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Numerical Modelling of Subsidence due to Tunnel Excavation

Master's thesis in Master's Programme Infrastructure and Environmental Engineering

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Department of Architecture and Civil Engineering
Division of Geology and Geotechnics
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Gothenburg, Sweden 2019

MASTER’S THESIS THE MASTER PROGRAMME INFRASTRUCTURE AND ENVIRONMENTAL ENGINEERING

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Abstract

The West Link Project, Västlänken in Swedish, is an infrastructure project in Gothenburg. The main goal of the project is to increase the capacity of the railway network, by constructing eight kilometres of railway tunnel and three new underground stations. The tunnel will go through both rock and soft soils, such as clay. The parts driven through clay will be constructed with the cut-and cover method and will therefore entail deep excavations well below the groundwater table. As the excavations will be both easier and safer to construct in dry conditions, may groundwater lowering actions during the construction time occur. When lowering the groundwater table, the surrounding soil can be affected. A change in the pore water pressure in the soil could result in consolidation settlements, called subsidence. Subsidence can cause differential settlements and other severe damage, affecting surrounding buildings and infrastructure. When reviewing the groundwater conditions, complex conditions were found at the Korsvägen site, where one of the underground stations is to be constructed. It was then decided that the aim of the thesis would be to analyse and model the subsidence due to groundwater lowering actions in an excavation at Korsvägen constructed as part of the West Link project. To carry out the analyses a representative cross section was created, using geometry and soil data from the Korsvägen site. A model was then created in Plaxis 2D to study the subsidence using representative constitutive models and consolidation analysis. The model was also used to investigate how the uncertainties in determining soil parameters affect the results. Furthermore, the influence radius of the subsidence was examined, to see if adjacent buildings and infrastructure would be affected.

When reviewing the stratigraphy, a rather horizontal soil layering was found, with a clay layer approximately 20 meters thick. In the middle of the clay layer a permeable layer of friction material was found. The bottom layer is a stiff friction layer, which mainly consists of sand and overlies the bedrock. The uncertainties of the model were mainly related to the two friction layers, specifically permeability and stratigraphy. Furthermore, the strength and stiffness of the bottom friction layer was varied. The chosen solution for the retaining wall was a back-anchored secant pile wall. Groundwater management will be complicated since the groundwater conditions are very sensitive to change. The secant pile wall is deemed impermeable and to avoid seepage a grout curtain was needed to be injected underneath the wall. The influence of the grout curtain was further studied, by running a fully coupled flow-deformation analysis to investigate the groundwater flows.

When reviewing the result could it be seen that the magnitude of settlements and wall displacements were most affected when the stratigraphy and the strength and stiffness parameters were varied. Varying the permeability mainly influenced the pore pressure distributions. When reviewing the grout curtain was it seen that a prolonged curtain efficiently cut off the groundwater flow beneath the wall. In an urban environment a small variance in settlements or differential settlements can cause severe damages. Subsidence could therefore be an issue at Korsvägen, mainly since the area round the future excavation is so densely built-up, which could be problematic since the potential subsidence is largest close to the excavation.

Key words: *Subsidence, West Link Project, Plaxis 2D, Uncertainties in Parametric Determination*

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Notations

γ_{sat}	Saturated Unit Weight	[kN/m^3]
γ_{unsat}	Unsaturated Unit Weight	[kN/m^3]
k	Permeability	[m/day]
φ	Friction angle	[°]
ψ	Dilatancy angle	[°]
ν_{ur}	Poisson's ratio for unloading/reloading	[-]
λ*	Modified compression index	[-]
κ*	Modified swelling index	[-]
c'	Effective cohesion	[kPa]
K_0	Coefficient of lateral stress	[-]
K_0^{NC}	Coefficient of lateral stress for NC soil	[-]
E	Young's Modulus	[kPa]

Abbreviations

LE	Linear Elastic Constitutive model
MC	Mohr-Coulomb Constitutive model
NC	Normally consolidated
OCR	Over-Consolidation Ratio
SS	Plaxis Soft soil model

1 Introduction

The West Link Project, Västlänken in Swedish, is one of the largest infrastructural projects in Gothenburg. The main goal of the project is to increase the capacity of the railway network and make commuting within Gothenburg and the whole of West Sweden more efficient. To succeed with this ambitious goal, eight kilometres of railway tunnel for regional and commuter trains and three underground stations will be constructed (Trafikverket, 2016). The tunnel will go through both rock and soft soils, such as clay. The parts driven through clay will be constructed with the cut-and cover method and will therefore entail deep excavations below the groundwater table (Trafikverket, 2014a). As the excavation will be both easier and safer to execute in dry conditions, there will be need for groundwater lowering actions during the construction time. When lowering the groundwater table, the surrounding soil may be affected. A change in pore water pressure in the soil could result in both elastic and plastic consolidation settlements, called subsidence. Subsidence could result in differential settlements which could damage surrounding buildings and infrastructure (Cashman & Preene, 2001). To investigate subsidence and the impact of groundwater lowering will a representative excavation from the West link project be analysed. The chosen excavation is located at Korsvägen, a central transfer point with important infrastructure and buildings. Around Korsvägen a variety of building types can be found, modern Gothia Towers and the Swedish Exhibition and Congresse Centre alongside buildings from the 18th Century. Foundation types also vary, generally the surrounding buildings are piled, either to bedrock or with cohesion piles. However, some foundations are constructed of sensitive wooden piles. Hence, subsidence and groundwater lowering actions could cause severe damages here (Sweco Civil AB, 2014).

Subsidence is often modelled as a one-dimensional problem, which is valid when the pore pressures are lowered in the whole aquifer via pumping. However, the flow of water to a cut-and-cover excavation is truly a three-dimensional, rate-dependent problem. Modelling and analysing the excavation and the surrounding conditions in three dimensions would prove very complicated, since the geological and hydrogeological conditions are quite irregular and complex. To attempt to model these complex conditions the finite element software Plaxis 2D will be used.

By investigating subsidence and the mechanism behind it, an increased understanding can be achieved. This knowledge can be used to optimize future excavation design, the application of observational methods and instrumentation, and lastly to prevent possible damage to nearby buildings.

1.1 Aim

The aim of the thesis is to better understand the issues regarding subsidence in urban areas due to lowering of the groundwater table when constructing deep excavations, by analysing an excavation with two-dimensional consolidation analyses. The analysis will be carried out by modelling a representative cross section of an excavation inspired by the ground conditions at Korsvägen, as part of the West Link project. Furthermore, is the aim to investigate how the ground conditions and the uncertainties in determining soil parameters affect the subsidence around the excavation. Lastly, the influence radius of the subsidence will be examined, to see if and how adjacent buildings with sensitive foundations will be affected.

1.2 Objectives

The specific objectives of the thesis are listed below:

- Gain knowledge of the mechanism and hazards of subsidence.
- Create a representative model excavation, based the soil conditions and geometry from the Korsvägen site.
- Investigate the subsidence of the model excavation using Plaxis 2D.
- Vary soil parameters to investigate the impact of certain soil properties.
- Examine and analyse the area influenced by the subsidence. Will any buildings in the proximity be affected?

