 Objective & Aim

The objective of this study is to investigate the time-dependency of heave caused by unloading, and the magnitude of the pressures that occur due to containment of this heave. The following questions are to be answered:

- How does the heave caused by unloading develop over time?
- If the heave is restrained, can this develop a pressure against a slab?
- Which material- and system parameters affect these possible pressures?
- Is there a relation between the created pressures and the amount of unloading?

The aim of this research project is to investigate the phenomena of heave and heave pressures caused by unloading of soft soils. Furthermore, the sub-aim is to provide a review of the pressures that might occur, the magnitude of these and the effect that they have on overlaying structures. The study aims to conclude in an introduction to the questions rather than an actual design document.

## 1.3 Method

To provide a basis for answering the aim, the research has been conducted in two parts: a literature survey and numerical analyses with finite elements. The intention of the literature survey is to provide a theoretical background to the issues and to the existing, available, knowledge of the phenomenon. Following the theoretical background was the development of a numerical model calibrated against field measurements of pore pressure change. For validation, semi-empirical calculations were performed for the stress states and groundwater conditions as a complement to the numerical analyses.

## 1.4 Limitations

The parameter setup for the numerical analyses are calibrated against measurements of pore pressure and heave during the construction of the garage underneath the Ullevi Stadium. The material properties used are collected from the geotechnical investigations performed prior to the construction of the garage.

The numerical software used is *PLAXIS 2D* and the model has been made as a simplified symmetrical case representing one-dimensional behavior. The groundwater table have been lowered, in the numerical analyses, from its original level of one meter below the surface to the bottom of the excavation for all excavation depths. Furthermore, the effect of creep has not been accounted for.

The system parameters that have been investigated and varied are: the clay depth, the excavation depth, the cc-distance between the piles and the cohesion factor of the piles. The clay depth has been investigated by changing the original clay depth of 35 m into 70 m, i.e. doubling the depth. The cohesion factors have been set to 0 and also calculated for a realistic case. The cc-distance has been chosen to 4 and 8 m. No other adjustments have been made for investigating the effect of each parameter. The effect of construction activities, such as piling, have been disregarded. The system has initially been considered to be infinitely stiff, and thereafter adjusted to realistic stiffness for the piles and the concrete slab.

The result of the heave pressure is based on the numerical analyses performed and have not been validated by field measurements. The pressures considered are those caused at the time when the negative excess pore pressures approaches zero, i.e. when the pore pressures have dissipated due to consolidation.

## 1.5 Terminology

The following concepts are commonly used throughout the report and thus explained for clarification.

|                                 |                                                                                                                                                                    |
|---------------------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| <b>Slab</b>                     | The concrete slab that is casted on top of the clay at the bottom of the excavation, i.e. the foundation slab.                                                     |
| <b>Heave</b>                    | An upward movement of the soil, referring to heave caused by unloading, i.e. by excavating soil, unless otherwise stated.                                          |
| <b>Heave pressure</b>           | The pressure that is caused on a slab which confines an ongoing heave, i.e. pressure caused by restrained, locked-in or contained heave.                           |
| <b>Pressure-unloading ratio</b> | The ratio between the heave pressure and the in-situ vertical effective stress at the excavation level, i.e. the weight of the unloading caused by the excavation. |
| <b>Swelling</b>                 | A volume increase of clay due to the inflow of water.                                                                                                              |
| <b>Swelling pressure</b>        | The pressure that is caused by swelling of expansive clay, related to the mineral composition and the crystalline structure due to change in water content.        |

# 2

## Unloading Behavior of Clay

This chapter explains the theory connected to the heaving and swelling of clay. There are two types of swelling in clay that are discussed, one that is related to the crystalline mineral structure, and one that is caused by unloading. The focus of this report is on the type of swelling and heave caused by unloading.

### 2.1 Basic Concepts of Soil Mechanics

Soil is composed of three parts: solids, liquids and gases (Budhu, 1999). These parts provide the soil with its physical properties. In the Gothenburg region, soft soils are usually fully saturated, and therefore consist solely of solids and liquids.

The stress acting on a soil specimen is called total stress,  $\sigma$ . The stress can be divided into two parts: the effective stress,  $\sigma'$ , and the pore pressure,  $u$ . The effective stress is the stress which acts between the solid particles in the soil and can be expressed as

$$\sigma' = \sigma - u \quad (2.1)$$

The deformation of soils are a function of these effective stresses (Budhu, 1999). The pore water pressure is the pressure induced by the water in the pores of the soil (Knappett and Craig, 2012). This pressure is often equal to the water head, unless the stress state of the soil is changed. The flow velocity of water to and from the pores is governed by the permeability,  $k$ , of the soil.

The elasticity of the soil represents how the material strive to return to its original state when loaded or unloaded (Knappett and Craig, 2012). This can be conceptualized as a spring where Hooke's law apply. Thus the elastic modulus,  $E$ , can be expressed as

$$E = \frac{\Delta\sigma}{\Delta\varepsilon} \quad (2.2)$$

where  $\varepsilon$  is the strain of the specimen. However, it is important to note that the elastic modulus is stress dependent.

### 2.2 Pore Pressure Change Due to Unloading

When loading or unloading a soil there is a change of total stress and the corresponding response in effective stress is dependent on the change in pore water pressure (Knappett and Craig, 2012). The initial pore water pressure is constant at a static pore water pressure,  $u_s$ , which is governed by the level of the groundwater. When there is an increase in total stress,  $\Delta\sigma$ , the solid particles try to rearrange. Due to the soil being laterally confined and that water is incompressible, it is not possible

for the particles to rearrange unless water is dissipated. Instead, the pore water pressure increases. This increase is known as excess pore water pressure,  $u_e$ . The increase of pore water pressure is equal to the increase in total stress, under the condition that the lateral strain is zero. This can be assumed if the area over which the change of total vertical stress takes place is large compared to the depth of the clay layer.

Due to the increase in pore water pressure, a hydraulic gradient is created (Knappett and Craig, 2012). Drainage of the soil will continue until the pore water pressure becomes equal to the value governed by the groundwater table, i.e. the static pore water pressure. If the groundwater table has changed due to loading, unloading or time, the excess pore water pressure should be referenced to the new groundwater table. The total pore water is expressed as

$$u = u_s + u_e \quad (2.3)$$

As the excess pore water pressure dissipates, the effective vertical stress increases and the volume decreases.

The same analogy can be used for unloading the soil (Persson, 2004). When a soil is unloaded, e.g. during excavations, the total normal stress is decreased. The pore water pressure will initially be reduced resulting in negative excess pore water pressure. As described by Budhu (1999) it is this pore pressure that is limiting the swelling of the soil. The soil will, opposite of consolidation, swell as the pores are filled with water. This swelling will prolong until the pore pressure reaches equilibrium, i.e.  $u_e \rightarrow 0$ , and the effective normal stresses will decrease correspondingly. Morissette et al. (1996) shows that the major factor affecting the negative pore pressure is the permeability of the soil. They also showed that there is no trend connecting the rate of an excavation with the magnitude of the developed negative pore pressures.

When the lateral strains are not zero, the magnitude of the pore water change has to be calculated in relation to the change of total vertical stress. According to Persson (2004), the pore pressure change after the excavation is normally calculated with

$$\Delta u = \Delta \sigma_v \frac{1 + \frac{2\nu}{1-\nu}}{3} \quad (2.4)$$

where  $\sigma_v$  is the total vertical stress. The final pore pressure distribution, at steady state, is normally assumed to be linearly distributed with depth from the lowered groundwater level to the initial pore pressure at the bottom.

The pore pressures will increase with time to the steady state conditions, and the effective stresses will decrease with time to steady state conditions, if the pore pressure decrease caused by the unloading is greater than the decrease caused by the lowering of the groundwater level (Persson, 2004). This will result in swelling of the soil and a time-dependent heave. However, if the pore pressure change from unloading is equal to, or smaller than, the pore pressure change from the groundwater lowering there will not be a time-dependent heave, instead there will be a settlement.

## 2.3 Heave

Knappett and Craig (2012) explains base heave as consisting of two different parts: one part caused by the soil outside of the excavation acting as a surcharge pushing the bottom of the excavation upwards, and one part caused by the swelling if the excavation remains unloaded for any period of time. However, according to existent Eurocode (SS-EN 1997-1, 2005) for designing geotechnical constructions there are three different types of heave that should be analyzed separately during design. Heave caused by:

1. A reduction of effective stresses
2. A volume expansion of partly saturated soil
3. Settlements of adjacent structures

According to Karlsson and Moritz (2016a) it should also be assumed that the heave caused by piling is equivalent to the total mass volume that has been pressed down below existing ground surface. Broms and Hansbo (1981), on the other hand, stated that for a pile group it can be shown that the heave will be around 40-60% of the total volume.

A similar division of different types of heave is made by Zeevaert (1983): driving heave, elastic heave, swelling heave and plastic heave. The driving heave is caused by pile driving and Zeevaert (1983) states that a soil volume equivalent to the pile volume displaces instantly. The elastic heave is caused by unloading and can be estimated if the elastic compressibility of the subsoil is known. The elastic heave occurs instantly during excavating and foundation construction, and is hence largely associated with shear strains. If the excavation is unloaded for any extent of time the elastic heave proceeds into a swelling heave.

Zeevaert (1983) presents a few methods to reduce the total heave during construction where emphasis is put on reducing the total change in effective stresses to a minimum. This can be done by, for example, lowering the water table, or constructing the excavation in stages, re-applying load at the same time as removing load which will suppress the elastic heave and thereby also reduce the swelling heave. The elastic heave and the swelling heave are also considerably reduced by ground improvements in the form of piles or deep mixing.

## 2.4 Swelling of Clay

When referring to swelling of clay it is important to specify which type of swelling that is referred to. One type of swelling occurs in expansive soils, as condition two in SS-EN 1997-1 (2005), and is related to the mineral composition and the crystalline structure of clay particles. Examples of countries that have notable problems with expansive clay are Canada, United States and China, with damages at a cost of billions of dollars worldwide annually (Phanikumar and Singla, 2016). Similar to, but different from, the type of swelling occurring in expansive clay is swelling caused by unloading, as condition one in SS-EN 1997-1 (2005). This is the type of swelling

currently known to occur in clay in the Gothenburg region.

Foundations, piles and slabs are all affected by heave- and swelling pressures which is why the uplifting force have to be considered in design (Ashayeri and Yasrebi, 2009). In order to avoid damage there is a need to understand the driving forces of the phenomenon of swelling, and thereby to understand the swelling- and heave pressure development in situations that can represent field conditions (Massat et al., 2016).

### 2.4.1 Swelling in Expansive Clay

The characteristics of expansive soils are that they swell when they absorb water, and they shrink when the water is evaporated (Phanikumar and Singla, 2016). The swelling of a soil due to the increase in moisture content can be expressed as the change in the void ratio in relation to the moisture content (Hanafy, 1991). It is only the volume of the void that changes in the moisture dependent swelling while the volume and weight of the solid soil particles remains. Furthermore, the swelling behavior due to a change of moisture content of a clay can be determined through laboratory testing e.g. free swelling test.

According to Briaud (2013) the swelling pressure of an expansive clay can be measured by a confined swell test where the soil is given access to water. Eventually the specimen will reach an equilibrium with swelling pressures that can be in the magnitude of 1 000 kPa or higher for high-plasticity clay. This type of swelling of clay is a function of two main processes: crystalline swelling and osmotic swelling (Massat et al., 2016). The crystalline swelling spans from a dry state to a more saturated one and it only occurs in an under saturated system. The water molecules are absorbed on to the unit layer surfaces and the interlayer cations.

### 2.4.2 Swelling Caused by Unloading

Swelling caused by unloading occurs when the effective stresses in the clay are decreased (Larsson, 1986). The magnitude of the decrease, i.e. the load reduction, will determine whether the clay will start to swell or not. Karlsrud and Hernandez-Martinez (2013) states that clay will start to swell after unloading. The swelling modulus of clay, also known as the unloading modulus, is normally high which according to Karlsrud and Hernandez-Martinez (2013) will result in the first phase of the swelling and effective stress reduction occurring within a few months following the finished excavation.

Swelling in clay can be explained as the reverse of consolidation (Knappett and Craig, 2012) as explained in Section 2.2. When the normal stresses decreases, the soil skeleton will expand causing a reduction of pore pressure and thereby negative excess pore pressure. This will allow water to flow into the soil, causing a reduction of the effective normal stresses, resulting in a volume increase.

## 2.5 Unloading Modulus

Multiple research studies have looked into the unloading behavior of soft soils and present different ways of calculating the unloading modulus. There seem to be no general consensus regarding which formulation to use for the unloading modulus, and the difference between formulations based on laboratory tests and on field measurements are substantial.

The unloading modulus decreases with decreasing effective stress and can according to Larsson (1986) be described by the following formula

$$M_{ul} = \frac{\sigma'_v}{a_s} \quad (2.5)$$

where

- $M_{ul}$  = unloading modulus
- $\sigma'_v$  = effective vertical stress
- $a_s$  = swelling index (0.007 - 0.012 for Swedish soft clays)

The unloading modulus is higher for stresses close to the preconsolidation pressure (Larsson, 1986). This is due to the turning point where the secondary compression is overcome by the secondary swelling, resulting in a change in direction of the secondary deformations.

According to Alén and Jendeby (1996), Equation 2.5 underestimates the unloading modulus and hence overestimates the heave since the empirical formula have been developed in laboratory settings, by oedometer tests. A possible explanation for this is that the oedometer test does not resemble a real case where the clay has been deformed by creep for a long period of time. The creep deformations can probably not be reverted by unloading and the heave will therefore be overestimated.

The unloading modulus has also been studied by Karlsrud (2003) who carried out oedometer tests in order to observe the unloading and reloading behavior of clay. The tests concluded that the unloading modulus are dependent on the magnitude of the unloading and also the preconsolidation pressure. The following formula was derived

$$M_{ul} = 250\sigma'_v \left( \frac{\sigma'_v}{\sigma'_c - \sigma'_v} \right)^{0.3} \quad (2.6)$$

where

- $\sigma'_v$  = effective vertical stress
- $\sigma'_c$  = preconsolidation pressure

Persson (2004) performed laboratory tests as well as field measurements on the construction of a part of *Götatunneln*, in order to gain understanding about the unloading behavior of soft clay. The study concluded that the initial unloading modulus is high but decreases with a decrease in effective stress. According to the results, the unloading modulus can be represented by the following equation

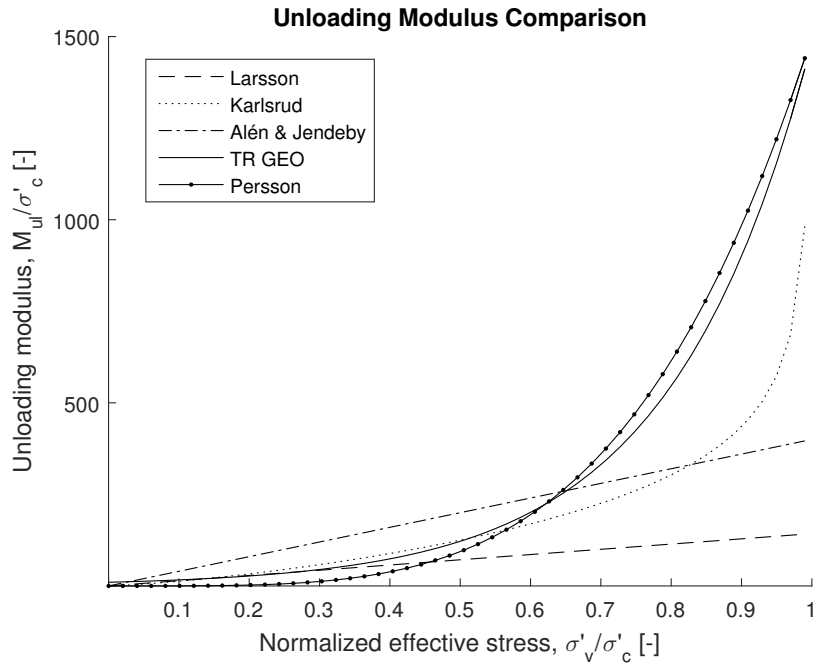
$$M_{ul} = 1500\sigma'_c \left( \frac{\sigma'_v}{\sigma'_c} \right)^4 \quad (2.7)$$

The formula has been derived on the basis of the field measurements and the laboratory tests, where all calculations are based on one-dimensional heave, the effect of piling on the heave have been disregarded and the total stress change assumed to occur momentarily. Moreover, the calculations included both shear strains related to slope movements and time-dependent swelling. If only time-dependent swelling were to be considered, the unloading modulus would increase for lower stress levels.

In *TR Geo*, technical advice on geotechnical construction (Karlsson and Moritz, 2016b), an unloading modulus based on the study performed by Persson (2004) is presented as

$$M_{ul} = 10\sigma'_c e^{5\frac{\sigma'_v}{\sigma'_c}} \quad (2.8)$$

A comparison of the unloading modulus calculated with all presented formulations is shown in Figure 2.1. The figure clearly shows that the unloading modulus vary depending on the equation used.



**Figure 2.1:** Comparison of unloading modulus, for  $\sigma'_c = 135$  kPa

### 2.5.1 Determination from Experimental Data

The Swedish Transport Administration (Karlsson and Moritz, 2016a) states, in *TK Geo*, technical requirements on geotechnical construction, that the unloading modulus should preferably be determined through laboratory testings in the form of oedometer test or triaxial tests. If the unloading modulus is to be determined from oedometer tests, one way is by having a one-dimensional oedometer test where the sample is unloaded from a correct stress state to the stress level corresponding to

the unloading in the planned excavation. Another alternative, is to do a drained triaxial test by first consolidating to the in-situ stress levels and then unloading to the appropriate stress level.



# 3

## Numerical Modelling

There are different methods available for numerical modelling where the finite element method is one. The finite element method offers approximate solutions to boundary value problems. Mathematical equations are approximated by algebraic equations and evaluated at discrete points which gives the results as nodal displacements. The accuracy of the results are dependent on the quality of the computational program, i.e. the compatibility of the constitutional models with real soil behavior, and also the assumptions and simplifications conducted by the user.

According to Muir Wood (2004) the quality of the results are only as good as the numerical approximations, and it is therefore important to verify the model. In order to use the programs in a correct way, Muir Wood (2004) imposes three questions in need of answers:

- Verification: Is the program doing what it claims to be doing?
- Are we getting the answers we think we are getting?
- Validation: Are we getting the answers we need?

It is therefore important to ensure that the program is correctly coded. If the software used is commercially available it is likely that there is a continuous correction of errors. However, it should be possible to check the constitutive models by testing individual elements. According to Muir Wood (2004) there are examples when different people have used the same software to model the same problem and obtained different results. Users are therefore encouraged to stay cautious and always double check the results for correct verification. Furthermore, the boundary conditions have to be correctly stated. If simplifications and axi-symmetry have been used, the model is only as good as the approximated boundary conditions. For soils, which are materials with non-linear history dependence, the entire modelling process needs to be considered in order to validate the results since the results only are a function of how they were obtained. Muir Wood (2004) states that all engineers using numerical modelling need to stay aware and understand that it is the user that is accountable for the results, not the software.

### 3.1 Finite Element Analysis of Soil

The behavior of a soil can be analyzed using a finite element program, e.g. *PLAXIS 2D*, which is used in this study. There are various methods for obtaining the solutions, all which are an approximation of reality. These methods are in soils defined by the behavioral properties of the soil, i.e. the material models (Brinkgreve et al., 2016).

## 3.2 Material Models

A material model is described by a set of equations relating effective stresses and strains (Brinkgreve et al., 2016). All models in *PLAXIS 2D* are defined by a relationship between effective stress rates and strain rates. According to Wong (2013), all material models also enables the permeability to change as the soil is consolidating. There are different material models available which have different levels of accuracy depending on what kind of problem is to be analyzed (Brinkgreve et al., 2016). Table 3.1 presents a classification of different soil models and their applicability with regard to type of soil, application and loading condition for the specific problem at hand in this study, according to Brinkgreve et al. (2016).

**Table 3.1:** Applicability of material models to the research setting (Brinkgreve et al., 2016)

| Model                                                                | Material:<br>NC Clay | Construction type:<br>Excavation | Loading type:<br>Unloading/Reloading |
|----------------------------------------------------------------------|----------------------|----------------------------------|--------------------------------------|
| Linear elastic model                                                 | -                    | -                                | C                                    |
| Mohr-Coulomb                                                         | C                    | C                                | B                                    |
| Hardening Soil                                                       | B                    | B                                | B                                    |
| HS Small                                                             | B                    | A                                | A                                    |
| Soft Soil Creep                                                      | A*                   | B                                | B                                    |
| Soft Soil                                                            | A*                   | B                                | B                                    |
| A: The best standard model in PLAXIS 2D for this application         |                      |                                  |                                      |
| B: Reasonable modelling                                              |                      |                                  |                                      |
| C: First order approximation                                         |                      |                                  |                                      |
| * Soft Soil Creep model in case time-dependent behavior is important |                      |                                  |                                      |

### 3.2.1 Mohr-Coulomb Model

The Mohr-Coulomb model is a simple material model approximating the soil behavior as linear elastic perfectly plastic (Brinkgreve et al., 2016). The Mohr-Coulomb material model can be used to obtain a first approximation of the soil behavior but does not account for the real non-linear behavior of soil.

### Input parameters

All parameters given as input can either be effective stress parameters (indicated by ') or undrained, total stress, parameters depending on if the analyses are to be drained or undrained (Brinkgreve et al., 2016). Needed input parameters are given in Table 3.2.

**Table 3.2:** Parameters of the Mohr-Coulomb model

| Basic Parameters    |                                      |                      |
|---------------------|--------------------------------------|----------------------|
| $E$                 | Young's modulus                      | [kN/m <sup>2</sup> ] |
| $\nu$               | Poisson's ratio                      | [-]                  |
| $c$                 | Cohesion                             | [kN/m <sup>2</sup> ] |
| $\varphi$           | Friction angle                       | [°]                  |
| $\psi$              | Dilatancy angle                      | [°]                  |
| $\sigma_t$          | Tension cut-off and tensile strength | [kN/m <sup>2</sup> ] |
| Possible Variations |                                      |                      |
| $G$                 | Shear modulus                        | [kN/m <sup>2</sup> ] |
| $E_{oed}$           | Oedometer modulus                    | [kN/m <sup>2</sup> ] |

The Mohr-Coulomb model uses a constant stiffness modulus unless advanced settings are used (Brinkgreve et al., 2016). If a constant value is to be used, it is important to choose a value that correlates to the current stress levels and the expected stress path development. In real settings, the stiffness tends to increase with increasing depth. In order to account for this, *PLAXIS 2D* offers an option where Young's modulus can be increased by depth according to the following formula

$$E(y) = E_{ref} + (y_{ref} - y)E_{inc} \quad (3.1)$$

The modulus will then take the  $E_{ref}$  value at and above the depth  $y_{ref}$  and below it will take the value  $E(y)$  adjusted by  $E_{inc}$ , which is the increase of stiffness with depth.

## Limitations

The Mohr-Coulomb material model only has a few components resembling real soil behavior (Brinkgreve et al., 2016). It does not account for stress-dependency, stress-path dependency, strain dependency or anisotropic stiffness. It is also important to note that the Mohr-Coulomb material model only uses an elastic modulus, and thus it is the same for both loading and unloading.

### 3.2.2 Hardening Soil Model

The Hardening Soil (HS) material model is an advanced model in *PLAXIS 2D* (Brinkgreve et al., 2016). It describes the soil stiffness with higher accuracy than the Mohr-Coulomb material model and requires three different reference moduli: The triaxial loading stiffness  $E_{50}$ , the triaxial unloading stiffness  $E_{ur}$  and the oedometer loading stiffness  $E_{oed}$ . The material model accounts for stress-dependency, i.e. that the stiffness increases with pressure, and for the initial stresses in the form of preconsolidation pressure and overconsolidation ratio (OCR). Some characteristics of the Hardening Soil material model are:

- The stiffness is stress dependent
- The unloading/reloading is elastic

- The Mohr-Coulomb failure criterion is used

## Input parameters

Needed input parameters for the Hardening Soil material model are given in Table 3.3.

**Table 3.3:** Parameters of the hardening soil model

| Failure Parameters               |                                                                         |                      |
|----------------------------------|-------------------------------------------------------------------------|----------------------|
| $c$                              | Effective cohesion                                                      | [kN/m <sup>2</sup> ] |
| $\varphi$                        | Effective angle of internal friction                                    | [°]                  |
| $\psi$                           | Angle of dilatancy                                                      | [°]                  |
| $\sigma_t$                       | Tension cut-off and tensile strength                                    | [kN/m <sup>2</sup> ] |
| Basic Parameters                 |                                                                         |                      |
| $E_{50}^{ref}$                   | Secant stiffness in standard drained triaxial test                      | [kN/m <sup>2</sup> ] |
| $E_{oed}^{ref}$                  | Tangent stiffness for primary oedometer loading                         | [kN/m <sup>2</sup> ] |
| $E_{ur}^{ref}$                   | Unloading/reloading stiffness (default $E_{ur}^{ref} = 3E_{50}^{ref}$ ) | [kN/m <sup>2</sup> ] |
| $m$                              | Power for stress-level dependency of stiffness                          | [-]                  |
| Advanced Parameters              |                                                                         |                      |
| $\nu_{ur}$                       | Poisson's ratio for unloading/reloading (default 0.2)                   | [-]                  |
| $p^{ref}$                        | Reference stress for stiffness (default 100 kN/m <sup>2</sup> )         | [kN/m <sup>2</sup> ] |
| $K_0^{nc}$                       | $K_0$ -value for normal consolidation                                   | [-]                  |
| $R_t$                            | Failure ratio $q_f/q_a$                                                 | [-]                  |
| $\sigma_{tension}$               | Tensile strength                                                        | [kN/m <sup>2</sup> ] |
| $c_{inc}$                        | As in Mohr-Coulomb model (default 0)                                    | [kN/m <sup>2</sup> ] |
| Alternative for Basic Parameters |                                                                         |                      |
| $C_c$                            | Compression index                                                       | [-]                  |
| $C_s$                            | Swelling index or reloading index                                       | [-]                  |
| $e_{init}$                       | Initial void ratio                                                      | [-]                  |

## Limitations

The Hardening Soil material model does not account for softening behavior and it does not distinguish between stiffness at different strains (Brinkgreve et al., 2016). The user have to choose stiffness for the dominating strain level. The Hardening Soil material model is computationally time demanding. Furthermore, the Hardening Soil material model needs higher accuracy of the input parameters (Kennedy, 2003).

### 3.2.3 HS-small Model

The HS-small material model has the same characteristics as the Hardening Soil material model but also accounts for the increase of stiffness at small strains (Brinkgreve et al., 2016).

The HS-small material model uses the same parameters as the Hardening Soil material model but two extra parameters are implemented to describe how the

stiffness varies with the strain. These parameters are the initial shear modulus,  $G_0$ , and the shear strain level,  $\gamma_{0.7}$ .

### 3.2.4 Soft Soil Model

Soft soils are characterized by a high degree of compressibility, occurring in near-normally consolidated clay, clayey silts and peat (Brinkgreve et al., 2016). The Soft Soil material model is primarily used for primary compression of normally consolidated clay and better capable of modelling compression of very soft soils. The Soft Soil material model is similar to the Hardening Soil material model which could be reasonable for most soft soil problems. However, for very soft soils the Soft Soil material model should be used. The characteristics of this model are the following:

- The stiffness is stress dependent
- There is a distinction between primary loading and unloading/reloading
- The preconsolidation stress is memorized
- The Mohr-Coulomb failure criterion is used

The Soft Soil material model assumes a logarithmic relation between the volumetric strain and the mean effective stress. It is possible to use the Soft Soil material model to account for the small strain stiffness by letting the swelling index,  $\kappa^*$ , vary within the soil cross section.

### Input parameters

All parameters are given as effective parameters, and the soil response during unloading and reloading is assumed to be elastic (Brinkgreve et al., 2016). Needed input parameters are given in Table 3.4.

**Table 3.4:** Parameters of the soft soil model

| Basic Parameters                            |                                                       |                      |
|---------------------------------------------|-------------------------------------------------------|----------------------|
| $\lambda^*$                                 | Modified compression index                            | [-]                  |
| $\kappa^*$                                  | Modified swelling index                               | [-]                  |
| $c$                                         | Effective cohesion                                    | [kN/m <sup>2</sup> ] |
| $\varphi$                                   | Friction angle                                        | [°]                  |
| $\psi$                                      | Dilatancy angle                                       | [°]                  |
| $\sigma_t$                                  | Tensile strength                                      | [kN/m <sup>2</sup> ] |
| Advanced Parameters (calculated by default) |                                                       |                      |
| $v_{ur}$                                    | Poisson's ratio for unloading/reloading               | [-]                  |
| $K_0^{nc}$                                  | Coefficient of lateral stress in normal consolidation | [-]                  |
| $M$                                         | $K_0^{nc}$ -parameter                                 | [-]                  |

#### **Limitations**

The Soft Soil material model is only suitable for soft soils that are normally- or near normally consolidated (Brinkgreve et al., 2016). Furthermore, it is most suitable for situations dominated by compression and it does not account for secondary deformations such as creep.

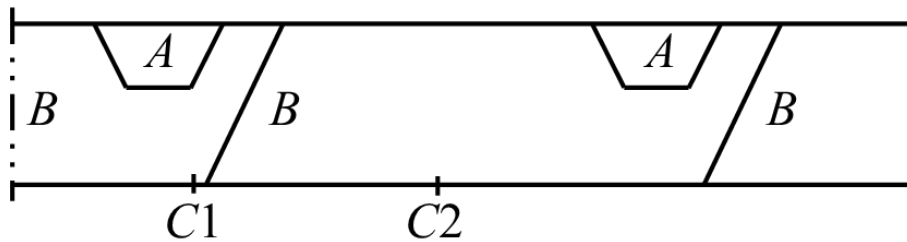
# 4

## Calibration Object

During the construction of the garage underneath *Nya Ullevi Stadium* in 1994 there was measurements of the pore pressure changes caused by the excavation and also the resulting heave. These measurements have been used for calibration of the numerical models, by trying to resemble the field conditions during the construction. The excavation at *Ullevi* was chosen since it is a wide excavation, resembling a one-dimensional situation. Due to this, the geometry of the excavation will influence the results in a low extent. The installed piles were bored piles instead of ground-displacement piles, which means that the piling had little effect on heave and pore pressure, thus making it easier to interpret the field measurements.

### 4.1 Background

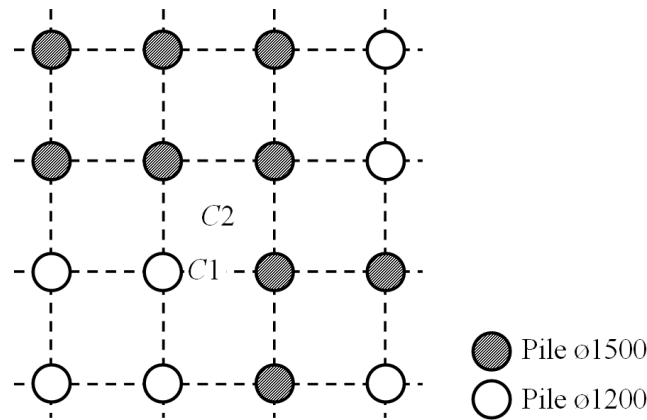
The excavation was carried out to a depth of 4.5 m, with a width of 100 m and a length of 180 m resulting in a total area of 18 000 m<sup>2</sup> (Friis and Sandros, 1994). The excavation stages are shown in Figure 4.1, where *A* is the primary excavations, *B* is the secondary excavations, *C1* and *C2* are measurement locations. After the secondary excavation, a 0.5 m layer of fill was placed at the bottom of the excavation. The depth to bedrock varies in the area from 10 - 60 m. The depth at the measurement locations *C1* and *C2* is about 35 m. The primary and the secondary excavation have, in this study, been approximated as one excavation carried out over the entire width down to 4.5 m and then a layer of 0.5 m fill have been added.



**Figure 4.1:** Cross section of excavation plan for *Ullevi* with measurement locations *C1* and *C2*

The foundation of the garage was done by the installation of bored piles placed at a cc-distance of 15 m. The piles have a diameter of 1 200 or 1 500 mm and have been casted into the underlying bedrock. A top view of the piles in the center of the excavation is shown in Figure 4.2 together with the measurement locations *C1* and

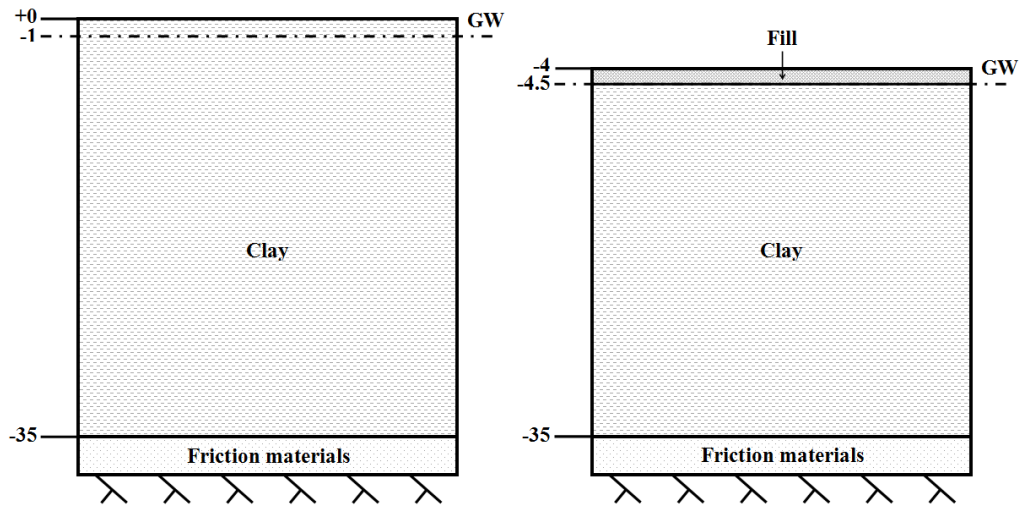
C2. Prior to the construction of the garage, during the construction of the stadium, the ground was stabilized with lime-cement columns (Johansson, 1992).



**Figure 4.2:** Top view of piling plan for *Ullevi* with measurement locations *C1* and *C2*

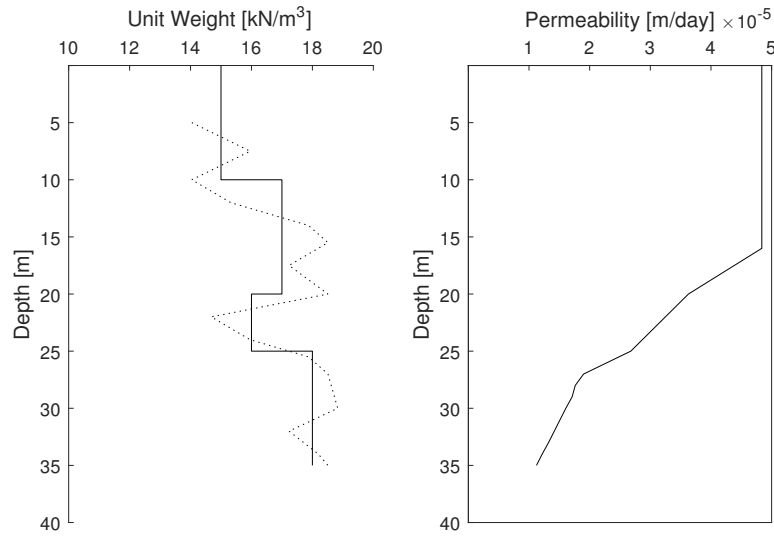
## 4.2 Ground Conditions

The soil profile is presented in Figure 4.3 where the initial ground level have been chosen as the zero-level.