1.3 Limitations

When investigating the subsidence only one excavation pit design will be used. The stratigraphy and the excavation geometry will be idealized, to simplify the calculations from a complex 3D situation to 2D. All calculations in Plaxis will be in 2D, as rate-dependent analyses, using standard soil models. Furthermore, the all analysis will be carried out using plane-strain conditions and the effects of creep is not considered.

1.4 Method

To assess subsidence the finite element software Plaxis 2D was used, to enable consideration of time dependent consolidation and dissipation of excess pore pressures. The analysis was carried out using plane-strain, to be able to assess the impact of groundwater lowering on an infinitely long excavation. How the analysis was conducted is further presented in Chapter 4. Also, a literature study was executed to understand the mechanisms and difficulties of the issue.

1.5 Thesis Outline

The thesis consists of 9 chapters, including the introduction chapter. A brief overview over each chapter is presented below:

1) *Introduction*
2) *Theoretical Background* – Theoretical background to the thesis, including the conducted literature study.
3) *Model Excavation* – Description of model excavation, consisting of geotechnical, geological and hydrological conditions as well as geometry and boundary conditions of the excavation.
4) *Method* – Explanation of the numerical analysis, presenting the models used, soil parameters and sources of data for determination.
5) *Result* – The result chapter consists of two parts, first the results from the parametric study and secondly from the numerical analyses.
6) *Discussion* – Obtained results are analysed and sources of error identified.
7) *Conclusion*
8) *Further Investigation*
9) *References*

2 Theoretical Background

The following chapter will address important aspects vital for understanding the subject of subsidence due to groundwater lowering and how to model it all using Plaxis 2D.

2.1 Groundwater

Groundwater is formed as a part of the hydraulic cycle and defined as the portion of precipitation that is infiltrated in the ground down to the water table. Only a very small part of the precipitation forms groundwater, since run-off, evaporation and evapotranspiration prevent the water from forming groundwater. The hydraulic cycle in urban environments can be seen as altered, since previously permeable surfaces has been paved, covered and built-up. Lerner (1990) explains this further and identifies both changes in run-off paths and new potential sources for recharge, such as leaking water mains and sewers. The groundwater conditions are thus important to consider when constructing and designing excavations in urban environments. The conditions are important to investigate, to ensure a safe and sound design, but also to determine how sensitive the system is. The following section aims to present how groundwater can be stored in the ground and how to investigate and monitor groundwater and its properties.

2.1.1 Aquifer types

When discussing groundwater, the term aquifer is often used. Cashman & Preene (2001) defines an aquifer as ‘A stratum of soil of rock which can yield groundwater in economic or productive quantities’. However, this definition is not very useful when performing groundwater lowering since any soil that yield water can cause problems when lowering the groundwater table. An other definition is therefore given by Cashman & Preene (2001), to better suit the groundwater lowering point of view. Definitions are also given for aquiclude and aquitards.

- *Aquifer* – ‘Soil or rock forming a stratum, group of strata or part of stratum that is water-bearing (i.e. saturated and permeable)’.
- *Aquiclude* – ‘Soil or rock forming a stratum, group of strata or part of stratum of very low permeability, which acts as a barrier to groundwater flow.’
- *Aquitard* - ‘Soil or rock forming a stratum, group of strata or part of stratum of intermediate to low permeability, which yields only very small groundwater flows.’

Permeability of the soil is often closely linked to the destinction of the aquifer, typical values of soil types are tabulated below in Table 2.1.

Table 2.1. Typical values of permeability based on soil type from Cashman & Preene (2001).

Soil Type	*Classification of permeability*	*Permeability*	*Unit*
Gravelly Sands	High to moderate	1E-3 to 5E-4	[m/s]
Fine to medium Sands	Moderate to low	5E-4 to 1E-4	[m/s]
Silty Sands	Low	1E-4 to 1E-6	[m/s]
Sandy Silts, with clay fractions	Low to very low	1E-7 to 1E-8	[m/s]
Intact Clays	Practically impermeable	<1E-9	[m/s]

2.1.2 Investigation Methods and Monitoring

Important properties to investigate regarding groundwater conditions to measure are pressure head, permeability of the strata and influence radius. Monitoring of groundwater parameter is also vital to ensure a successful project. The parameters to monitor largely depend on the project characteristics, but the groundwater level is generally monitored. Piezometers are instruments for continuous measurement of hydraulic head. Since the measuring is continuous, the piezomenters can also be used for monitoring the hydraulic head. Groundwater levels can also be investigated and monitored used installed standpipes and dipmeters (Cashman & Preene, 2001). Additionally, a test pumping can be performed. This pumping results in reliable measurements of transmissivity, re-charge and boundaries of the aquifer, as well as storage coefficient (Carlsson & Gustafson, 1997). The test well is most commonly a deep well with an electric pump and surrounding it piezometers are installed to measure the drawdown and possible recharge. The test pumping can also help determining the influence radius (Powers & al, 2007).

Lastly, to investigate the hydraulic conductivity of the soil, an additional slug test can be performed. The test is conducted in an observational well, by creating a instantaneous pressure disturbance and measuring the response. Creating a disturbance is often made by quickly lowering or raising the groundwater level in the borehole using a slug (Engelbrektsson, 2016).

2.2 Dewatering in Construction

Tunnelling and excavating below the groundwater table is today common, often in urban environments. The following section will further present why such actions are necessary, how the groundwater can be lowered and modelled, and lastly the consequences and risks of dewatering are discussed.

2.2.1 Purpose

Dewatering excavations serves many different purposes. First and foremost, dewatering does increase the work environment safety. Leakage of groundwater could cause a failure of the supporting walls, risking the workers safety as well as the surrounding environment. When lowering the groundwater level in the area surrounding excavation the lateral forces are reduced, which reduces the failure risks of temporary constructions such as sheet-pile walls. Secondly, a groundwater lowering could reduce the risk of heave failure, if the groundwater pressure is greater than the weight of the overburden soil an uplift failure could occur. Thirdly, the lowering of the water table can improve the stability of excavation slopes, since the erosion due to seepage is minimized. However, when only lowering the groundwater inside the excavation the working environment is improved, the construction work can be carried out in a dry environment instead of under water (Cashman & Preene, 2001).

2.2.2 Methods of Dewatering

How to lower the groundwater table or manage the groundwater depends on many factors, the most important parameters are the geotechnical and hydrological properties. These properties must be carefully determined and identified to ensure that an appropriate dewatering measure is used. Powers & al (2007) identify six additional aspects to keep in mind when deciding what dewatering method to use. The aspects are as presented below;

- Geometry of excavation pit, including depth and size.
- Chosen excavation method and proposed ground support measures.
- Proposed foundation of building or structure, including geometry and type.
- Proximity to existing buildings and infrastructure, important to also take their foundation type into account.
- Time plan for the construction of the excavation.
- Contaminated soil must be considered, both on site and in the surrounding area.