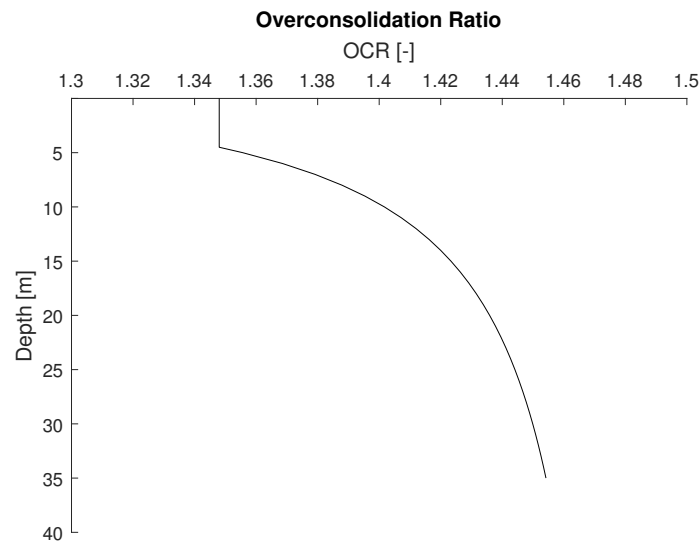


**Figure 4.3:** Cross section of soil profile before and after excavation

The density and the permeability of the clay layers have been evaluated on the basis of the geotechnical investigations presented in Friis and Sandros (1994). Figure 4.4 shows the variation with depth. The unit weight has been simplified in four intervals. The overconsolidation ratio has been determined from the geotechnical investigations by Johansson (1992) and varies with depth according to Figure 4.5. Available oedometer tests are presented in Figure A.1 in Appendix A and have been used to determine the oedometer modulus for unloading.



**Figure 4.4:** Simplified ground profile for *Ullevi*



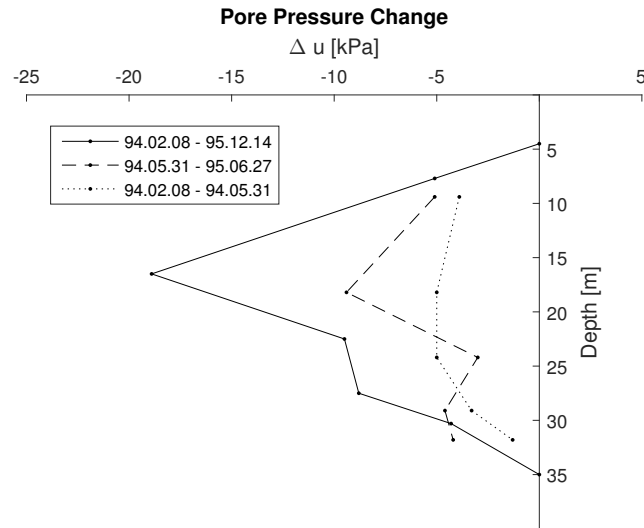
**Figure 4.5:** Simplified overconsolidation ratio variation with depth

### 4.3 Field Measurements

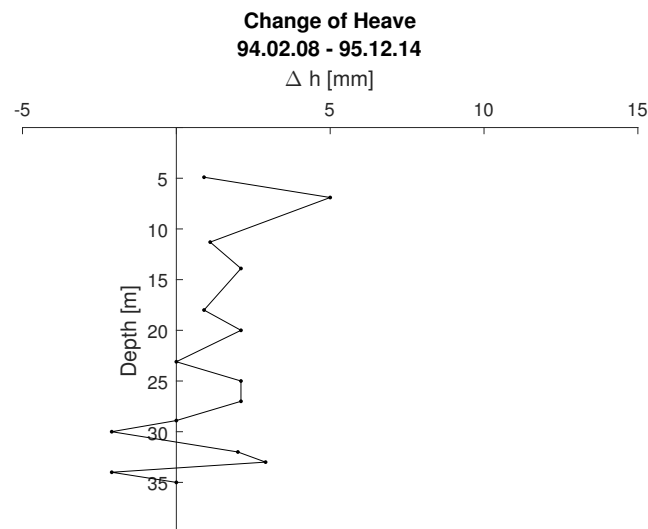
The large excavation caused an unloading of the subsoil of about 70 kPa. This, in combination with the pore pressure decrease due to drainage of the garage, would according to Alén and Jendeby (1996) result in a decrease in effective stress of 20 kPa right below the excavation and 60 kPa at a depth of 30 m. The groundwater level was initially located 1 m below the ground level and lowered to the bottom of the excavation.

#### 4. Calibration Object

The pore pressure and heave measurements that have been used for calibration of the numerical models are shown in Figure 4.6 and Figure 4.7. The calibration has been based on the measurements conducted by Alén and Jendeby (1996), since the time series was longer than the one conducted by Friis and Sandros (1994). Moreover, the measurements by Alén (1998) were conducted after the construction, and thus less affected by construction activities.



**Figure 4.6:** Measured pore pressure change at C2 for the three different time intervals



**Figure 4.7:** Measured heave change at C2 for the time period of 1994.02.08 to 1995.12.14

# 5

## Method

This chapter presents the methods used for obtaining the time-dependency of heave and the heave pressures, i.e. pressures caused by restrained heave. As a start, semi-empirical calculations have been performed with regard to the stress states of the soil, pore pressures, unloading modulus and the maximum heave. After this, numerical models have been set up and calibrated against the field measurements performed after the construction of the garage at *Ullevi*. This has been done for validation of the soil parameters used as input in the numerical analyses. After obtaining a calibrated parameter setup, the numerical model has been adjusted for calculations of heave pressure. From this point, it is only the material properties and geometry from the calibration site at *Ullevi* that are used in the following calculation steps.

### 5.1 Semi-Empirical Calculations

Semi-empirical calculations have been performed in order to determine the initial and final stress states for the Ullevi excavation. The pore pressure and stresses have been compared to numerical calculations as a verification method of the numerical models. The unloading modulus has been chosen based on existing theory and the heave has been calculated semi-empirically in order to compare with numerical results.

#### 5.1.1 Pore Pressure

The initial pore water pressure,  $u_{s, initial}$ , is governed solely by the initial groundwater head,  $z_w$ , and calculated as

$$u_{s, initial} = \gamma_w(z - z_w) \quad (5.1)$$

where  $\gamma_w$  is the unit weight of water and  $z$  is the depth. The pore water pressure right after the excavation is composed of two components,  $u_{s, initial}$  and  $u_e$  which is the excess pore water pressure. The pore water pressure right after the excavation,  $u_{new}$ , can be calculated with

$$u_{new} = u_{s, initial} + u_e \quad (5.2)$$

where the excess pore water pressure according to Knappett and Craig (2012), as explained in Section 2.2, is equal to the change in mean stress, i.e.

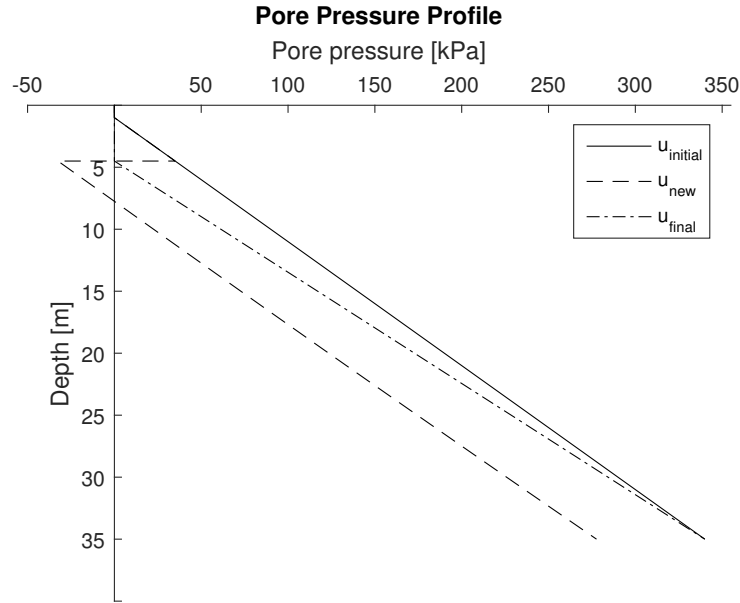
$$u_e = \Delta\sigma_{exc} \quad (5.3)$$

This is valid for excavations where the area is large compared to the depth, as for the Ullevi excavation.

At steady state conditions, the groundwater head has been lowered to the bottom of the excavation and all of the excess pore water pressures have dissipated. At *Ullevi*, there are friction materials underneath the clay layers assumed to act as a constant groundwater source. This means that the groundwater head at the bottom of the clay layer will have a constant head. The pore water pressure at steady state conditions has therefore been calculated as

$$u_{final} = \gamma_w \left( z - z_{w_{new}} \right) \frac{z_d - z_w}{z_d - z_{w_{new}}} \quad (5.4)$$

where  $z_{w_{new}}$  is the new groundwater level and  $z_d$  is the initial total clay depth. The analytically calculated pore water conditions for *Ullevi* are shown in Figure 5.1.



**Figure 5.1:** Pore pressure variation with depth at *Ullevi*

### 5.1.2 Stress State

In order to determine the stress state variation of the soil, the effective stresses at different times are needed. The initial effective stress,  $\sigma'_0$ , have been calculated with

$$\sigma'_0 = \sigma - u_{initial} \quad (5.5)$$

where

$$\sigma = \gamma z \quad (5.6)$$

and  $\gamma$  is the unit weight of the soil. The preconsolidation pressure,  $\sigma'_c$ , has been calculated as

$$\sigma'_c = \text{OCR} \cdot \sigma'_0 \quad (5.7)$$

where OCR is the overconsolidation ratio which varies according to Figure 4.5.

The effective stress immediately after the excavation has been assumed to be unchanged, due to the change of total normal stress being equal to the decrease of excess pore pressure as shown in Equation 5.3. The stress state immediately after the excavation,  $\sigma'_{new}$ , can therefore be determined as

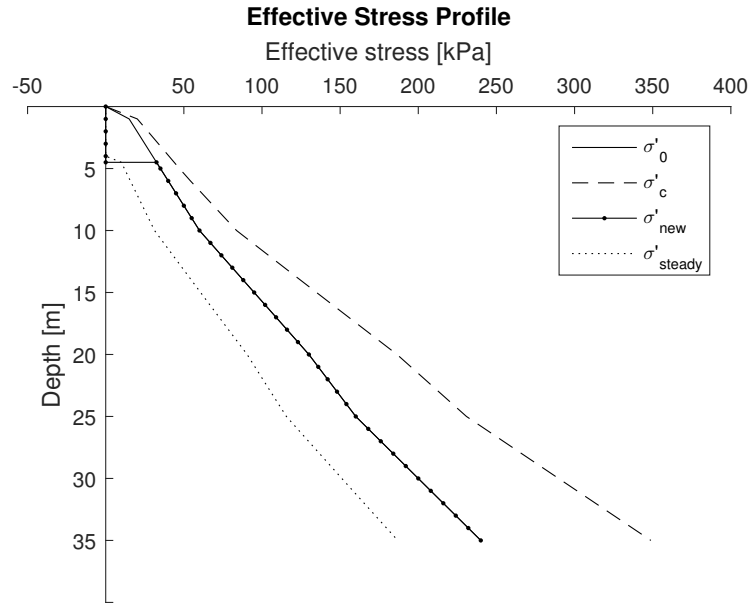
$$\sigma'_{new} = \sigma + \Delta\sigma_{exc} - u_{new} \quad (5.8)$$

where the vertical stress distribution of the excavation,  $\Delta\sigma_{exc}$ , is calculated with Boussinesq equation for stress distribution with depth.

At steady state conditions, the groundwater have been lowered as explained in Section 5.1.1 and the excess pore water pressure has dissipated. The effective stress that the soil will have at steady state,  $\sigma'_{steady}$ , has been calculated as

$$\sigma'_{steady} = \sigma + \Delta\sigma_{exc} + \Delta\sigma_{fill} - u_{final} \quad (5.9)$$

where the vertical stress distribution of the fill,  $\Delta\sigma_{fill}$ , is calculated with Boussinesq equation for stress distribution with depth. The analytical stress profiles for the specific conditions at *Ullevi* are shown in Figure 5.2.

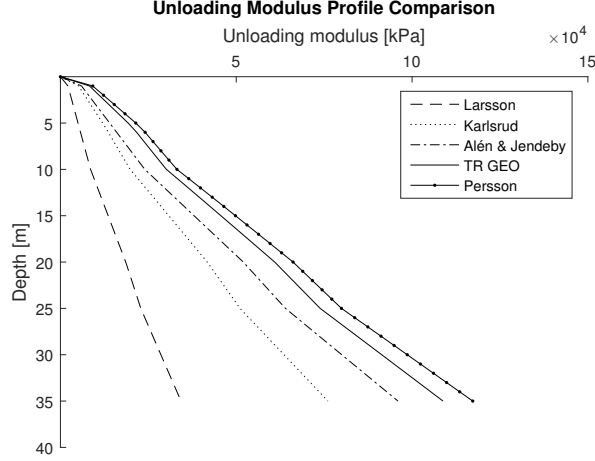


**Figure 5.2:** Effective stress variation with depth at *Ullevi*

### 5.1.3 Unloading Modulus

As explained in Section 2.5, there is no general consensus in Sweden of how the unloading modulus should be determined. The comparison in Figure 2.1 shows that the unloading modulus according to Persson, Karlsrud and *TR Geo* (Karlsson and Moritz, 2016b) has an exponential increase whereas the others are linear. In order

to clarify the differences further, the variation of the unloading modulus with depth has been calculated for the specific conditions at *Ullevi* with the use of all equations presented in Section 2.5. Figure 5.3 shows the unloading modulus according to all presented empirical equations, calculated for the initial stress state at *Ullevi*.



**Figure 5.3:** Different unloading modulus variation with depth at *Ullevi*

The modulus proposed by the Swedish Transport Administration, in *TR Geo*, (Karlsson and Moritz, 2016b) has been assumed to be the most appropriate estimate, since it is based on the study by Persson (2004). This calculation method for determining the unloading modulus is therefore used as basis for the following heave calculations.

#### 5.1.4 Heave

In order to give an estimation of the potential heave of the soil an semi-empirical solution is used. To estimate the unloading modulus the Swedish Transport Administration (Karlsson and Moritz, 2016b) propose to use

$$M_{ul} = 10\sigma'_c e^{\frac{5\sigma'_v}{\sigma'_c}} \quad (5.10)$$

The unloading modulus can also be written as the derivative of the effective vertical stress in respect to the vertical strain

$$M_{ul} = \frac{d\sigma'_v}{d\varepsilon_v} \quad (5.11)$$

which gives

$$\frac{d\sigma'_v}{d\varepsilon_v} = 10\sigma'_c e^{\frac{5\sigma'_v}{\sigma'_c}} \quad (5.12)$$

Thus the heave can be calculated with the integral

$$\int d\varepsilon_v = \int \frac{1}{10\sigma'_c} e^{-\frac{5\sigma'_v}{\sigma'_c}} d\sigma'_v \quad (5.13)$$

which gives

$$\varepsilon_v = \frac{1}{50} e^{-\frac{5\sigma'_v}{\sigma'_c}} + C_0 \quad (5.14)$$

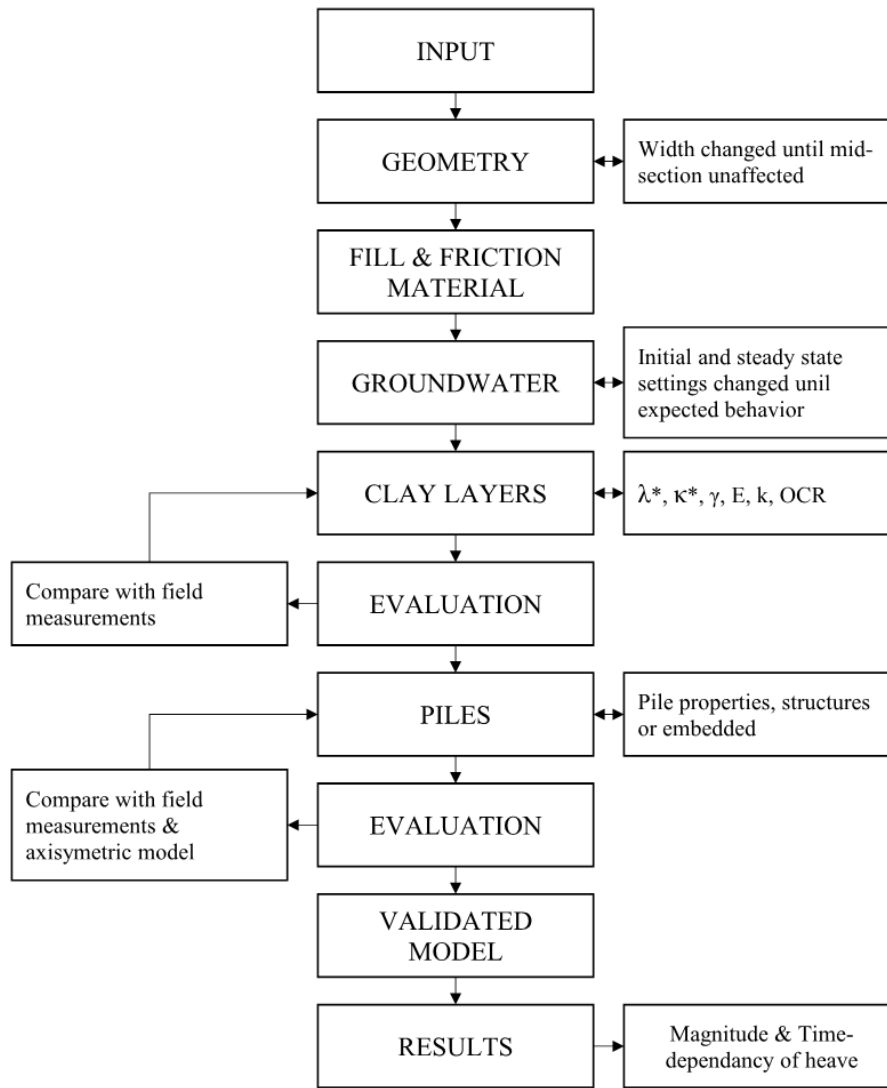
where the integration constant  $C_0$  is relating the function to a reference stress state, here chosen as the in-situ stress state. The total heave of the soil is given by the discretion of the soil into elements with a given height and stress state, which gives the following equation to estimate the total heave

$$h = \sum_{k=1}^n \delta^k \left( \frac{1}{50} e^{-\frac{5\sigma'_v}{\sigma'_c}} + C_0^k \right) \quad (5.15)$$

where  $\sigma'_v$  is the constant stress in the element,  $\delta$  is the thickness of the element,  $C_0$  is the integration constant for the element and  $n$  is the number of elements.