Based on these aspects a control or dewatering measure can be decided. Powers & al (2007) propeses four different basic models:

1) *Open pumping* – groundwater is allowed to enter the excavation pit and thereafter collected in ditches and pumped out of the pit.
2) *Predrainage* – the groundwater level is lowered before excavating, using deep wells or wellpoints.
3) *Groundwater cut-off* – the groundwater flow towards the excavation is cut off, by creating an impermeable barrier. Common barriers are sheet pile walls and diaphragm walls, other less conventional methods such as ground freezing can be used.
4) *Groundwater exclusion* – the groundwater is excluded from the excavation by compressed air, ground freezing or earth pressure shields.

2.2.3 Modelling of Dewatering

When modelling groundwater lowering or dewatering can a number of different approaches be used. Since all analyses will be carried out in Plaxis 2D will the following section entail methods of modelling dewatering in said software.

- *Well* – A predefined feature in Plaxis, input needed for using the feature is well behaviour, discharge of well and minimum head of well. The well feature can also be used for infiltration, as well as extraction (Plaxis, 2019a).
- *Drains* – Also a predefined feature. To use the drain feature in analysis is the behaviour type and groundwater head needed as input (Plaxis, 2019a).
- *Manual Lowering* – Since the groundwater level is user defined can it be moved manually when defining the calculation phases. The lowering can be made stepwise or all-in one.
- *Flow Function* – A time dependent lowering can also be implemented, by introducing a time dependent decrease in hydraulic head (Plaxis, 2019a).
- *Setting Cluster Dry* – To model a dewatering can the soil inside the excavation be set to dry, meaning the degree of saturation is assumed to be 0 %. The global water level is thus kept at the initial level throughout the calculation (Plaxis, 2012).

To decide which method to choose can prove difficult, however can some general remarks be made. Firstly, quite elementary, the modelled system should be as close to reality as possible. However, if no knowledge is available must one keep in mind that the chosen modelling method could affect the result. According to Schweiger (2002) an all in one manual lowering can produce larger horizontal displacements, compared to a stepwise.

2.2.4 Consequences

When lowering the groundwater in construction, a number of possible negative consequences must be considered. Cashman & Preene (2001) identify several potential risks of groundwater lowering, these consequences are as follows; ground settlements as a result of inadequate groundwater control, loss fine soil particles or an increase in effective stresses. Settlements as a result of change in effective stress is in this report the definition of subsience, and the term and mechanism will be further presented in Section 2.3. Futhermore, the aquifer can be drained or emptied as a result of the dewatering. The aquifer could also be polluted, either by contaminants or saline intrusion. Lastly, can the surrounding environment be effected, lowing the groundwater can expose or dry out surrounding timper pile foundations, resulting in decomposing. Surrounding wetlands and vegetation could also be drained or desiccated (Cashman & Preene, 2001). The Swedish Transport Administration

(Trafikverket, 2018) further identifies a more local consequence; geothermal wells in the surrounding area could be affected by the groundwater lowering actions.

2.3 Subsidence

Subsidence is a very general term and can include a number of different mechanisms. However, the definition used here for subsidence refers to a consolidation settlement due to groundwater lowering. The following chapter will in further depth investigate the mechanisms and consequences of subsidence, but also limiting values and monitoring methods.

2.3.1 Mechanism

To further explain the mechanism of subsidence will the theory of consolidation first be very briefly presented. Knappett & Craig (2012) defines consolidation as 'The gradual reduction in volume of a fully saturated soil of low permeabilitydue to change of effective stress'. This change in effective stress could be due to lowering of groundwater, the reduction in pore pressures result in an increase of the effective stress according to Terzagi's formulation of effective stress. The increase in effective stress induces a reduction in void ratio, and thus also in volume. The magnitude of the consolidation settlement, due to change in effective stress, mainly depends on three factors (Cashman & Preene, 2001):

- *Soil type* – Presence of a highly compressible soil layer below the groundwater table highly influence the magnitude of settlements if it is affected by the change in pore pressure. The softer the material, and the thicker the deposit, the greater the resulting settlement.
- *Drawdown* – A great drawdown results in greater potential settlements.
- *Period of Pumping* – The longer the time of pumping is, the greater the settlement is in general. The pumping time can cause two different time-dependent effects on the settlements;
    - *Increasing drawdown* – When pumping during a longer time an increasing drawdown will naturally occur.
    - *Drainage from aquitards* – Potential aquitards drain vertically into the aquifer, leading to a subsequent settlement in the aquitard.

The type of groundwater basin also affects the subsidence. The drawdown in the aquifer is instantaneous, and the pore water reductions occurs simultaneously. Any settlements are due to compression of the soil and happen immediately. Consolidation settlements are gradual and can thus only arise in aquitards and aquicludes, where excess pore pressures can be accumulated (Cashman & Preene, 2001).

2.3.2 Limit values and Consequences

To evaluate the possible consequences or damages following subsidence can a number of methods be used. One way of assessing the possible damage of subsidence is according to Cashman & Preene (2001), as presented in Table 2.2.

Table 2.2. Possible consequences of subsidence divided into four categories. From Cashman & Preene (2001).

Risk Category	*Maximum Settlement [mm]*	*Building Tilt*	*Anticipated Effects*
Negligible	< 10	< 1/500	Superficial damage unlikely.
Slight	10 – 50	1/500 – 1/200	Possible superficial damage, unlikely to have structural significance.
Moderate	50 – 75	1/200 – 1/50	Expected superficial damage and possible structural damage to buildings; possible damage to rigid pipelines.
Severe	> 75	> 1/50	Expected structural damage to buildings and expected damage to rigid pipelines or possible damage to other pipelines.

Consequences can thus range from no to severe structural damage, depending on settlement magnitude and direction. Soil deformation tend to vary both in vertical and horizontal directions, due to spatial variation in soil conditions. Where the general movement of subsidence is vertical the risk of structural damages to surrounding infrastructure and buildings is minor. Where, on the other hand, the general deformations are horizontal or sloping, the risks of damage are greater since the stresses are more anisotropic (Feng, et al., 2008). The economical aspect is also vital, damages caused by extreme subsidence in areas such as Santa Clara Valley in California have been estimated at over 130 million dollars (Fowler, 1981).

2.3.3 Monitoring

Monitoring of subsidence, or of general settlements, can be carried out in several different way either by manual or automatic reading. Andersson & al (2015) divide the monitoring methods into three different categories; fixed geotechnical systems, geodetic methods based on terrestrial measuring and geodetic systems based on remote sensing. Below are some common methods listed based on Andersson & al (2015);

- *Extensometers* – An extensometer can measure a change in thickness of a set interval. The measured change is one-dimensional and limited to the specified depth. The method is sorted into the fixed geotechnical system categories.
- *Measuring stubs* – To measure the settlement is first a settlement plate installed at a certain depth and a rod is attached reaching the ground surface. On the surface is a measuring stub fixed to the rod and by levelling the stub can the elevation of the plate be given. The method is also a fixed geotechnical measuring system.
- *Spirit-levelling* – Manual terrestrial geodetic method, simple yet precise. The method entails measuring of vertical elevation, by using a spirit level, vertical rods and a reference point of known height.
- *Interferometric Synthetic Aperture Radar (InSAR)* – This solution uses Earth-orbiting satellites for measuring settlement, it is therefore a remote sensing geodetic method. Any changes are detected by first reflecting a radar signal off a specified area and measuring the travel time back to the satellite. The obtained image of then compared to an image of the same area at another time. By comparing these two is an interferogram produced, on this map can displacements be seen.