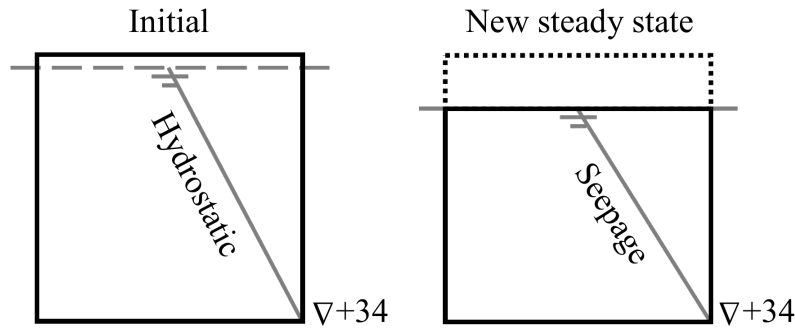
## 5.2 Numerical Model Configuration

The numerical models have been simplified to resemble a one dimensional heave situation, which was possible due to the very large area of the excavation at *Ullevi*. The numerical models have been developed one step at the time to be able to separate different phenomena and to be able to evaluate the influence of each parameter. Figure 5.4 shows the modelling procedure used.



**Figure 5.4:** Flow chart of modelling process in this study

The geometry of the analyzed section was chosen as the total width of the Ullevi excavation and the depth at measurement location *C2* shown in Figure 4.2, which is also from where all the results are obtained. The width of the model was thereafter decreased and increased in order to ensure that the mid-section remained unaffected which was the case for the final width of 60 m. Fill was added above the excavated surface which decreased the surface heave as expected. Below the clay layers, friction materials were added accordingly, acting as a source of groundwater. The original groundwater head located 1 m below the initial ground level was modelled with a hydrostatic profile, whereas the new steady state groundwater head located in the bottom of the excavation had a constant head corresponding to its initial value at the bottom of the clay layers. To ensure that the new steady state groundwater was modelled accurately, the groundwater flow conditions were changed until the expected behavior was obtained, see Figure 5.5 for clarification of groundwater settings.



**Figure 5.5:** Groundwater conditions for the numerical models

During the consolidation analyses, the new steady state groundwater has been given as a condition for the pore pressure calculation type for the phase after the excavation, which means that *PLAXIS 2D* performs a flow calculation to determine the new steady state pore pressures. The succeeding phases are using the pore pressures from previous phase as input ensuring that the analyses are striving against obtaining the new steady state conditions.

The clay layer of 35 meters was divided into sublayers since the properties differ for each meter. The division has been done based on the input data and the number of layers have been adjusted to ensure a correct estimation. The results have been analyzed and compared to the field measurements presented in Section 4.3, some adjustments were made until satisfactory values of the calibration were obtained.

When all settings this far were considered correctly modelled, the next step was to add piles. At *Ullevi*, bored piles were used which has low disturbance to the surrounding soil, and due to the large area of the excavation, embedded piles were chosen as the most appropriate way of modelling. The properties of the embedded piles are shown in Table 5.1.

**Table 5.1:** Assumed properties of embedded piles for *Ullevi*

| Parameter                  | Name                 | Value                 | Unit              |
|----------------------------|----------------------|-----------------------|-------------------|
| Stiffness                  | $E$                  | $100 \cdot 10^6$      | kN/m <sup>2</sup> |
| Unit weight                | $\gamma$             | 24                    | kN/m <sup>3</sup> |
| Pile type                  | –                    | Predefined            | –                 |
| Predefined pile type       | –                    | Massive circular pile | –                 |
| Diameter                   | $D$                  | 1.5                   | m                 |
| Pile spacing               | $L_{spacing}$        | 15                    | m                 |
| Skin resistance            | $T_{skin,start,max}$ | 1                     | kN/m              |
| Skin resistance            | $T_{skin,end,max}$   | 100                   | kN/m              |
| Base resistance            | $F_{max}$            | 100                   | kN                |
| Interface stiffness factor | –                    | Default               | –                 |

An axi-symmetric model was composed for validating the behavior of the embedded pile rows. The same results as for the plain strain model were obtained as expected, which validated the model procedure. The entire procedure as explained

above and visualized in Figure 5.4 resulted in several separate models whereof only a few are presented in the calibration results in Section 5.3.

### 5.2.1 General Settings

The analysis type for all performed analyses are undrained (A), which is an undrained effective stress analysis with effective strength parameters. The settings presented in Table 5.2 are the same for all numerical analyses. The element type was chosen as 15-noded since they give more accurate results than the 6-noded elements. The mesh was set to medium since it was considered fine enough due to the simplicity of the geometry, and then refined around the piles.

**Table 5.2:** General model settings

|           |              |
|-----------|--------------|
| Model:    | Plane strain |
| Element:  | 15-noded     |
| Mesh:     | Medium       |
| $x_{min}$ | = -30        |
| $x_{max}$ | = 30         |
| $y_{min}$ | = -45        |
| $y_{max}$ | = 0          |

The calculation phases are presented in Table 5.3. The first five phases represents the construction process. The time for phase one to five have been set to zero since it makes it easier to keep track of the total time. The last five phases are consolidation phases, where the time for each of these phases are the same as the measurements performed by Alén and Jendeby (1996). The loading type has been set to staged construction for all phases except the last one, which instead has been set to minimum excess pore pressure calculation. The results from these consolidation phases have been used to calibrate the model against measurements of the pore pressure and heave change.

**Table 5.3:** Calculation phases

| Nr. | ID                     | Calculation   | Pore pressure | Time     |
|-----|------------------------|---------------|---------------|----------|
| 1   | Initial phase          | K0 procedure  | Phreatic      | -        |
| 2   | Piling                 | Plastic       | Phreatic      | 0        |
| 3   | Excavation             | Plastic       | Phreatic      | 0        |
| 4   | Steady GW              | Plastic       | Steady state  | 0        |
| 5   | Fill                   | Plastic       | Use previous  | 0        |
| 6   | Consolidation 94.02.08 | Consolidation | Use previous  | 100 days |
| 7   | Consolidation 94.05.31 | Consolidation | Use previous  | 110 days |
| 8   | Consolidation 95.06.27 | Consolidation | Use previous  | 390 days |
| 9   | Consolidation 95.12.14 | Consolidation | Use previous  | 160 days |
| 10  | Final consolidation    | Consolidation | -             | -        |

### 5.2.2 Material Models

The available material models in *PLAXIS 2D* have been presented in Section 3 and their applicability for deep excavations in clay are presented in Table 3.1. The Mohr-Coulomb material model shows low applicability but have been chosen for a first approximation of the problem. The reason for starting with this material model is that the input only consist of a few basic parameters which makes it easier to determine the influence of these and determine the effect of different phenomena, also resulting in the lowest possible computational time.

The Hardening Soil material model shows medium high applicability according to Table 3.1 which, in combination with the higher number of needed input parameters, have resulted in the choice of not using this model. The HS-Small material model shows the highest applicability but has the downside that the same parameters as for the Hardening Soil material model, as well as a few additional parameters, are needed. As a result of the initial lack of available geotechnical investigations and the time it would take to evaluate these parameters the choice was made not to use this material model.

The Soft Soil material model accounts for stress dependent stiffness and shows high applicability for the investigated problem. The needed parameters, in addition to the normal basic parameters, are  $\kappa^*$  and  $\lambda^*$  which need to be either evaluated from oedometer tests or recalculated from stiffness formulas. This material model has been chosen as the most appropriate one, considering the reasons above, for modelling the effect of heave.

### 5.2.3 Parameter Evaluation

The needed input data for the Mohr-Coulomb and the Soft Soil material model are presented in Table 3.2 and Table 3.4 respectively. The common parameters that are set the same for all layers are defined in Table 5.4. The clay parameters  $k$ ,  $\gamma$  and OCR have been evaluated from Figure 4.4 and 4.5. From the figures the mean value have been chosen for each layer for that depth interval.

**Table 5.4:** Basic soil properties set the same for all clay layers

|         |           |                                       |
|---------|-----------|---------------------------------------|
| $c'$    | = 0.1 kPa | effective cohesion                    |
| $\phi'$ | = 30°     | effective friction angel              |
| $\psi'$ | = 0°      | effective dilatancy angel             |
| $K_0$   | = 0.65    | coefficient of lateral earth pressure |

The properties of the fill and friction material were not known from geotechnical investigations, they have therefore been assumed from an example by Brinkgreve et al. (2016). The material model for the friction layer has been set to Hardening Soil since it better represents the material. Since they affect the result to a very low extent, the assumed properties have been considered to be within reason. Table 5.5 and Table 5.6 summarizes the assumed parameters.

**Table 5.5:** Properties of the fill

|                               |              |
|-------------------------------|--------------|
| Material model                | Mohr-Coulomb |
| Drainage type                 | Drained      |
| $\gamma$ [kN/m <sup>3</sup> ] | 18           |
| $E'$ [kN/m <sup>2</sup> ]     | 20 000       |
| $\nu'$ [-]                    | 0.33         |
| $\phi$ [°]                    | 30           |
| $k_{x,y}$ [m/day]             | 10           |

**Table 5.6:** Properties of the friction material

|                                       |                |
|---------------------------------------|----------------|
| Material model                        | Hardening Soil |
| Drainage type                         | Drained        |
| $\gamma_{unsat}$ [kN/m <sup>3</sup> ] | 17             |
| $\gamma_{sat}$ [kN/m <sup>3</sup> ]   | 20             |
| $E_{50}^{ref}$ [kN/m <sup>2</sup> ]   | 40 000         |
| $E_{oed}^{ref}$ [kN/m <sup>2</sup> ]  | 40 000         |
| $E_{ur}^{ref}$ [kN/m <sup>2</sup> ]   | 120 000        |
| power (m)[-]                          | 0.5            |
| $\phi$ [°]                            | 32             |
| $\psi$ [°]                            | 2              |
| $k_{x,y}$ [m/day]                     | 1              |

## Mohr-Coulomb Material Model for Clay

The unloading modulus has been calculated with depth according to Equation 5.10. Since the Mohr-Coulomb material model does not account for a change of stiffness with change of stress, the stiffness has to be calculated for the occurring stress state for each depth interval. The vertical effective stress was therefore studied for two different cases: the initial stress state, and as a mean value between the initial and the steady stress state. The function  $E_{inc}$  has been used in *PLAXIS 2D* to account for the increase of stiffness with depth according to Equation 3.1.

The Mohr-Coulomb model has seven clay layers, each with a depth of five meters. The stiffness for all layers are presented in Table 5.7 calculated for the initial stress state and in Table 5.8 calculated for the mean stress state.

**Table 5.7:** Stiffness for initial stress state

| Depth | $E'$ [kN/m <sup>2</sup> ] | $E_{inc}$ [kN/m <sup>2</sup> ] | $y_{ref}$ [m] |
|-------|---------------------------|--------------------------------|---------------|
| 0-5   | 8 500                     | 2 740                          | -1            |
| 5-10  | 19 500                    | 1 740                          | -5            |
| 10-15 | 29 900                    | 2 610                          | -10           |
| 15-20 | 45 500                    | 2 620                          | -15           |
| 20-25 | 61 300                    | 2 080                          | -20           |
| 25-30 | 73 700                    | 2 910                          | -25           |
| 30-35 | 91 200                    | 2 910                          | -30           |

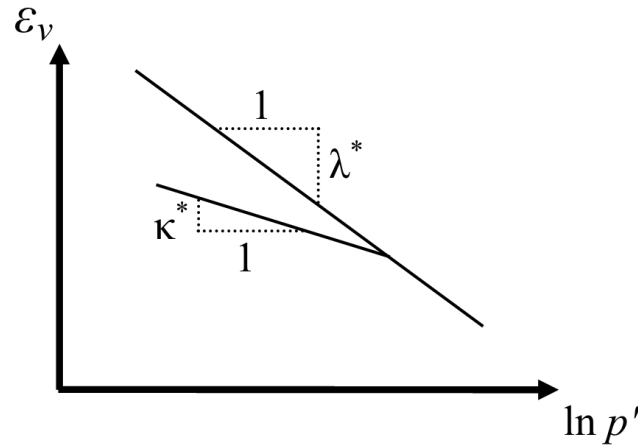
**Table 5.8:** Stiffness for mean stress state

| Depth | $E'$ [kN/m <sup>2</sup> ] | $E_{inc}$ [kN/m <sup>2</sup> ] | $y_{ref}$ [m] |
|-------|---------------------------|--------------------------------|---------------|
| 0-5   | 1 300                     | 1 110                          | -1            |
| 5-10  | 5 730                     | 1 380                          | -5            |
| 10-15 | 12 600                    | 2 330                          | -10           |
| 15-20 | 24 300                    | 2 300                          | -15           |
| 20-25 | 35 800                    | 2 000                          | -20           |
| 25-30 | 45 600                    | 2 800                          | -25           |
| 30-35 | 59 800                    | 2 910                          | -30           |

### Soft Soil Material Model for Clay

The input parameters that are needed specifically for the Soft Soil material model is the modified swelling index,  $\kappa^*$ , and the modified compression index,  $\lambda^*$ . Of most importance for unloading problems is the swelling index. Focus have therefore been put into evaluating  $\kappa^*$  properly.

According to Brinkgreve et al. (2016)  $\kappa^*$  and  $\lambda^*$  can be estimated as the inclination of the slope in an  $\ln p' - \varepsilon$  graph as shown in Figure 5.6.

**Figure 5.6:** Evaluation of  $\kappa^*$  and  $\lambda^*$ 

where  $p'$  is the mean effective stress, calculated as

$$p' = \frac{\sigma'_1 + \sigma'_2 + \sigma'_3}{3} = \sigma'_v \frac{1 + \frac{2\nu}{1-\nu}}{3} \quad (5.16)$$

One evaluation method for  $\kappa^*$  has been based on available oedometer tests. The oedometer tests have been recalculated as explained in Figure 5.6, and the results are displayed in Table 5.9.

**Table 5.9:**  $\kappa^*$  calculated from oedometer tests according to Figure 5.6

| Depth [m] | $\kappa^*$          | $\lambda^*$          |
|-----------|---------------------|----------------------|
| 10        | $3.0 \cdot 10^{-3}$ | $13 \cdot 10^{-3}$   |
| 16        | $3.5 \cdot 10^{-3}$ | $11.6 \cdot 10^{-3}$ |
| 24        | $4.3 \cdot 10^{-3}$ | $12.3 \cdot 10^{-3}$ |
| 32        | $6.7 \cdot 10^{-3}$ | $17.3 \cdot 10^{-3}$ |

Another way of evaluating  $\kappa^*$  is presented by Olsson (2010) where the relation between oedometer modulus and the swelling index,  $\kappa^*$ , is written as

$$\kappa^* = \frac{2\sigma'_v}{M} \quad (5.17)$$

where  $M$  is the oedometer modulus. The second evaluation method for  $\kappa^*$  have therefore been based on Equation 5.17 with the unloading modulus obtained from the equation by *TR Geo*, for the initial stress state. The results are presented in Table 5.10. The value of  $\lambda^*$  have not been calculated, instead they have been assumed to be five times higher than  $\kappa^*$ .

**Table 5.10:**  $M$ ,  $\sigma'_0$  and  $\kappa^*$  calculated according to Equation 5.17 with the unloading modulus equation from *TR Geo*

| Depth [m] | $M$ [kPa] | $\sigma'_0$ [kPa] | $\kappa^*$           |
|-----------|-----------|-------------------|----------------------|
| 0-5       | 14 000    | 22.5              | $3.21 \cdot 10^{-3}$ |
| 5-10      | 24 700    | 47.5              | $3.85 \cdot 10^{-3}$ |
| 10-15     | 37 700    | 77.5              | $4.11 \cdot 10^{-3}$ |
| 15-20     | 53 400    | 113               | $4.21 \cdot 10^{-3}$ |
| 20-25     | 67 500    | 145               | $4.3 \cdot 10^{-3}$  |
| 25-30     | 82 500    | 180               | $4.37 \cdot 10^{-3}$ |
| 30-35     | 100 000   | 220               | $4.4 \cdot 10^{-3}$  |

There has been some difficulty in evaluating  $\kappa^*$ , since different approaches gives different results. Evaluation from CRS tests give values of  $\kappa^*$  up to ten times higher than evaluation from other oedometer tests (by  $\ln p' - \varepsilon$  graph) and *TR Geo* calculations, see Appendix B. Although, the obtained  $\kappa^*$  from the oedometer tests from *Ullevi* and *TR Geo* gives similar results.

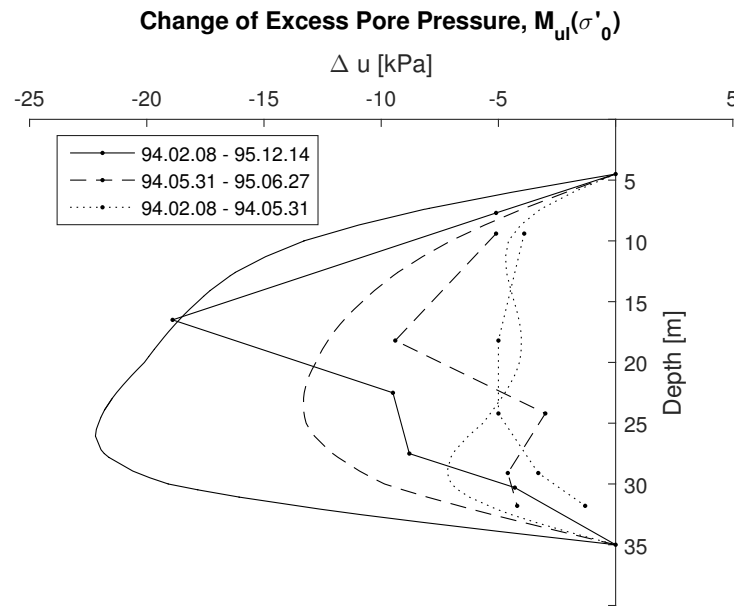
### 5.3 Calibration Results

To ensure reliable results the model has been calibrated. To calibrate the model it has been made to simulate the construction site for the garage at *Ullevi*, in regards to the geometry and the geological and hydrological conditions. To validate the model the pore pressure and heave changes have been verified by comparison of the measurement presented in Figure 4.6 and 4.7. Note that in all of the following figures displaying the pore pressure change, the coarse lines indicated with dots are the field measurements and the lines without dots are the results from the numerical models.

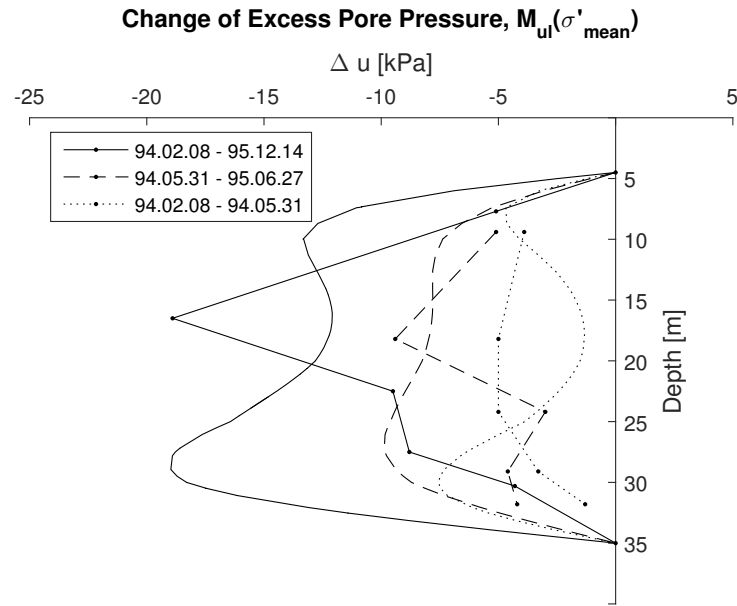
The field measurements and the numerical results are displayed for the same time periods.

## Mohr-Coulomb Model

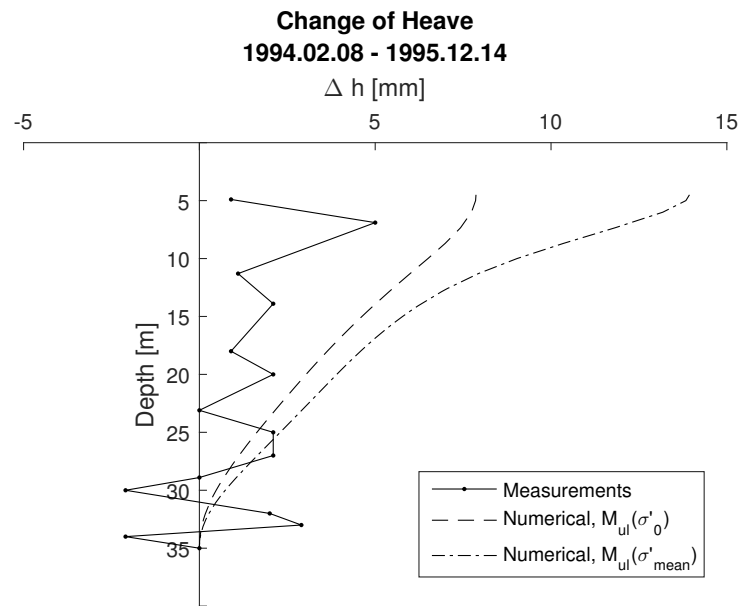
The results from the Mohr-Coulomb material model have been compared to the measured data from *Ullevi*, see Figure 4.6 and 4.7. Two different unloading moduli setups have been used to calculate the change in pore pressure; one for the initial stress state,  $\sigma'_0$ , see Figure 5.7 and one for the mean stress state,  $\sigma'_{mean}$ , see Figure 5.8. The two moduli setups have also been used to calculate the heave, see Figure 5.9.



**Figure 5.7:** Comparison of the measured and calculated pore pressure change,  $M_{ul}$  calculated for initial stress state



**Figure 5.8:** Comparison of the measured and calculated pore pressure change,  $M_{ul}$  calculated for the mean stress state

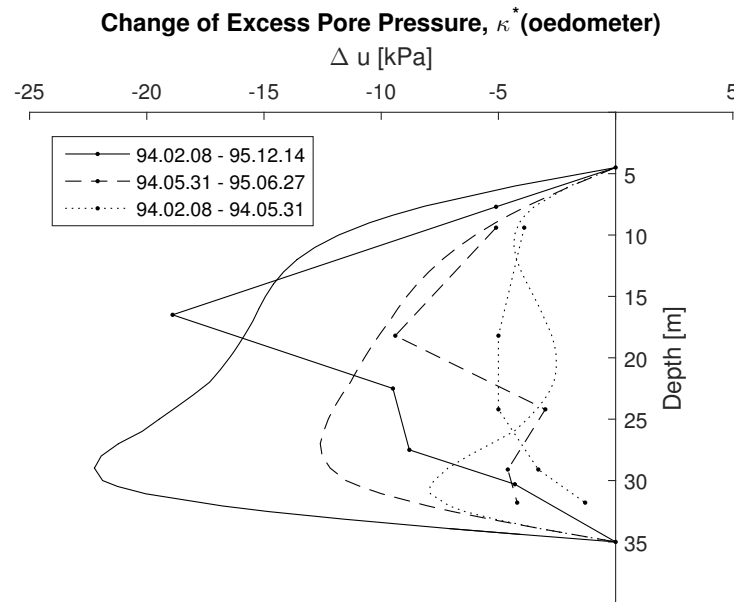


**Figure 5.9:** Comparison of the measured and calculated change of heave

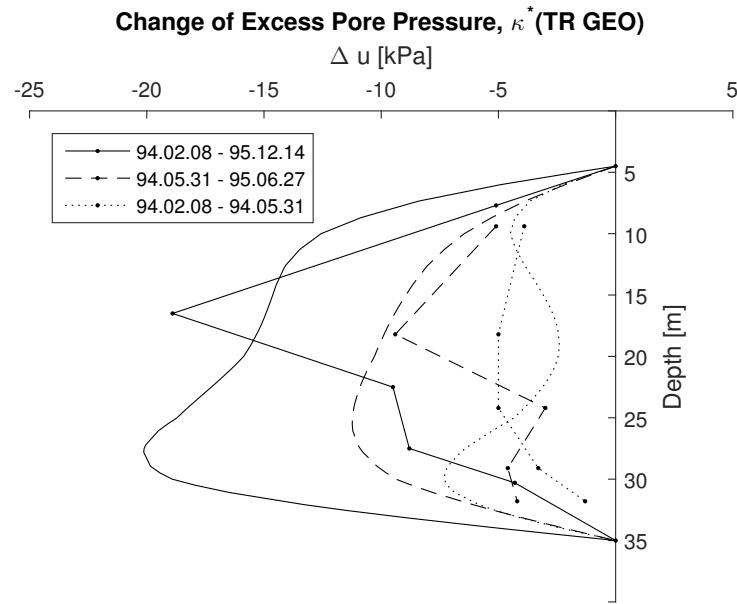
The results from the change in pore pressure and heave show that the calculations using the initial stress match better than for the mean stress state. However, the match was assessed to be insufficient for validating the parameters, and hence a new model with the use of the Soft Soil model was created.

## Soft Soil Model

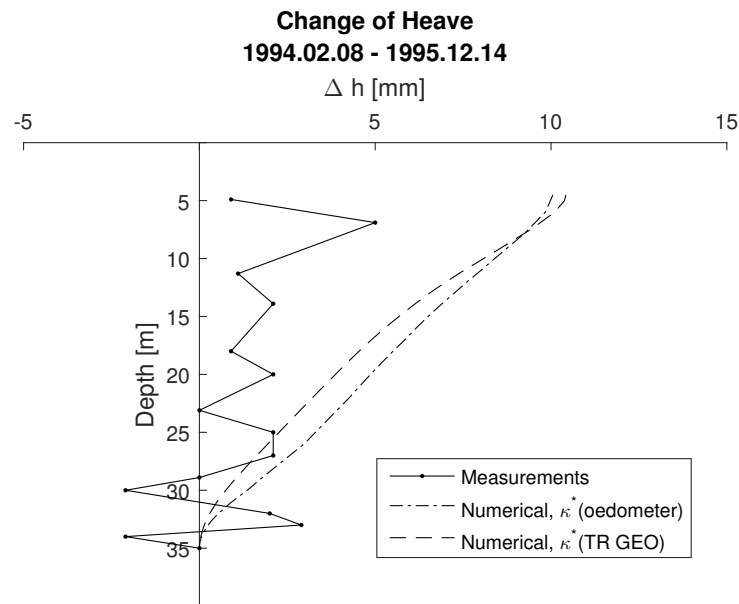
The pore pressure and heave change results from the Soft Soil model have been calculated with two different parameter setups; one where  $\kappa^*$  and  $\lambda^*$  are derived from the *TR Geo* unloading modulus and one where they are derived from oedometer tests from *Ullevi*. The results in the pore pressure change are shown in Figure 5.10 and 5.11 respectively. The change in heave is plotted for both parameter setups in Figure 5.12.



**Figure 5.10:** Comparison of the measured and calculated pore pressure change,  $\kappa^*$  calculated from oedometer tests



**Figure 5.11:** Comparison of the measured and calculated pore pressure change,  $\kappa^*$  calculated from *TR Geo*



**Figure 5.12:** Comparison of the measured and calculated change of heave

The results show that there is a small difference between the two parameter setups in the pore pressure change, the results with  $\kappa^*$  from oedometer tests has slightly higher changes. Moreover, the change in heave show that the *TR Geo* setup results are slightly closer to the measurements than the oedometer setup, see

Figure 5.12. Thus the best fit with measured data is when  $\kappa^*$  is as little as possible. The evaluation method, with *TR Geo*, giving the smallest  $\kappa^*$  have, therefore, been chosen.

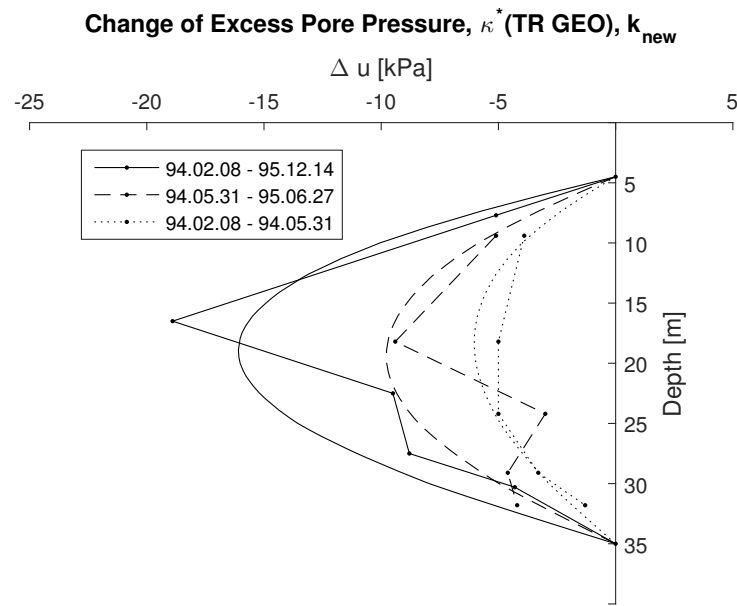
## Adjustment of Parameters

The permeability of each layer have been adjusted for better resembling the field measurements of heave and pore pressure changes. The original permeabilities from the geotechnical investigations presented in Figure 4.4 are obtained from compression tests and not from unloading tests which probably would have given higher values. With this in mind, it was considered reasonable to increase the values of permeability which also gives results more alike the field measurements. The permeability was therefore increased five times for the first three layers, and thereafter set to decrease with  $4 \cdot 10^{-6}$  meter per day per meter from 15 to 35 meters depth, giving the best correspondence to the measurement data. The new, assumed, permeability are presented in Table 5.11 for each layer.

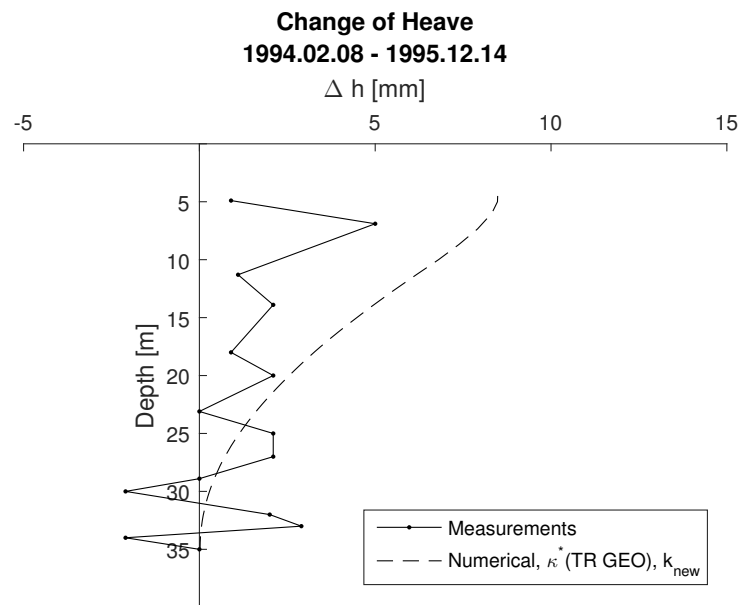
**Table 5.11:** Adjusted permeability

| Depth [m] | $k_{original}$ [m/day] | $k_{new}$ [m/day]  |
|-----------|------------------------|--------------------|
| 0-5       | $4.84 \cdot 10^{-5}$   | $24 \cdot 10^{-5}$ |
| 5-10      | $4.84 \cdot 10^{-5}$   | $24 \cdot 10^{-5}$ |
| 10-15     | $4.84 \cdot 10^{-5}$   | $24 \cdot 10^{-5}$ |
| 15-20     | $4.33 \cdot 10^{-5}$   | $22 \cdot 10^{-5}$ |
| 20-25     | $3.15 \cdot 10^{-5}$   | $20 \cdot 10^{-5}$ |
| 25-30     | $1.99 \cdot 10^{-5}$   | $18 \cdot 10^{-5}$ |
| 30-35     | $1.37 \cdot 10^{-5}$   | $16 \cdot 10^{-5}$ |

The results for the new adjusted permeability, see Figure 5.13, shows a better match for all the curves. Given the reasoning in the behavior of the permeability the adjusted values can be used in the further analysis. The change seen in the heave is not of the same magnitude but it presents a better match with the measured data, see Figure 5.14.



**Figure 5.13:** Comparison of the measured and calculated pore pressure change, with adjusted permeability



**Figure 5.14:** Comparison of the measured and calculated change of heave, with adjusted permeability

### 5.3.1 Final Parameter Setup

The calibration has resulted in a final model and parameter setup that will be used to model heave pressures. That model is the Soft Soil model with the adjusted permeability as it show the best match to the calibration data. The geometry of the model is as shown in Figure 4.3 and Table 5.2 and the properties of the embedded piles, basic clay properties, fill and the friction material are given in Table 5.1, 5.4, 5.5 and 5.6 respectively. All advanced parameters are set to their default values. The remaining parameters are summarized in Table 5.12.

**Table 5.12:** Clay properties for the final model setup

| Depth<br>[m] | $\gamma$<br>[kN/m <sup>3</sup> ] | $\lambda^*$<br>[-]    | $\kappa^*$<br>[-]    | $k$<br>[m/day]     | OCR<br>[-] |
|--------------|----------------------------------|-----------------------|----------------------|--------------------|------------|
| 0-5          | 15                               | $16.05 \cdot 10^{-3}$ | $3.21 \cdot 10^{-3}$ | $24 \cdot 10^{-5}$ | 1.35       |
| 5-10         | 15                               | $19.25 \cdot 10^{-3}$ | $3.85 \cdot 10^{-3}$ | $24 \cdot 10^{-5}$ | 1.39       |
| 10-15        | 16.7                             | $20.55 \cdot 10^{-3}$ | $4.11 \cdot 10^{-3}$ | $24 \cdot 10^{-5}$ | 1.42       |
| 15-20        | 17                               | $21.05 \cdot 10^{-3}$ | $4.21 \cdot 10^{-3}$ | $22 \cdot 10^{-5}$ | 1.43       |
| 20-25        | 16.2                             | $21.50 \cdot 10^{-3}$ | $4.30 \cdot 10^{-3}$ | $20 \cdot 10^{-5}$ | 1.44       |
| 25-30        | 17.7                             | $21.85 \cdot 10^{-3}$ | $4.37 \cdot 10^{-3}$ | $18 \cdot 10^{-5}$ | 1.45       |
| 30-35        | 18                               | $22.00 \cdot 10^{-3}$ | $4.40 \cdot 10^{-3}$ | $16 \cdot 10^{-5}$ | 1.45       |

## 5.4 Heave & Heave Pressure

For investigating the time-dependency of heave and the magnitude of heave pressure, the numerical model calibrated against *Ullevi* has been used and adjusted. From this point on, the results are predictive and can not be compared to any field measurements. The construction procedure at the garage beneath *Ullevi* has no longer been simulated since heave pressure were not relevant at that site, due to the free connection between the foundation slab and the piles. The calibration have therefore been used as a way of validating the material properties and the simplified geometry.

### 5.4.1 Time-Dependency of Heave

For obtaining the time-dependency of heave, the analysis was run until all excess pore pressure had dissipated. The piles were set to have zero cohesion, hence not influencing the heave, and thus the maximum heave was obtained. The results were checked in the middle of the model at the bottom of the excavation. Four different excavation depths were checked, 2, 4, 6 and 8 m.