2.4 Deep Excavations

In today's society are construction and infrastructure projects becoming increasingly complex, deep excavations is nowadays often a necessity. To create an excavation can several different techniques be used. Some commonly used retention walls are presented below (Knappett & Craig, 2012);

- *Sheet Pile Wall* – A wall is installed by driving interlocking sheet piles into the ground. The result is a satisfactory barrier for water.
- *Contiguous Bored Piles* – Piles are cast using continuous flight auger rigs, meaning the piles are cast in place and not bored. The piles are not connected, in order for the wall to be impermeable must further grouting or other measures be performed.
- *Secant Pile Wall* – An impermeable wall is creating by boring interlocking piles into the ground.
- *Diaphragm Wall* – The wall is cast sequentially in an excavated trench, often reinforced using rebar cages. The obtained result is an impermeable wall with some permeable joint in between the panels.

To further reinforce the retaining wall can a support system be installed, three common solutions are presented below in Figure 2.1.

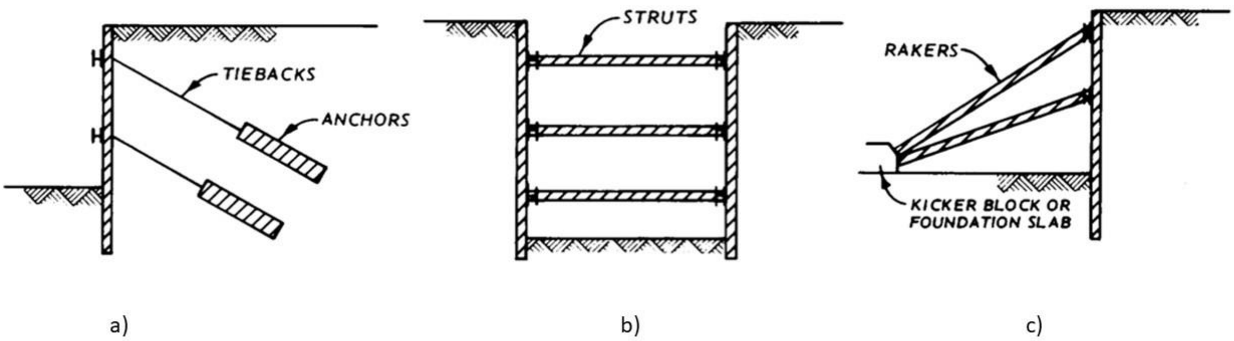

Figure 2.1. Support systems for retaining walls, a) Anchors b) Struts and c) Rakers. From (Guyer, 2013)

When constructing a deep excavation, a common approach is the Top/down method. This method entails a construction sequence, staring with installing the retaining walls, followed by excavating. If any reinforcement is needed, such as struts, anchors or braces, these are installed during the excavation phase. When the soil has been excavated down to a predefined depth the reinforcement is installed before continuing the excavation. According to Moorman (2004) this construction method does produce relatively small displacements.

2.4.1 Secant Pile Wall

A secant pile wall consists of impermeable bored piles, forming a wall by overlapping. This technical solution is favourable when wanting to construct a watertight retention wall, for a short period of time. The walls are constructed by first installing unreinforced pillars, called primary piles. When desired strength is attained are the secondary pillars constructed. By reinforcing the secondary pillars and partially boring these through the primary piles are an impermeable, stable wall created (Skanska Cementation, 2009). A schematic overview of how the wall is formed is presented in Figure 2.2.

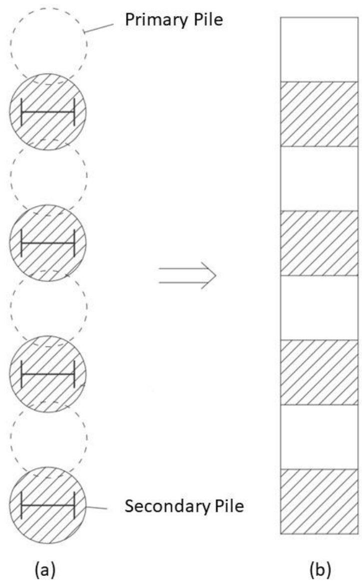

Figure 2.2. Schematic overview of Secant Pile Wall, a) as constructed b) as modelled in Plaxis 3D. Modelled from Bryson & Zapata-Medina (2010).

When modelling this type of wall in Plaxis 2D, firstly the way the wall is modelled must be decided. Bryson & Zapata-Medina (2010) propose a method creating the effect of constructing the overlapping piles by modelling the piles as rectangular instead of circular, again see Figure 2.2. However, for a 2D analysis can one type of material parameter be used, thus the properties of reinforced and unreinforced concrete must be smeared. Secondly, one must keep in mind how the groundwater will affect the calculation. The wall itself is rather impervious, but groundwater can during the dewatering and construction seep into the excavation through any permeable layer beneath the wall. The seepage will reduce the effective stress on the passive side of the walls, while increasing the effective overburden pressure on the active side. Extensive seepage can thus cause boiling or piping (Tjie-Liong, 2014). To avoid seepage a grout curtain can be injected underneath, prolonging the way the groundwater flows or sealing gaps completely between wall and bedrock or impermeable layer. When modelling a grout curtain all soil parameter can be kept constant, only the permeability is reduced (Chan, 2005).

2.5 Soil Stabilization

When coming across a soil with unwanted properties, four different options are available to ensure a safe construction. One can accept the soil and its insufficient properties and adapt the design to the conditions. Secondly, the inapt soil can be removed or bypassed. Finally, can the soil be treated to improve its properties. By treating the soil is the soil stabilized, and properties such as strength and durability (Sabry, 1977). Common stabilizing agents for clayey soils are lime, cement and a combination of the two. Depending on soil type and soil type are different strength increments reached, a summary produced by Holm et al (1995) can be seen in Table 2.3.

Table 2.3. Effect of stabilizing agent depending on soil type on strength (Holm, et al., 1995).

Stabilizer	*Soil Type*		
	Clayey Silt	**Silty Clay**	**Clay**
Lime	Slightly Increased	Increased	Slightly Increased
Lime – Cement	Increased	Increased	Increased
Cement	Significantly Increased	Significantly Increased	Increased

Further aspects impacting the result of a stabilization action are temperature, time for hardening, mixing method and the amount of stabilizing agent. To estimate and evaluate the result can laboratory testing or field investigation be performed (Holm, et al., 1995).

2.6 Plaxis 2D

Plaxis is a finite element software, developed at Deft University of Technology. The software is used to model and analyse deformations, stability and groundwater flow in geotechnical engineering (Brinkgreve & al, 2019). The following sections will investigate and present relevant aspects of Plaxis and modelling subsidence due to groundwater lowering.