### 5.4.2 Heave Pressure

To investigate the potential pressure that the restrained heave can cause on a concrete slab, the structural elements of the calibrated model was adjusted. The pile properties were changed and the spacing used was 8 and 4 m in both directions.

A continuous concrete slab was added on top of the piles, and the top connection point was set as rigid, creating a scenario where the heave is locked and hence the soil's reaction to the unloading will create a contact pressure instead of a heave. To retrieve a direct effect of the heave onto the slab, the fill was removed, connecting the slab directly to the clay. The width of the model was changed in order to keep a symmetric cross section even though the pile spacing was changed.

The heave pressure could then be obtained as a uniform load by equilibrium of the vertical forces as stated in Equation 5.18.

$$q_{heave} = \frac{\sum Q_{pile} + q_{slab} \cdot l}{l} \quad (5.18)$$

where  $Q_{pile}$  is the axial load at the top of the pile,  $q_{slab}$  is the load of the concrete slab acting on the piles and  $l$  is the length of the slab. The slab has also been modelled as weightless for verification of the above stated equation which gives

$$q_{heave} = \frac{\sum Q_{pile}}{l} \quad (5.19)$$

### Maximum Obtainable Pressure

To be able to investigate the theoretical maximum pressures, the system was locked down by making the slab and the piles infinitely stiff. The piles were modelled with no cohesion, so that the soil was free to slide along the pile shafts. Furthermore, the piles were assigned a zero displacement in the bottom, creating a scenario where there is no deformation of the structures and hence maximum pressures against the slab.

The effect of different excavation depths and the consolidation time before casting of the slab has been examined. The excavation depth was chosen to be in the range of 2-8 m and the time before casting was set to 0, 30, 90 and 180 days. The effect of different parameters were thereafter investigated by increasing the clay depth, adding cohesion to the piles and changing the cc-distance. When increasing the clay depth, the clay properties have been assumed to be constant for the last clay layer to obtain an approximation of what would happen if the total clay depth were to be 70 m instead of 35 m.

### Realistic Simulation of Housing Project

In order to investigate the pressures that might be created during construction, the model was adjusted to simulate a typical housing project. The properties of the piles were calculated by assuming square piles in groups of three with a width of 0.27 m with 8 $\phi$ 12 reinforcement and with a free bottom connection. To simulate the use of pile groups, each pile was assumed to represent three piles and the properties were therefore recalculated for this case. The reinforcement were assumed to take all of the load, since the piles would be in tension, and the stiffness was therefore calculated only for the reinforcement and uniformly distributed on the pile cross section. The skin resistance of the piles was calculated as the shear strength of the surrounding clay multiplied with the circumference, where the shear strength

of the clay is linear from 15 kPa in the top to 60.5 kPa in the bottom. The slab was assumed to be 0.5 m thick, the concrete C45/55 and  $\phi 16s150$  reinforcement. The model properties of the piles and slab can be seen in Table 5.13 and Table 5.14 respectively.

**Table 5.13:** Assumed properties of embedded piles for a typical housing project

| Parameter                  | Name                 | Value               | Unit              |
|----------------------------|----------------------|---------------------|-------------------|
| Stiffness                  | $E$                  | $2.48 \cdot 10^6$   | kN/m <sup>2</sup> |
| Unit weight                | $\gamma$             | 24                  | kN/m <sup>3</sup> |
| Pile type                  | –                    | Predefined          | –                 |
| Predefined pile type       | –                    | Massive square pile | –                 |
| Width                      | –                    | 0.468               | m                 |
| Pile spacing               | $L_{spacing}$        | 8 / 4               | m                 |
| Skin resistance            | $T_{skin,start,max}$ | 45                  | kN/m              |
| Skin resistance            | $T_{skin,end,max}$   | 182                 | kN/m              |
| Base resistance            | $F_{max}$            | 1000                | kN                |
| Interface stiffness factor | –                    | Default             | –                 |

**Table 5.14:** Assumed properties of concrete slab calculated for a typical housing project

| Parameter       | Name  | Value             | Unit              |
|-----------------|-------|-------------------|-------------------|
| Material type   | –     | Elastic           | –                 |
| Isotropic       | –     | Yes               | –                 |
| Stiffness       | $EA$  | $18.1 \cdot 10^6$ | kN/m <sup>2</sup> |
| Stiffness       | $EI$  | $375 \cdot 10^3$  | kN/m <sup>2</sup> |
| Weight          | $w$   | 12.5              | kN/m/m            |
| Poisson's ratio | $\nu$ | 0.2               | –                 |



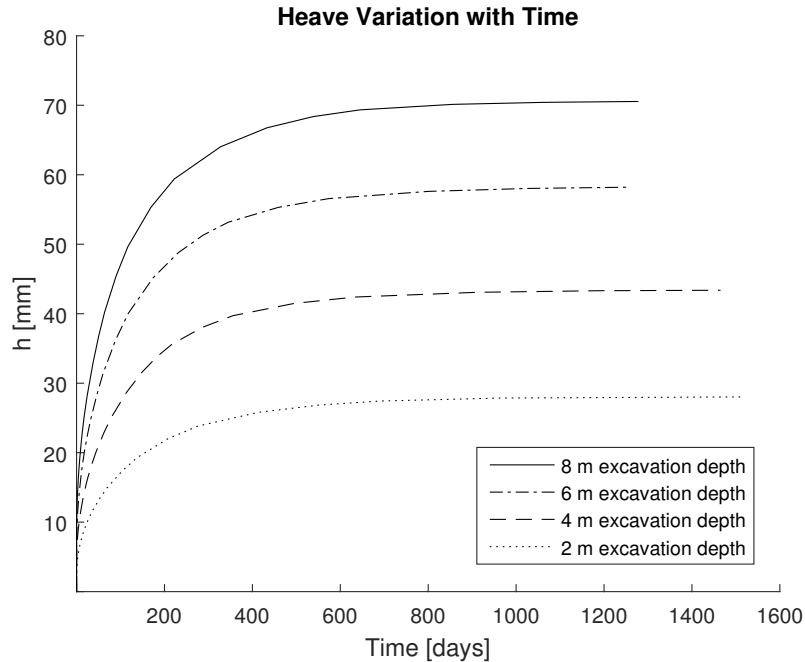
# 6

## Results

This chapter presents the results connected to the time-dependency of heave and the heave pressures. It should be noted that the following results are only validated for the geometry, parameter setup and input data used. The pressures are influenced by the lowering of the groundwater, as well as a number of other factors. The results should therefore be interpreted with care and are not to be used for settings that are different from the scope of this thesis. The results presented are the basis of the evaluation and discussion of the heave pressures within the limitations of this thesis.

### 6.1 Time-Dependency of Heave

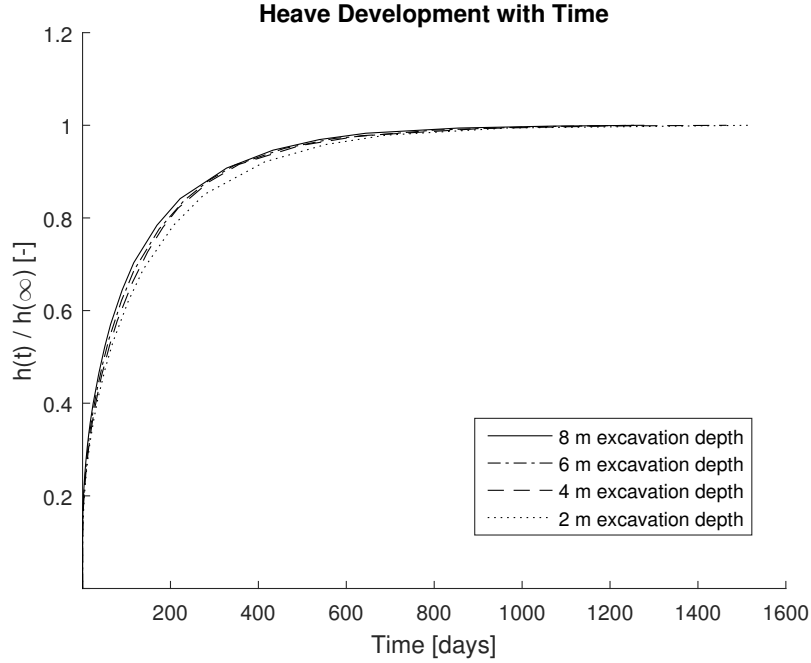
Firstly the heave development has been investigated without the effect of piles. In Figure 6.1 the predicted heave with time is plotted for different excavation depths. From the figure it can be seen that the magnitude of the heave increases with the excavation depth while the shape remain rather similar.



**Figure 6.1:** Heave variation with time for different excavation depths

From the heave variation with time, see Figure 6.1, the shape of the curves can be further investigated. Hence, if the heave is plotted as a ratio of the total

heave, i.e. the heave development  $h(t)/h(\infty)$ , it shows that the heave development is approximately the same regardless of excavation depth, see Figure 6.2. However, there is a small increase of the heave development with excavation depth likely due to the small decrease of the total clay depth.



**Figure 6.2:** Developed heave with time

The heave is also dependent on the thickness of the underlying clay layer, i.e. the clay depth  $z_d$ . Table 6.1 shows how long it takes for the heave to develop for a clay depth of 35 m compared to a clay depth of 70 m.

**Table 6.1:** Time until certain developed heave,  $h(t)/h(\infty)$ , for 4 m excavation depth presented for  $z_d=35$  and 70 m

| $h(t)/h(\infty) \approx$ | $t(35 \text{ m})$ [days] | $t(70 \text{ m})$ [days] |
|--------------------------|--------------------------|--------------------------|
| 20%                      | 4                        | 9                        |
| 40%                      | 32                       | 66                       |
| 60%                      | 87                       | 190                      |
| 80%                      | 200                      | 440                      |
| 100%                     | 1 465                    | 2 362                    |

The results from the numerical analyses have been compared with semi-empirical calculations performed according to Equation 5.15 and are presented in Table 6.2. The semi-empirical calculations performed in this study only give a value of the total heave, and have hence not been used to investigate the time-dependency. The comparison show that the calculation according to Equation 5.15 give almost double the total heave obtained from the numerical analyses.

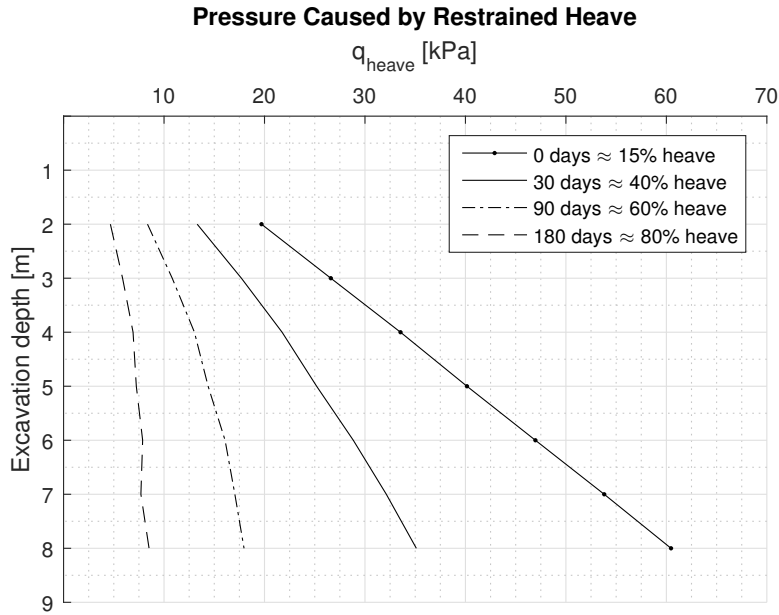
**Table 6.2:** The total heave for different excavation depth with *TR Geo* and *PLAXIS 2D* calculations

| $z_{exc}$ [m] | $h$ , <i>TR Geo</i> [mm] | $h$ , <i>PLAXIS 2D</i> [mm] |
|---------------|--------------------------|-----------------------------|
| 2             | 42.4                     | 28.0                        |
| 4             | 73.6                     | 43.4                        |
| 6             | 107.6                    | 58.2                        |
| 8             | 139.3                    | 70.5                        |

When considering the heave pressures,  $q_{heave}$ , the heave development has been used to determine how much of the soil that has been heaved before the slab is casted. From this, the effect of the time before casting the slab could be considered when analyzing the heave pressure.

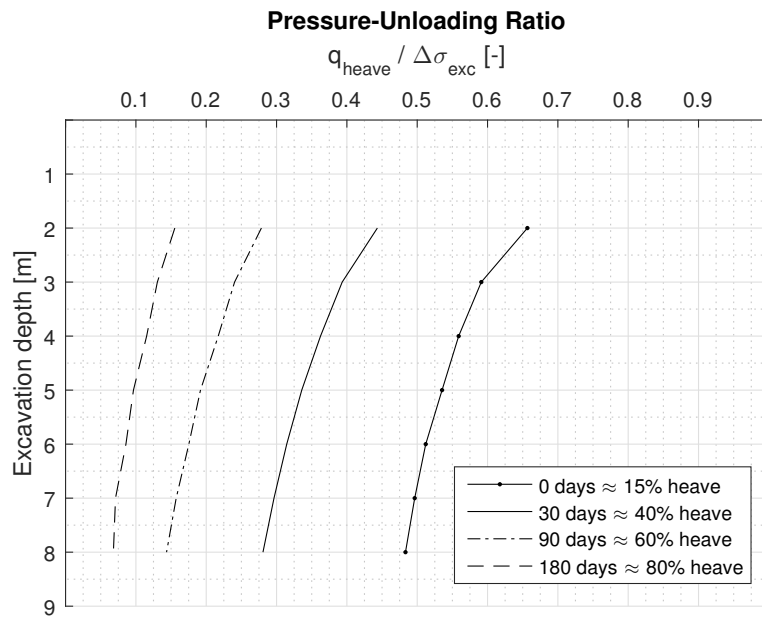
## 6.2 Maximum Obtainable Pressures

For the infinitely stiff case, maximum pressures are obtained. Figure 6.3 presents the magnitude of the pressure caused by restrained heave, i.e. the heave pressures, for different excavation depths and times before casting the slab. The calculation performed with a weightless system gives the same results as when including the weight. Thus, validating the calculation method for slab load consideration.

**Figure 6.3:** Pressure caused by restrained heave for different excavation depths and the time before casting of the slab

In order to set the pressures in relation to the amount of unloading, Figure 6.4 presents the ratio between the pressures and the unloading for different excavation

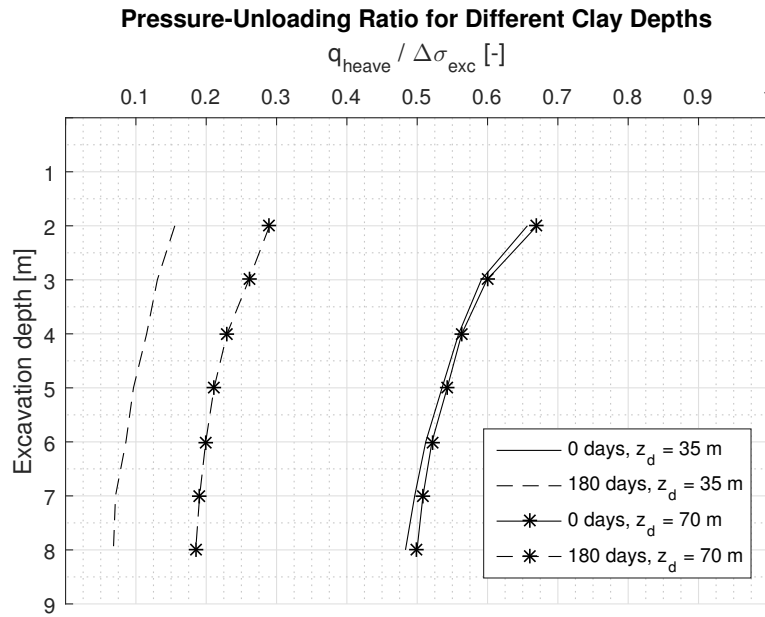
depths and different consolidation times before casting of the concrete slab. It can be seen that the largest ratio is around 65%, obtained for a 2 m excavation where the slab is casted immediately after the excavation is performed. It should be noted that this is not a realistic situation, since the time for performing the excavation and reinforcing the concrete slab is not taken into account. If the excavation is left open longer before the slab is casted, the clay will heave. This means that the pressures will be lower. As seen in the figure, the pressure drop to a maximum ratio of around 45% at 2 m excavation depth if the excavation is open for 30 days, and to below 15% if the excavation is open for 180 days before the slab is casted.



**Figure 6.4:** The pressure-unloading ratio for the maximum obtainable pressures

### 6.2.1 Increased Clay Depth

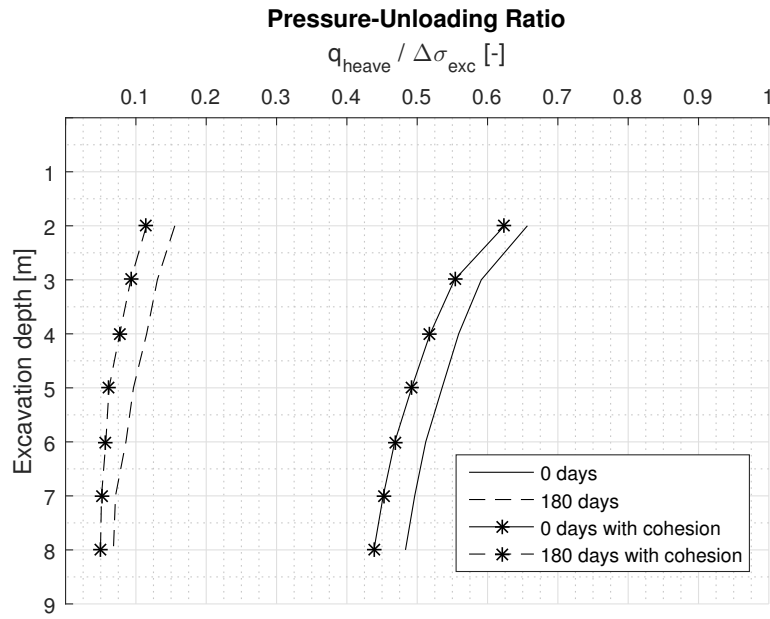
The clay depth has been increased to be twice as large. The results can be seen in Figure 6.5 and shows that there is basically no difference if the slab is casted immediately after the excavation. If the clay is allowed to consolidate for 180 days before the slab is casted, the obtained pressures are higher if the clay depth is increased. This is due to the fact that it takes longer for the clay to consolidate and hence there are higher values of the negative excess pore pressures left when the slab is casted.