2.6.1 Calculation Models

When modelling in Plaxis 2D either a Plane Strain or an Axisymmetric model is used. Both kind of models only have two degrees of freedom in each node, x-and y-direction. Using the Plane strain model is advisable when the cross section, including loading scheme and geometry, is uniform over a certain length. The Plane strain model estimates the cross section to be infinitely long, using the conditions for the cross section along the whole length. As the name of the model suggests the strains and deformations in the z-direction are assumed to be zero, creating a plane strain condition. The normal stresses in the z-direction are on the other hand considered (Plaxis, 2019a).

The Axisymmetric model on the other hand is preferred for circular models with uniform radial geometry. The given input is rotated around the symmetry line, creating a circle. Stresses and deformations are assumed to be the same in all radial directions. However, all forces are given as the force acting on the boundary of a circle subtending the angle of one radian. The force is thus given per radian, to calculate the corresponding force acting in the problem, each force should be multiplied with a factor of 2π (Plaxis, 2019a). The different calculation models are shown in Figure 2.3.

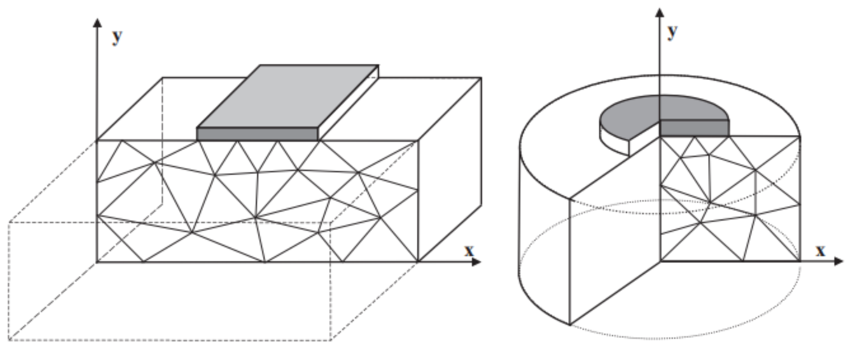

Figure 2.3. Calculation models used in Plaxis, Plane Strain model to the left and an Axisymmetric model to the right (Plaxis, 2019a).

2.6.2 Analysis Types

When using Plaxis must the analysis type of each phase be defined. The types can be divided into three categories; initial stress generation, groundwater flow analysis and deformation analysis. These are briefly described according to Plaxis (2019a) in Table 2.4.

Table 2.4. Plaxis analysis types.

Initial Stress Generation		
	K0 Procedure	The K0 procedure produces initial stresses in Plaxis by generating vertical effective stresses in equilibrium with the self-weight of the soil. K0 is the coefficient describing the ratio between horizontal and vertical stresses. The software can thus take the soil's loading history into account and produces horizontal stresses based on the K0 chosen. The procedure is primarily appropriate to use when all soil surfaces are horizontal.
	Gravity Loading	The initial stress state is with the gravity loading method produced based on the volumetric weight of the soil. K0 is in this method not an input but generated by the software dependant of Poisson's ratio.
Groundwater Flow Analysis		
	Groundwater Flow Only	As the name suggest, is this a calculation type for investigating only groundwater flow and performing calculations in saturated and unsaturated conditions.
Deformation analysis		
	Plastic	The plastic deformation analysis assumes the deformation to be elastic-plastic and do not consider any pore pressure change. A time interval can be given in the calculation, however are time effects not taken into account.
	Consolidation	The consolidation calculation takes, in addition to the elastic-plastic deformation, consolidation into account. The analysis allows for time dependent analysis of development and dissipation of excess pore pressures in saturated soft soil. The analysis can be performed with or without additional loading.
	Safety	The safety calculation type produces global safety factors. The calculation approach is simply to reduce the shear strength parameters friction angle (φ) and cohesion (c) of the soil and the tensile strength until failure occurs.
	Dynamic	The dynamic calculation approach is appropriate when one must consider stress waves and vibrations in the ground, i.e. dynamic loads, in the analysis.
	Dynamic with Consolidation	As the name implies this calculation type is advisable to use when in addition to dynamic load, consolidation needs to be considered. The applicable soil conditions are partially drained.
	Fully Coupled Flow-Deformation	The Fully coupled flow-deformation calculation type is used when the aim of the analysis is to study deformations and changes in pore pressures, due to time-dependant changes of the hydraulic boundaries, simultaneously. The soil conditions must be saturated or partially saturated.

2.6.3 Choosing Constitutive Model

When modelling the soil or rock behaviour there are various constitutive models to use, with varying accuracy. The simplest stress-strain relationship is Hooke's Law, where the soil behaviour is modelled as linear, isotropic elastic. Mohr-Coulomb's material model is often seen as a first order approximation, with its elastic perfectly plastic material behaviour. Plaxis can provide higher order approximations, these constitutive models provide a more realistic behaviour and therefore more accurate results (Plaxis, 2019b).

When deciding what model to use there are several aspects to consider, first of all what type of analysis will be performed. In geotechnical engineering there is a distinction between Ultimate Limit State (ULS) and Service Limit State (SLS), the analysis differ since either failure or deformations is of interest. An overview of the two analysis types can be seen in Table 2.5. Since the Mohr-Coulomb model does not allow changes in stiffness or differentiating between loading and re-loading stiffnesses, it is not an appropriate model for deformation analysis. More appropriate for service limit stet analysis are the more advanced constitutive models, such as Hardening Soil model or Plaxis Soft Soil model. However, when available information about the soil and its parameters is scarce can the Mohr-Coulomb model be preferable.

Table 2.5. Overview of geotechnical engineering ULS and SLS, modelled from Obrzud (2010).

Geotechnical Engineering Computations	
Limit State Analysis	*Service Limit State*
Analysis of:	Analysis of:
· Bearing capacity · Slope or wall stability	· Pile or retaining wall deflection · Supported deep excavations · Consolidation problems, including groundwater lowering
Typically used models:	Recommended models:
· Basic linear models, e.g. Mohr-Coulomb	· Advanced non-linear models, e.g. Hardening-Soil

A further aspect affecting the choice of model is what knowledge of the soil is available. It can be argued that the model parameters for the Plaxis Soft Soil model (SS model) are easier to derive than the Hardening Soil model (HS model). Furthermore, the wanted precision for the predictions affects the choice of model (Obrzud, 2010). In addition to these two basic aspects, the time span should be considered. If a long-term creep deformation is of interest then it is advisable to use a model which takes creep into account, e.g. Plaxis Soft-Soil Creep model. One last remark regarding modelling consolidation and groundwater lowering, for which both the Soft Soil model and the Hardening soil model can be used. The differences in the predictions made by the two models are, for many stress paths, minor even though the yield surfaces are modelled differently. Both models also have a user defined K_0^{NC}, meaning the parameter is given as input. However, according to Karstunen & Amavasai (2017) is the Soft-Soil preferable to use since there are no substantial benefits of using the Hardening soil model. The HS model requires additional input parameters, compared to the SS model, the model also prohibit input parameter combinations of stiffnesses that represents the Swedish soft soils.

Lastly, the soil type of interest is also of importance when choosing constitutive model. Soils suitable for Plaxis Soft model are, as the name suggests, near normally consolidated clays, clayey silts and peat (Plaxis, 2019b). Other models, like the Mohr-Coulomb or the Hardening Soil model, are more general and can be used for all soil types (Lade, 2005).

2.6.4 Constitutive Model – Linear Elastic

When the Linear Elastic model is applied is the Hooke's law of isotropic elasticity used. The model produces a very crude approximation of soil behaviour, however for stiff volumes, such as solid concrete walls or bedrock, can the model be applied (Plaxis, 2019b). To describe the material is only two parameters needed;

Young's Modulus – E
Poisson's Ratio – v

2.6.5 Constitutive Model – Mohr-Coulomb

The Mohr-Coulomb model describes the soil as a linear elastic, perfectly plastic material. By combining Hooke's law, for the isotropic elastic behaviour, and the Mohr-Coulomb failure criteria, describing the perfectly plastic behaviour, the model is created. Since the model is perfectly plastic the yield function is fixed, governed by the model parameters and not influenced by plastic straining. The full yield condition contains six yield functions, as seen in Equation 1a to f. When fixing the yield functions is a hexagonal cone created in the stress space (Plaxis, 2019b).

$$f_{1a} = \frac{1}{2}(\sigma_2' - \sigma_3') + \frac{1}{2}(\sigma_2' + \sigma_3') - c \cdot \cos(\varphi) \qquad \text{(Eq. 1 a)}$$

$$f_{1b} = \frac{1}{2}(\sigma_3' - \sigma_2') + \frac{1}{2}(\sigma_3' + \sigma_2') - c \cdot \cos(\varphi) \qquad \text{(Eq. 1 b)}$$

$$f_{2a} = \frac{1}{2}(\sigma_3' - \sigma_1') + \frac{1}{2}(\sigma_3' + \sigma_1') - c \cdot \cos(\varphi) \qquad \text{(Eq. 1 c)}$$

$$f_{2b} = \frac{1}{2}(\sigma_1' - \sigma_3') + \frac{1}{2}(\sigma_1' + \sigma_3') - c \cdot \cos(\varphi) \qquad \text{(Eq. 1 d)}$$

$$f_{3a} = \frac{1}{2}(\sigma_1' - \sigma_2') + \frac{1}{2}(\sigma_1' + \sigma_2') - c \cdot \cos(\varphi) \qquad \text{(Eq. 1 e)}$$

$$f_{3b} = \frac{1}{2}(\sigma_2' - \sigma_1') + \frac{1}{2}(\sigma_2' + \sigma_1') - c \cdot \cos(\varphi) \qquad \text{(Eq. 1 f)}$$

Finally, to use the Mohr-Coulomb model five basic parameters are required, these are presented below (Plaxis, 2019b);

Young's Modulus – E
Poisson's Ratio – v
Cohesion – c
Friction Angle – φ
Dilatancy Angle – ψ

2.6.6 Constitutive Model – Soft Soil Model

The Plaxis Soft Soil model is based on the Cam-clay model, with certain modifications. The basic outline of the Soft Soil model is a failure behaviour according to the Mohr-Coulomb criterion with stress dependent stiffnesses. The states of stresses and strains are isotropic. The yield function, describing the plastic volumetric strain in primary compression, is defined as an ellipsoidal cap. To describe the cap has Equation 2 been formulated. The height of the ellipse is determined by the parameter M, whereas the extension of the ellipse along the x-axis is decided by the pre-consolidation pressure p_0. However, for normally consolidated soil in plastic triaxial shearing the yield surface expands until the failure criterion is reached. The relationships are shown in Figure 2.4, with mean effective stress on the x-axis and deviatoric stress on the y-axis. No stress states above the failure line are tolerable, therefore the model cannot represent heavily over-consolidated clays or softening. The model is thus very sensitive to OCR values (Karstunen & Amavasai, 2017).

$$f_c = \frac{q^2}{(M^*)^2} + p'(p' - p'_0) \qquad \text{(Eq. 2)}$$

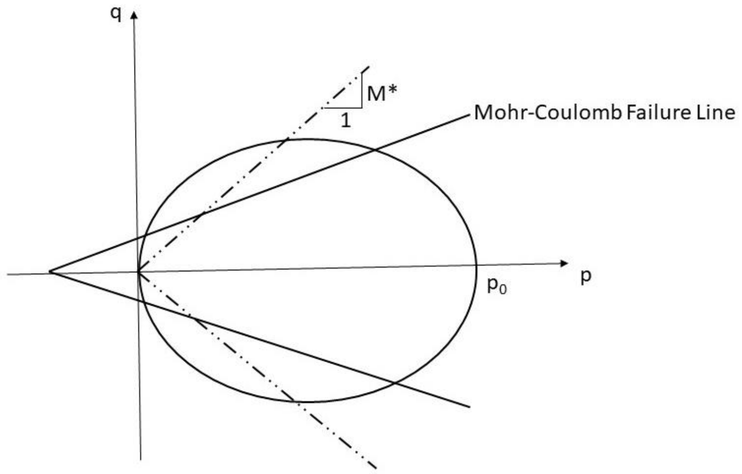

Figure 2.4. Yield function of Plaxis Soft Soil model based on Plaxis (2019b) and Karstunen & Amarvasai (2017).

Additional features are the model's ability to distinct between primary loading and unloading-reloading and a memory for pre-consolidation stress.

To perform an analysis using the Plaxis Soft Soil certain specific parameters are required. To determine these required parameters must field tests and laboratory testing results be examined and correlations be used. Based on what tests have been performed can different correlations be used to produce the required parameters. Parameters needed for Plaxis Soft Soil model are now listed, along with the correlations used for producing said parameter.

*Modified Compression Index – λ**

The modified compression index be determined graphically from the logarithmic relation between stress and strain, according to Figure 2.5a). To produce such graph in Sweden typically a CRS test is performed. However, when using the Swedish standard CRS testing is another graph produced, depicted in Figure 2.5b). The same correlation can be used, since the compression in a CRS test is one-dimensional and therefore is equivalent to the volumetric strain. The parameters can also be produced from an Incremental Loaded Oedometer test.

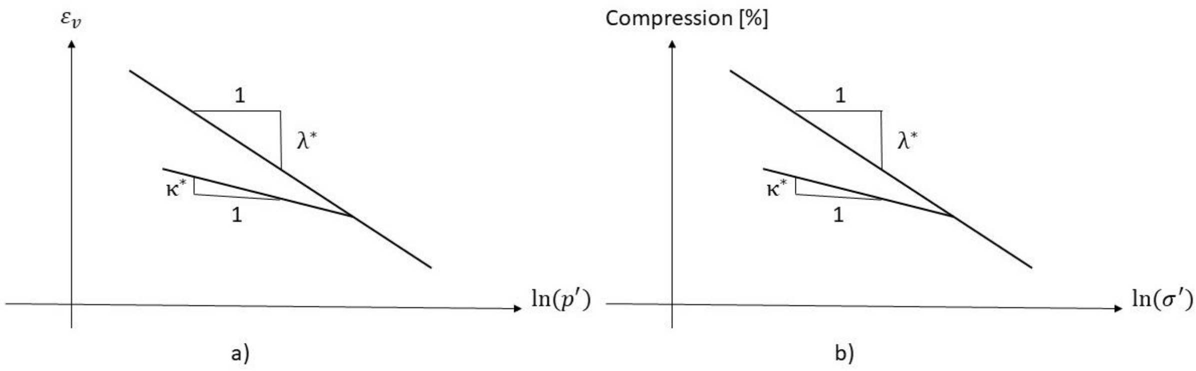

Figure 2.5. Logarithmic relationship between a) volumetric stress and mean strain from Plaxis (2019b) and b) compression and effective stress.