**Figure 6.5:** Comparison between different initial total clay depths,  $z_d$ , impact on pressure-unloading ratio for different times before casting of slab

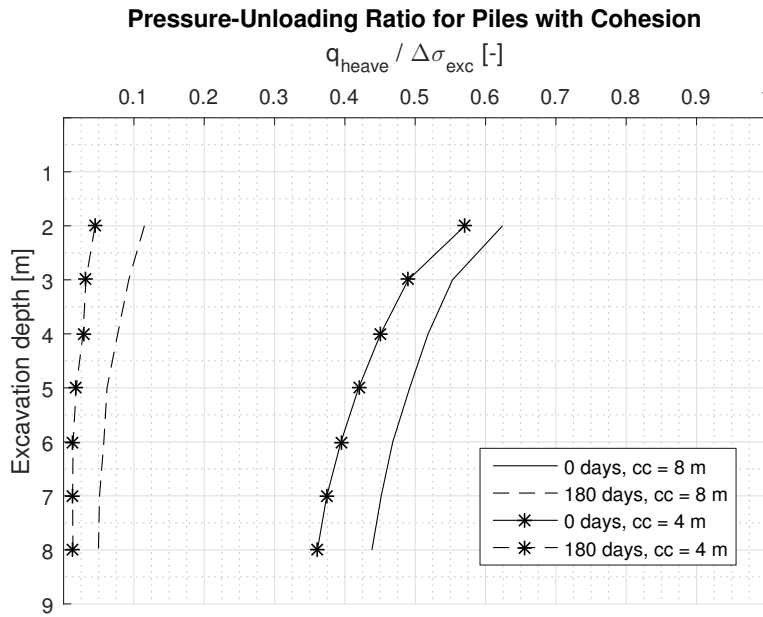
### 6.2.2 Piles with Cohesion

To make the model less theoretical and to resemble a typical housing project better, interaction factors have been added along the pile shafts, making it harder for the soil to press upwards, but the system has been kept infinitely stiff. The difference between the results in Figure 6.6 are therefore only given by the additional cohesion. When comparing the results with and without cohesion, it can be seen that the pressure-unloading ratio decreases when cohesion is added to the piles. The magnitude of the decrease is dependent on the consolidation time before casting the concrete slab and also the excavation depth.



**Figure 6.6:** Pressure-Unloading Ratio for different excavation depths with and without pile shaft cohesion

The piles with cohesion have been added both at a cc-distance of 8 m and a cc-distance of 4 m. The largest reason for the difference between the obtained pressures for piles with a distance of 8 and 4 m, as seen in Figure 6.7, is the additional cohesion. The cohesion along the piles are holding the system down, not allowing it to heave, this is more prominent the more piles there are in the system.



**Figure 6.7:** Pressure-Unloading Ratio for different excavation depths compared for different cc-distances between piles

### 6.3 Realistic Simulation of a Housing Project

An analysis trying to resemble a realistic housing project was performed by reducing the stiffness of the piles and the slab, as well as allowing the piles to move in the bottom connection. The obtained pressures were, however, negative down to 4 m excavation depth, and then positive but at a very low magnitude. When leaving the excavation open for a larger extent of time, all pressures obtained were negative.

One reason for not obtaining any pressures is that the system was not stiff enough to lock in the heave. There was a displacement of the piles occurring before casting of the slab, resulting in small settlements of the piles when the slab was casted. For the cases where negative pressures were obtained, the piles have continued to settle during the consolidation time after the slab has been casted. The slab is not stiff enough either, deforming between the piles which allows the soil to heave and hence no pressures are generated. It was also observed that the weight of the slab was not taken by the piles, instead it goes into the soil.

It should, however, be noted that the stiffness of the piles was set to the tension stiffness, i.e. the stiffness of the reinforcement, and since  $q_{heave}$  is negative for these cases, the piles are in fact in compression, which means that the results may not be accurate. It is, however, safe to say that the pressures caused by restrained heave are severely reduced when the system is made less stiff.

The results of the realistic simulation are not presented since they are not considered to be accurate and there is no way of validating the obtained pressures. In order to investigate the pressures further, it would be possible to check the moment curves of the slab and compare these for different cases. This was, however,

considered to fall outside the scope of this thesis.

## 6.4 Comparison with Current Practice

The maximum obtained pressure-unloading ratios have been compared with different ways of determining it according to current practice. Table 6.3 shows the pressure-unloading ratio as obtained from the numerical calculations, estimated as maximum 40% according to guidelines for the project *Götetunneln*, see Equation 1.1, and as the remaining consolidation potential accounting for the time the excavation has been open, calculated according to Section 1.1.2. It can be seen that the values according to current practice are higher than the ones obtained from the numerical analyses. The value of 40% is far from accurate when the excavation has been open for more than 30 days, the values obtained as the remaining consolidation potential are closer to the numerical analyses when the excavation has been open for more than 30 days, but it still overestimates the pressures.

**Table 6.3:** Comparison between the pressure-unloading ratio for numerical calculations and current practice

| Consolidation time<br>before lock-in | Maximum ratio<br>$q_{heave}/\Delta\sigma_{exc}$ | Current practise<br>Equation 1.1 | Current practise<br>( $1 - U$ ) |
|--------------------------------------|-------------------------------------------------|----------------------------------|---------------------------------|
| 30                                   | 45%                                             | 40%                              | 60%                             |
| 90                                   | 28%                                             | 40%                              | 40%                             |
| 180                                  | 16%                                             | 40%                              | 20%                             |

# 7

## Discussion & Conclusion

The discussion is an important part of the study, to review potential drawbacks and reliability issues. It will hence in this chapter be discussed how the assumptions and simplifications affect the results and what could be done to minimize these effects. As stated by Muir Wood (2004), the results are only a function of how they were obtained, and hence the entire modelling process needs to be considered in order to validate the results. To relate back to Chapter 3, three questions cited by Muir Wood (2004) were presented regarding verification and validation. These questions will be treated by dividing the discussion chapter into three parts: if the chosen method was appropriate, if the obtained results are providing the correct answers and what else can be done to obtain the answers in need. Followed by a conclusion summarizing the findings of this study.

### 7.1 Development of Numerical Model

The phenomenon of heave pressure is relatively unknown, and the existing theory and data are limited, on the verge to non-existent. Due to this, it was decided that the research should start simple with a one-dimensional model. This would provide a base for further studies regarding the phenomenon. The choice of calibrating the numerical model against the Ullevi excavation was made for two reasons: firstly, the excavation covered a large area, minimizing the three-dimensional effects, and lastly because of the previous studies done there which provided measurement data of pore pressures and heave.

The choice of making the model one-dimensional meant that it would be easier to separate the different phenomena and effects. However, it should be noted that it also means that the model basically acts in one dimension even though the phenomenon most likely have three-dimensional components. Hence the obtained results will only be a crude approximation of reality.

The numerical models were developed one step at the time as explained in Figure 5.4, which made it easier to determine the effects of different parameters. It was noted that not all parameters have the same influence on the heave and pore pressures. The parameters with the largest effect, and hence the parameters which most focus have been put into, is the unloading modulus, or  $\kappa^*$  for the Soft Soil material model, and the permeability, together with boundary conditions and groundwater settings. Since the model was calibrated against the excavation at *Ullevi*, all results are only valid for the specific geometry and the clay properties at this location.

### 7.1.1 Evaluation of Parameters

By determining the stress state analytically it has been possible to evaluate the soil parameters in another way than by solely using the available oedometer tests. For the Soft Soil material model the semi-empirical calculations, especially regarding the unloading modulus and  $\kappa^*$ , have been compared to measured parameters from oedometer tests. These two parameter setups show similar results. This gives a validation to the semi-empirically calculated unloading modulus, based on the equation in *TR Geo*, which then could be used to further calibrate the model.

For the final model, the permeability was adjusted for a better match against the measured data. It was considered reasonable to change the permeability as it was determined from compression tests. Furthermore, it was also more likely that the permeability was incorrectly estimated than  $\kappa^*$ , as the permeability was based only on laboratory measurements while  $\kappa^*$  was based on both laboratory tests and semi-empirical calculations.

For further validation of the input parameters used in the numerical analyses, it would have been preferable to perform laboratory testing at proper stress levels. To obtain values for  $\kappa^*$  and  $\lambda^*$ , oedometer tests, with loading and unloading, are recommended. Since  $\kappa^*$  is the parameter determining the unloading behavior which is of the highest importance for excavation problems, focus should have been put into unloading tests at proper stress intervals. The available data were mostly loading tests, making it hard to evaluate  $\kappa^*$  properly.

### 7.1.2 Validation of Model Setup

The calibration is an important part to retrieve reliable results. The purpose of the calibration was to create a model which had a good representation of an actual excavation. With a model that to some extent can represent the reality, the investigations could be continued from a validated model into a predictive model for estimations of non-measured data.

The groundwater and pore water pressures is one of the most influential factors in this research. Hence, the models validity was greatly improved when it could be shown that the groundwater in the numerical models acted as calculated in the analytical solution. The analytical calculations provided pore pressures for the different times and could thus be used as a reference to the same times in the numerical model, ensuring that the groundwater was modelled correctly.

The pore pressure has been considered the most important to calibrate against. This because it is the negative excess pore pressure that strongly governs the heave, and thus also the pressure caused by restraining the heave. Firstly, the pore pressures were matched at the boundaries top and bottom, and lastly the behavior of the pore pressure, i.e. the change with time, was matched. By matching both the boundaries and the behavior of the pore pressure the model was concluded to represent the calibration site well for these properties. As the model after this final calibration presented a good match with the calibration data, it was deemed suitable for further analysis of heave pressures for the specific model and parameter setup.

## 7.2 Heave Pressure

The focus of this research has been to find a relationship between the unloading of a soft soil and the pressure that can be caused by a followed restrained heave. This phenomenon of heave pressure is created due to the negative excess pore pressures from the unloading. As these pore pressures prohibit the swelling of the clay, a pressure can be created when the system is confined and these pore pressures dissipate.

The heave pressure as seen in Figure 6.3 is increasing with excavation depth. However, even though the total pressure is increasing the pressure-unloading ratio is not, indicating that far from all of the unloading is resulting in an upward lifting force. There are multiple known reasons for why the pressure-unloading ratio cannot be equal to one; the initial momentary heave, the groundwater lowering, the decrease of clay layer thickness and the non-purely elastic process of heave.

A possible reason for why the pressure-unloading ratio is decreasing with excavation depth could be as a result of the small increase of heave development with excavation depth. Furthermore, since the shape of the curves are relatively similar in respect to excavation depth, it suggests that the shape is coupled with the amount of unloading, rather than the consolidation time. However, the time affects the ratio in magnitude, and also in a manner which decreases the gradient rather than altering the trend. It is also likely that the pressure-unloading ratio will continue to decrease with depth, as less and less clay will remain in the system below the excavation level and be able to heave.

### 7.2.1 Influence of Time

The consolidation time before the system is locked down, i.e. before the slab is casted, highly affects the remaining pressures. In order to determine the correlation between the consolidation time and the pressures caused by the restrained heave it is essential to investigate the time-dependency of heave.

The heave is dependent on how fast the water can travel through the clay and how far it is to the water source. This means that the boundary conditions of the model, and the permeability of the clay, have high impact on the developed heave. The permeability of the clay layers were increased in order to match the calibration against the Ullevi excavation which means that if they were set too high, the heave develops faster than it should in reality. It is not possible to set a general time-limit for how long it takes for a soil to heave, since the permeability and the clay depth vary in different places. It can however be stated that the heave is strongly time-dependent. It should be noted that only around 10 to 15% is seen to develop momentarily, whereas the rest develops over time.

From the figure displaying heave development with time, Figure 6.2, it can be seen that the heave development is almost unaffected by the excavation depth. This is due to that the total clay depth, and hence the distance that the water have to travel, is only slightly affected by the excavation depths which are small in relation to the total depth. From the graph it can also be seen that the heave development is faster in the beginning and then it slows down exponentially.

It should be noted that the numerical analyses does not account for the time it takes to perform the excavation. When considering how long the excavation is open before the system is locked down, it is important to also account for the time it takes to excavate. Moreover, the time it takes to build the framework and to reinforce the concrete effects how long the excavation will be left open. In theory, it is of course possible to lock the system down immediately after the excavation is finished, but in reality the time that passes before the system is locked is most often substantial. A large part of the heave will, therefore, have been developed before the remaining part of the heave is locked down. Also, if there is a layer of fill or air beneath the slab, this will reduce the contact between the slab and the clay, hence decrease the heave pressure.

### 7.2.2 Influence of Material Properties

Shifting the focus of the heave pressure from a theoretical result to a more realistic result, cohesion was added to the piles. As expected, the pressure decreased as results of piles holding back the heave, as seen in Figure 6.6. The effect is seen regardless of the time before the slab is casted. The magnitude of the effect is also dependent on the clay-pile interaction factor. As seen in Figure 6.7, if more piles are added, the pressure decreases. It can thus be concluded that the cohesion affect the pressure caused by restraining the heave. However, it is hard to say with which magnitude it affects the pressure, as it is a direct result of what is given as an interaction factor, i.e. the more cohesion the less pressure. As a limitation of this study, the effect of pile installation has not been considered. However, the installation of piles would result in a reduction of stiffness of the clay and an increase of pore pressure, making the total effect on heave pressure hard to predict.

Another factor influencing the obtained pressures is the clay depth. The further the distance is to the water source, the longer it takes for the pore pressures to equalize, and the longer it takes for the soil to heave. The clay cannot swell, and hence it does not want to heave if it has no access to water. This explains the results as presented in Figure 6.5, for the pressure-unloading ratio for different depths, where the difference is small if the slab is casted immediately after the excavation, but substantial if the slab is casted after 180 days. The additional clay depth does not influence the maximum heave pressure, but only the decay rate of the pressure as a result of the increased heave time.

One of the most influential structural properties is the stiffness of the slab and the stiffness of the piles. When making these infinitely stiff, the system is held down and can withstand almost any pressure pushing against the slab without deforming hence obtaining maximum pressures. If the stiffness of the slab is decreased, without decreasing the stiffness of the piles, the slab will deform between the piles, allowing the soil to heave and hence not creating a heave pressure. If both the slab and the piles are assigned realistic stiffness, and with a free bottom connection, the slab will not only deform, but the entire system will be free to move. The two observed phenomena of deformation and displacements makes it difficult when wanting to model a realistic system with realistic stiffness since deformations will occur and hence less pressures will be generated. The heave pressure is, therefore, directly

linked to the stiffness of the system, i.e. how locked the system is. This means that in real construction projects, where the system is not infinitely stiff, the pressures will decrease due to the ability the system has to displace and deform.

### 7.2.3 Uncertainties

The greatest uncertainty with the model has been that there are no measurements of the heave pressure, and thus could not the results be validated in that way. It should be remembered while reviewing the results that the model is calibrated against one of the most influential parameters, i.e. the pore pressure, but not the actual heave pressure. However, the scope of this study is to determine if the phenomenon could cause a problem, and not to determine a template which could be used when designing slabs. This is something that the model is believed to do as it is calibrated and because of the mechanism used to lock the heave. The locking mechanism is weightless and infinitely stiff thus resulting in a system which acts as a cap on the soil but without influencing the stress state or groundwater.

When moving from a theoretical case, as done in this study, to a more realistic one applicable to reality, more factors of influence needs to be considered. Some which affect the heave pressure, e.g. generated pore overpressure and soil disturbance from piling which would decrease this pressure. Hence, the pressure caused by restrained heave would likely be lower than what this study shows. Furthermore, it is not only the pressure caused by restrained heave which acts on the slab but also other forces, e.g. shear surfaces from adjacent slopes and water pressures which also generates pressures on the slab.

To relate back to the question regarding if the obtained results are providing the correct answers, there is no uncomplicated interpretation. With the above discussion and the uncertainties stated it could be justified to say; yes, they are. Of course, the results are only valid for the stated parameters and model setup and hence not in a general way but still verified and validated for the specific case. However, the answer could also be; no, they are not. The results are theoretical and not applicable for a real construction project. It does, however, provide a theoretic base for the maximum magnitude of the heave pressure. It should be noted though, as stated, that making the model more realistic would severely reduce the pressures, and hence they might not be a problem during construction. Before that question can be answered it is of great importance to continue the study and also perform field measurements of heave pressure against slabs.

## 7.3 Further Studies

This thesis provides a first investigation of the heave pressures that can be created due to containment of heave. For further clarification of the phenomena of heave and swelling of soft soils, further studies needs to be carried out.

### Field Measurements

The most effective way of determining the heave pressure would of course be to measure these pressures in the field during, and after, construction. It would, therefore, be an idea to install equipment attached to a concrete slab for in-situ measurements of the pressures. These measurements would give an idea of the problem, and could be used for validation of the numerical analyses. If this data were available, a continued study calibrated against the actual pressures, could be performed with higher validity.

### Modelling of Realistic Case

Even though it is not possible to capture the heave pressure in a model with realistic stiffness, it could be an idea to instead investigate the deformations and displacements. It has been seen that the pressures decrease when the system is made less stiff but the actual deformations have not been looked into. If the deformations, together with the moment curves of the slab, were studied it would be possible to back-calculate the pressures that has caused these deformations. It should also be noted that it is not only the pressures that are of interest, instead the actual affect that these have on the existing structures should be further investigated.