Modified swelling index – κ*
Finally, can the modified swelling index be determined by using a similar approach as the previous parameter, i.e. graphically determine the index using the correlation seen in Figure 2.5.

Friction angle – φ
Estimating the friction angle can be carried out using the triaxial result and Mohr-Coulombs failure criterion. By plotting the results from the triaxial tests in a $\tau_m - \sigma_m$ plot can the friction angle de derived by using Equation 3. The equation can be solved by graphically determine τ_m, σ_m and $c \cdot \cos(\varphi)$ according to Figure 2.6 and then using said parameters as input.

$$\tau_m = \sigma_m \cdot \sin(\varphi) + c \cdot \cos(\varphi) \qquad \text{(Eq. 3)}$$

- $\tau_m = \frac{\sigma_1 - \sigma_3}{2}$
- $\sigma_m = \frac{\sigma_1 + \sigma_3}{2}$
- φ – Friction angle
- c – Cohesion

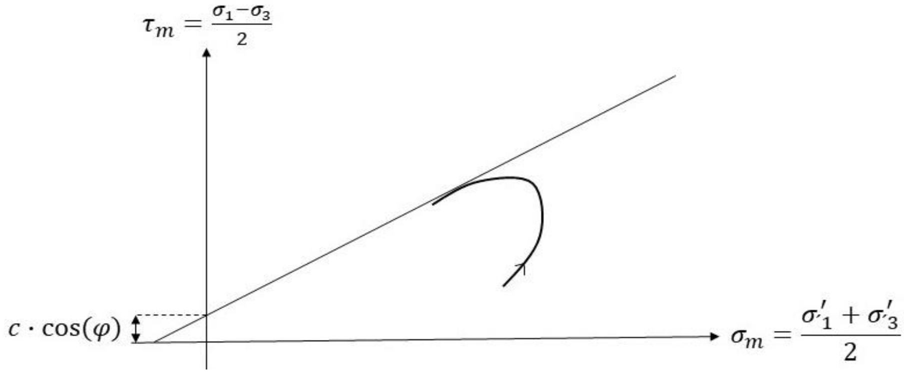

Figure 2.6. Graphical determination of friction angle and cohesion based on triaxial testing.

Cohesion – c
Cohesion can be estimated from undrained triaxial tests. By using the plot represented in Figure 2.6 can the cohesion be graphically determined when the friction angle is known.

Dilatancy angle – ψ
For most soft soils is the dilatancy not considered, dilatancy angle equal to zero is a standard setting (Karstunen & Amavasai, 2017).

Poisson's ratio for unloading/reloading – ν_{ur}
When determining Poisson's ratio for unloading/reloading for soft soils is typically between 0.1 and 0.2 (Karstunen & Amavasai, 2017).

Coefficient of lateral stress in normal consolidation – K_0^{NC}
The coefficient of lateral stress is typically determined according to Jaky's formula, Equation 4. When using the setting of Automatic determination in Plaxis is this the formula the software uses.

$$K_0^{NC} = 1 - \sin(\varphi') \qquad \text{(Eq. 4)}$$

Coefficient of lateral stress
However, if the vertical and horizontal stresses are known can the definition of K_0 be used for determination, according to Equation 5.

$$K_0 = \frac{\sigma_h'}{\sigma_v'} \qquad \text{(Eq. 5)}$$

K_0^{NC}*-parameter* – M
Automatically determined in Plaxis when implementing the K_0^{NC}-values.

3 Characterization of Model Excavation

The West Link project is, as previously stated, one of the largest infrastructural projects in Gothenburg. The planned tunnel route begins east of the Gothenburg Central Station, following a U-shape, and ending southeast of Korsvägen reconnecting with the existing railway. The locations of the underground stations will be underneath Gothenburg Central Station, in Haga and at Korsvägen. The exact route of the tunnel along with the locations of the stations is presented in Figure 3.1. The whole project is estimated to cost 20 billion SEK (2009 price level) and have a construction time of eight years. The construction started in 2018 and the railway tunnel is to be in service 2026 (Trafikverket, 2016).

Figure 3.1. Overview of the West Link Project, the Korsvägen contract area is marked in yellow (Trafikverket, 2017).

Construction works will be extensive in the area around Korsvägen, an underground station is to be built along with connecting railway and service tunnels. The underground station will be built partly in soft soil and partly in rock. The middle section of the future station will be constructed in soft soil. This part of the station will be constructed with the cut- and cover method, which requires deep excavations. The idealized excavation for investigating subsidence in an urban environment will be modelled from this excavation. To perform an analysis in Plaxis information regarding soil properties, excavation geometry, construction and the structural elements is required, these aspects are presented in the following sections.

3.1 Material Properties

When constructing a deep excavation, knowledge of the soil is vital, field investigation and laboratory testing is therefore also vital to gain this knowledge. Extensive testing has been made in the area; the tests performed are tabulated below in Table 3.1.

Table 3.1. Tabulated field investigation and Laboratory Testing, along with investigation depth.

Field Investigation	
Investigation Method	*Investigation Depth [m]*
Cone Penetration Tests (CPT)	-3 to -21
Seismic Dilatometer Tests (SDMT)	0 to -22
Slug Test	-1.5, -10, -18 and -21
Test Pumping	-
Standpipes	0 to -24
Laboratory Testing	
Investigation Method	*Investigation Depth [m]*
Triaxial Test – Undrained, active	-4, -6, -8 and -13
Oedometer Test – CRS	-4, -6, -8, -12 and -15
Direct Shear Test – Undrained, consolidated	-4, -8, -12 and -15
Sieving	-13, -14 and -15

The following section will present the result of the investigations, i.e. soil conditions and hydrogeological properties. Lastly will the soil parameters required by Plaxis be presented, along with the sources of data.

3.1.1 Soil Conditions

Korsvägen is located in a valley formed as part of a fault zone. The bedrock is overlaid by a stiff friction layer, it is somewhat complicated to distinguish between the rock and the stiff friction layer. Therefore, the bottom layer has no strict limit. The soil layering above the friction layer is quite horizontal, with a clay layer approximately 15 to 20 metres thick. In the middle of the clay layer there is a permeable layer of friction material, mainly sand and silt fractions, which is about 1 to 2 metres thick. A schematic overview of the soil conditions can be seen in Figure 3.2.