### Mineralogy of Clay

Finally, it has been stated earlier in this thesis that the type of swelling occurring in expansive clay are not relevant for the Gothenburg region. However, there could be a possibility, or risk, that some areas in this region may contain the mineral Montmorillonite to a very low extent, which could result in crystalline swelling. If this were to be the case, the magnitude of the swelling pressures that could be created could be higher (previous studies show that pressures of around 1 000 kPa can be obtained from crystalline swelling) than the ones created by unloading. For further studies it is therefore recommended to look into the mineralogy of the clay of interest and to determine the impact of these.

## 7.4 Conclusions

The study have shown that the heave created due to unloading is a strongly time-dependent process. Furthermore, this heave will cause a pressure against the slab if the system is locked and prevented from heaving. With regard to the time-dependency, it is not possible to state a general limit for how long it takes for the soil to heave, but it has been shown that the heave development is faster in the beginning and then slowing down exponentially. For the studied case, with a clay depth of 35 m, it takes around 200 days for 80% of the heave to develop but almost 1 500 days for 100% of the heave to develop. If the clay depth is doubled, the time it takes for the soil to heave is significantly longer.

It can be concluded that the heave pressure is increasing with excavation depth while the pressure-unloading ratio is not. It has been shown that the magnitude of

the maximum obtainable heave pressures are dependent on the excavation depth as well as the time before the slab is casted and the system is locked. If the slab is casted immediately after the excavation is finished, the heave pressure is around 20 kPa at an excavation depth of 2 m and 60 kPa at an excavation depth of 8 m. However, if the excavation is left open for 90 days, which is more realistic, the same pressures are 8 kPa and 18 kPa respectively. As for the relation between the heave pressure and the amount of unloading, the ratio is decreasing with increasing excavation depth, indicating that far from all of the unloading is resulting in an upward lifting force. The percentage of the unloading that comes back as an upward heave pressure is in the range of 55% if the slab is casted immediately and 22% if the excavation is open for 90 days.

The heave pressure is affected by several factors besides the consolidation time before the system is locked: the material parameters of the structural elements and the parameters of the underlying clay. The clay parameters with the largest effect are the unloading modulus and the permeability, together with boundary conditions and groundwater settings. Furthermore, the heave pressure is directly linked to the stiffness of the structural system, i.e. how locked the system is. This means that in real construction projects, where the system is not infinitely stiff, the pressures will decrease due to the ability the system has to displace and deform. The pressure also decreases when cohesion is added to the piles, as a result of the piles holding back the heave. In addition, the amount of developed heave will depend on the time the excavation has been open and also the total clay depth. However, the clay depth does not influence the maximum heave pressure, but only the decay rate of the pressure as a result of the increased heave time. Further, a layer of fill or air beneath the slab would reduce the contact between the slab and the clay. Due to this, in combination with several other factors, the heave pressures would likely be lower in reality than what this study shows.

The greatest uncertainty with the analysis has been that there are no measurements of the pressure caused by the restrained heave and thus the results could not be verified in that way, creating a situation where a realistic solution would include too many uncertainties. It was therefore concluded that for this study it was of more interest to create an upper bound solution for the magnitude of the heave pressures and the pressure-unloading ratio. As it is believed that these upper bound solutions give a first insight to the phenomenon of heave pressures and what causes them. It should also be considered that the analyses have been performed numerically, with the finite element method, only providing approximate solutions. Nevertheless, as the analysis is made with a simple, one-dimensional model with few components, it is considered sufficient enough for providing an introduction to the phenomenon.

Finally, it should be noted that the heave pressures need to be combined with other types of pressure, e.g. water pressures, shear force etc., when designing concrete slabs to withstand pressure from underneath. The phenomenon of heave pressures also needs to be further investigated and adjusted for specific conditions at the construction site in order to provide better knowledge of how the concrete slabs will be affected. When comparing the results to calculations according to current practice, it is indicated that the heave pressure is overestimated by the current methods. The results presented within this thesis and the conclusions that have been drawn

are only providing an approximation of the heave pressures within the limitations and scope of this thesis.

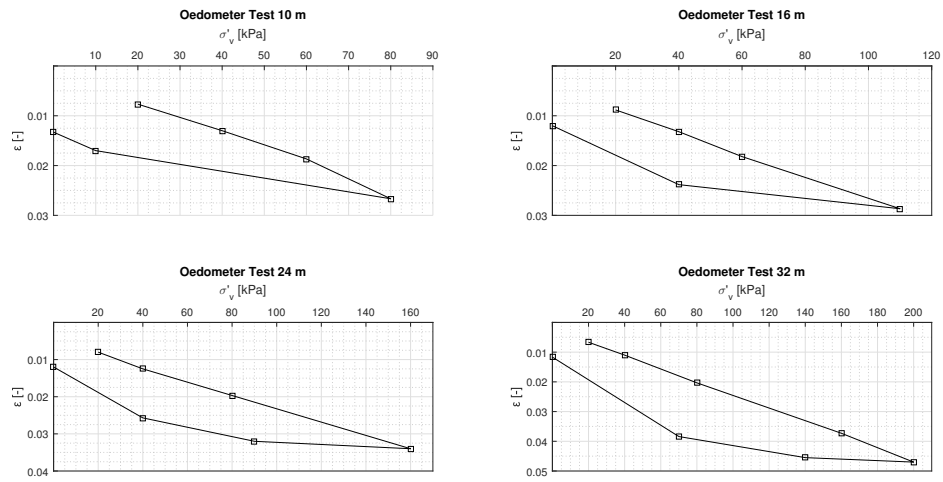
# Bibliography

- Alén, C. (1998), On probability in geotechnics, Vol. 2, PhD thesis, Department of Geotechnical Engineering, Chalmers University of Technology.
- Alén, C. and Jendeby, L. (1996), Hävning vid avschaktning - Ullevi, ett praktikfall, *in* 'NGM-96', Reykjavík, Island, pp. 353–358.
- Ashayeri, I. and Yasrebi, S. (2009), 'Free-swell and swelling pressure of unsaturated compacted clays; experiments and neural networks modeling', *Geotechnical and Geological Engineering* **27**(1), 137–153.
- Briaud, J. (2013), *Geotechnical Engineering: Unsaturated and Saturated Soils*, 1 edn, John Wiley & Sons, Incorporated, Hoboken, New Jersey.
- Brinkgreve, R., Kumarswamy, S. and Swolfs, W., eds (2016), *Plaxis Material Models Manual*, Plaxis bv, Delft, Netherlands.
- Broms, B. and Hansbo, S. (1981), 'Foundations on soft clay', *Soft Clay Engineering: Developments in Geotechnical Engineering* **20**, 415–478.
- Budhu, M. (1999), *Soil Mechanics and Foundations*, 1 edn, John Wiley & Sons inc., New York.
- Friis, Å. and Sandros, C. (1994), *The heave due to large excavations in clay, Ullevi - a case study*, Chalmers Univeristy of Technology (Master Thesis in Department of Geotechnical Engineering), Gothenburg.
- Hanafy, E. (1991), 'Swelling/Shrinkage Characteristic Curve of Desiccated Expansive Clays', *Geotechnical Testing Journal, GTJODJ* **14**(2), 206–211.
- Jendeby, L. (1986), Friction piled foundations in soft clay - a study of load transfer and settlements, PhD thesis, Chalmers University of Technology.
- Johansson, B. (1992), Ullevi, Göteborg Grundförstärkning och garage, Technical report, NCC, Göteborg.
- Karlsrud, K. (2003), Deformation Properties for Use in Geotechnical Calculations, Technical Note 2G-201, Technical report, Norwegian Geotechnical Institute, Oslo (*In Norwegian*).
- Karlsrud, K. and Hernandez-Martinez, F. G. (2013), 'Strength and deformation properties of Norwegian clays from laboratory tests on high-quality block samples 1', *Canadian Geotechnical Journal* **50**(12), 1273–1293.
- Karlsson, M. and Moritz, L. (2016a), Trafikverkets tekniska krav för geokonstruktioner-TK Geo 13, Technical report, Trafikverket.

- Karlsson, M. and Moritz, L. (2016b), Trafikverkets tekniska råd för geokonstruktioner-TR Geo 13, Technical report, Trafikverket.
- Kennedy, H. (2003), *Displacements due to deep excavation in soft clay - comparison between calculated and measured displacement*, Chalmers University of Technology (Master Thesis in Department of Geotechnical Engineering), Göteborg.
- Knappett, J. and Craig, R. F. (2012), *Craig's Soil Mechanics*, 8 edn, Spon Press, Abingdon.
- Larsson, R. (1986), Consolidation of soft soils, Technical report, Statens geotekniska institut, Linköping.
- Massat, L., Cuisinier, O., Bihannic, I., Claret, F., Pelletier, M., Masrouri, F. and Gaboreau, S. (2016), 'Swelling pressure development and inter-aggregate porosity evolution upon hydration of a compacted swelling clay', *Applied Clay Science*.
- Morissette, L., McRostie, G. C. and St-Louis, M. W. (1996), 'Bottom-heave control of a deep sensitive clay excavation in Ottawa, Canada', *Canadian Geotechnical Journal* **33**(6), 926–936.
- Muir Wood, D. (2004), *Geotechnical modelling*, Spon Press, New York.
- Olsson, M. (2010), Calculating long-term settlement in soft clays, Technical report, Swedish Geotechnical Institute, Linköping.
- Persson, J. (2004), *The unloading modulus of soft clay: A field and laboratory study*, Chalmers University of Technology (Licentiate in Department of GeoEngineering), Gothenburg.
- Phanikumar, B. R. and Singla, R. (2016), 'Swell-consolidation characteristics of fibre-reinforced expansive soils', *Soils and Foundations* **56**(1), 138–143.
- SS-EN 1997-1 (2005), 'Eurocode 7: Geotechnical design - Part 1: General rules'.
- Vägverket (2001), Reviderings PM 1, Entreprenad J2 Götatunneln, Technical report, Vägverket, Göteborg.
- Wong, Y. K. (2013), Comparison of Drainage Line Elements in PLAXIS 2D and 3D applied in Consolidating Marine Clay Deposits, Technical report, SEIC, Singapore.
- Zeevaert, L. (1983), *Foundation engineering for difficult subsoil conditions*, 2 edn, Van Nostrand Reinhold Company Inc, New York.

# A

## Oedometer Tests from Ullevi



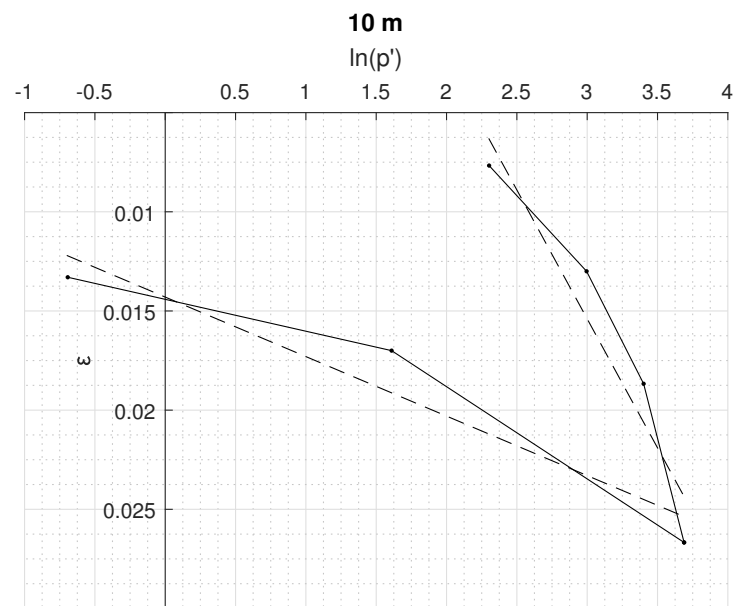
**Figure A.1:** Oedometer tests (Friis and Sandros, 1994)



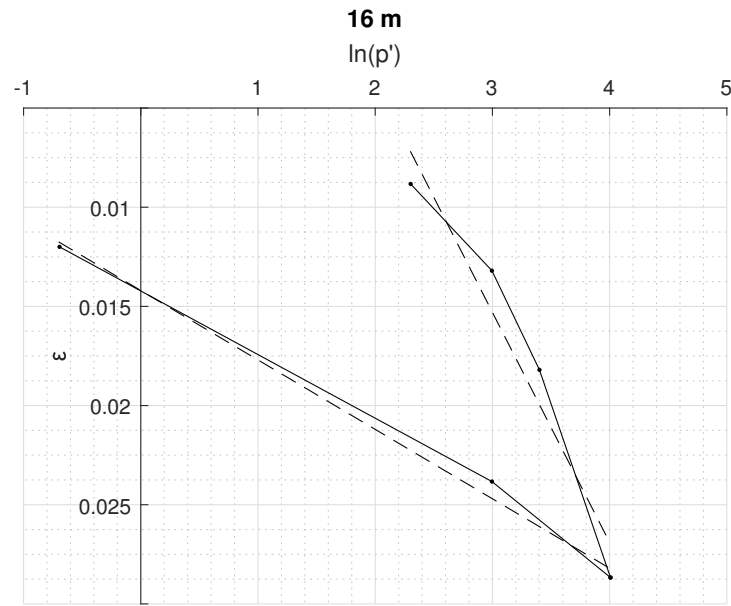
# B

## Evaluation of $\kappa^*$ & $\lambda^*$

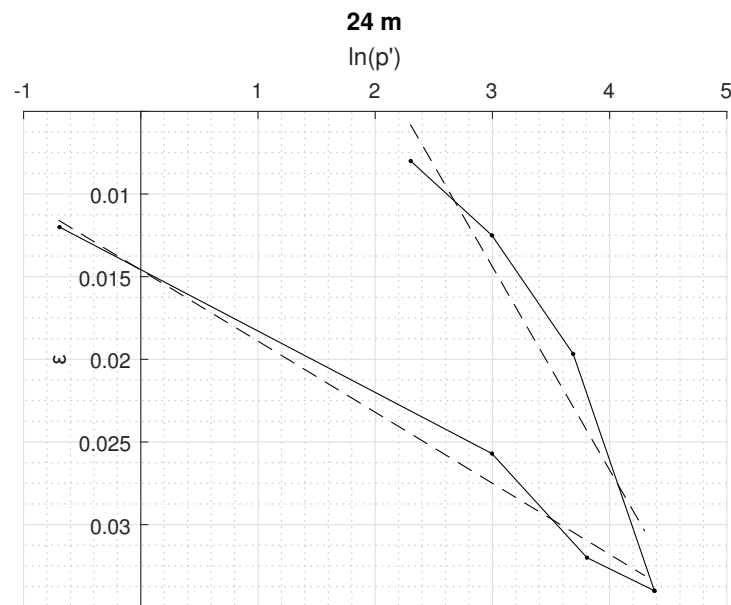
### Oedometer Tests



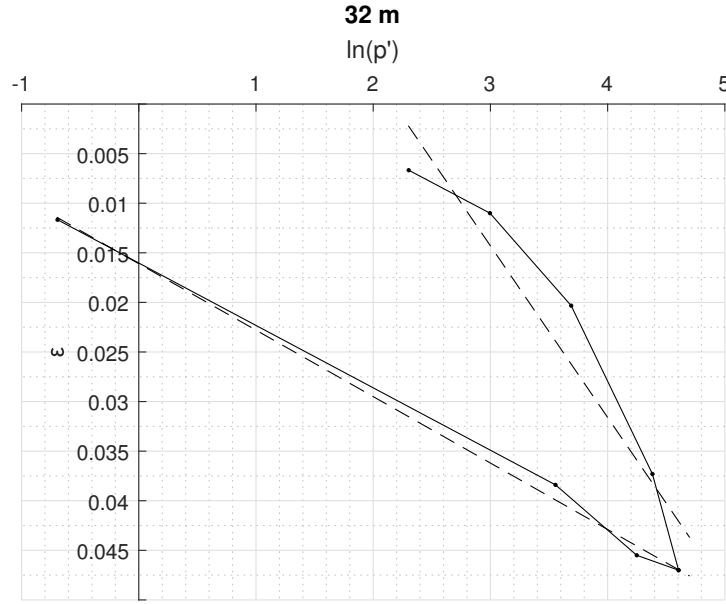
**Figure B.1:** Logarithmized oedometer test with the  $\kappa^*$  and  $\lambda^*$  trend lines at 10 m depth



**Figure B.2:** Logarithmized oedometer test with the  $\kappa^*$  and  $\lambda^*$  trend lines at 16 m depth



**Figure B.3:** Logarithmized oedometer test with the  $\kappa^*$  and  $\lambda^*$  trend lines at 24 m depth



**Figure B.4:** Logarithmized oedometer test with the  $\kappa^*$  and  $\lambda^*$  trend lines at 32 m depth

## CRS Tests

**Table B.1:**  $\kappa^*$  and  $\lambda^*$  from CRS tests (Johansson, 1992)

| Depth | $M_L$ | $\sigma'_c$ | $\sigma'_l$ | $\sigma'_0$ | $\sigma'_{vc}$ | $\lambda^*$ | $\kappa^*$ |
|-------|-------|-------------|-------------|-------------|----------------|-------------|------------|
| 3     | 721   | 74          | 133         | 25          | 79             | 0.121       | 0.012      |
| 5     | 566   | 62          | 132         | 35          | 83.5           | 0.162       | 0.016      |
| 7     | 426   | 80          | 115         | 45          | 80             | 0.207       | 0.021      |
| 10    | 497   | 115         | 149         | 60          | 104.5          | 0.231       | 0.023      |
| 15    | 573   | 163         | 206         | 95          | 150.5          | 0.289       | 0.029      |
| 20    | 967   | 197         | 255         | 130         | 192.5          | 0.219       | 0.022      |
| 26    | 1192  | 236         | 302         | 168         | 235            | 0.217       | 0.022      |

With  $\lambda^*$  calculated according to the following equation as proposed by Olsson (2010)

$$\lambda^* = \frac{1.1\sigma'_{vc}}{M_L} \quad (\text{B.1})$$

where  $\sigma'_{vc}$  is the average between the preconsolidation stress and the defined stress  $\sigma'_c$  and  $M_L$  is the constrained modulus between  $\sigma'_c$  and  $\sigma'_L$ .