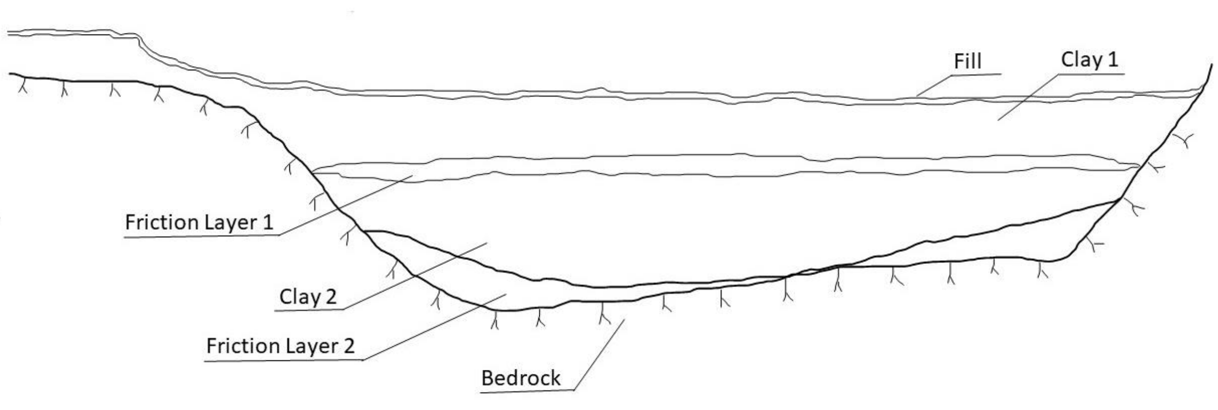

Figure 3.2. Graphic stratigraphy of soil conditions at Korsvägen, modelled from Sweco Civil AB (2014).

Generally, the clay is slightly over consolidated with a density increasing with depth. An approximate density is starting at 1.6 Mg/m^3 in the upper layer increasing to about 1.9 Mg/m^3. The natural water content on the other hand is decreasing over depth, in the upper layer is the water content is about 75 %, whereas in the lower layers it is estimated to be 30 %. A distinction is thus made between the clay above and below the friction layer, where the upper clay layer is more sensitive and softer, while the lower clay layer is stiffer. Furthermore, the clay in general is prone to settle and is sensitive to additional loads, such as groundwater lowering. On-going settlements are closely monitored using measuring stubs and estimated to be of the magnitude of 1 to 2 mm/year (Sweco Civil AB, 2014). The underlying bedrock is a granitoid metamorphic rock (SGU, n.d.).

3.1.2 Hydrogeology

The hydrogeological conditions in the area are complicated, due to the permeable layer in the middle of the clay layer. There are consequently two groundwater basins, one upper and one lower. The upper basin mainly consists of fill materials and generally has a low permeability. The lower basin on the other hand has a higher permeability, since it contains larger soil fractions such as silt and sand. When measuring groundwater conditions, hydrostatic conditions were found with a groundwater level approximately 1.5 meters below ground surface (Sweco Civil AB, 2014). It can also be seen from the slug test the hydraulic conductivity is significantly higher in both friction layers.

In order to investigate the influence radius a test pumping was conducted in the lower basin. From the pumping was an influence radius of approximately 600 meters found, which proves how sensitive the system is to groundwater lowering actions. The influence and the groundwater lowering can be seen in Figure 3.3. The general direction of the natural groundwater flow is from south to north, through the valley. All changes, manmade and seasonal, are closely monitored to ensure a safe and effective groundwater management (Trafikverket, 2014a).

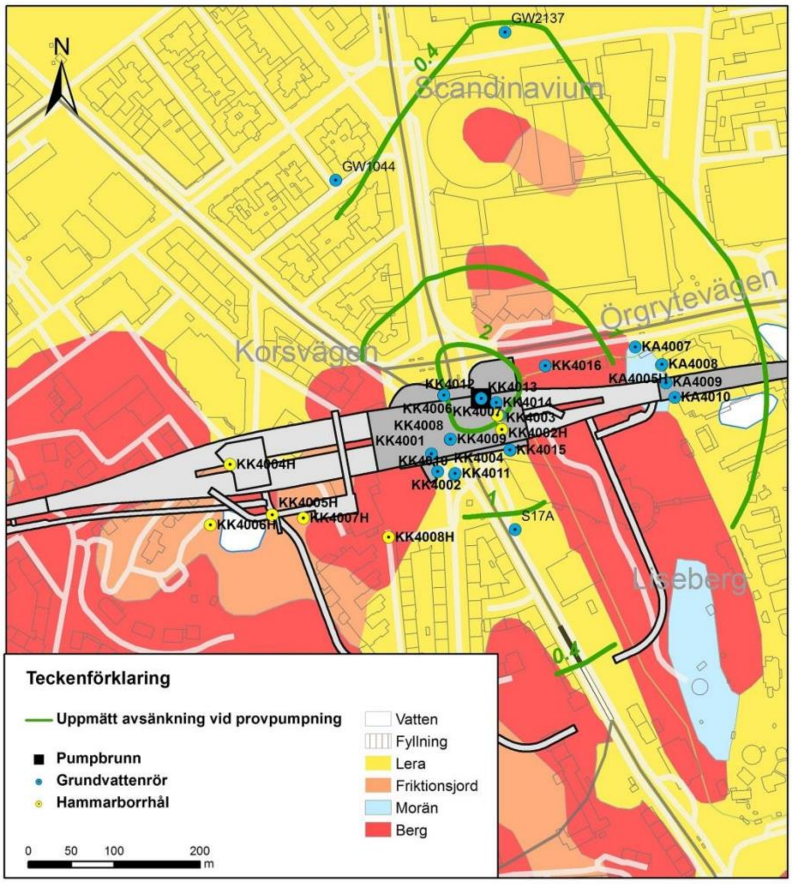

Figure 3.3. Influence radius of test pumping. Excavation in clay is marked with dark grey (Trafikverket, 2014a).

3.2 Construction

To be able to construct a tunnel and an underground station in soft soil, a deep excavation will be necessary. The design for the excavation and supporting measures are advanced, since the pit will be approximately 23 meters deep with complicated groundwater conditions. One proposed solution for the support is back-anchored secant pile walls. By using four rows of anchors, tied beck to bedrock, will no struts or cross-braces be needed. The exact cross-section design used is shown in Section 3.2.1. The secant pile wall will be installed first, followed by excavation. In order to avoid excavation of liquid materials the soil, which is to be excavated, will be stabilized using lime and cement. Groundwater management will be complicated since the groundwater conditions are deemed to be very sensitive to change. Where the excavation is in the proximity to bedrock the friction layer, in between wall and bedrock will be carefully injected to seal any gaps, creating an impermeable extension to the secant pile wall. To avoid creating a barrier for groundwater infiltration wells will be installed north and south of the excavation pit, leading the water past the pit (Trafikverket, 2014b). What dewatering method will be used is not stated, the method was therefore assumed in the model.

As previously mentioned, the Korsvägen station will be constructed in both soft soil and rock. The middle section of the future station will be constructed in clay, this section is located where the bus and tram station currently is situated. To ensure traffic and public transfer flows during construction the middle part will be constructed in two stages. First, will the east and west part of the section be excavated, and the concrete tunnel elements be built. The east part will be constructed, and the excavation refilled before the second stage is initiated, to enable relocation of the tram tracks and stations during the second stage of excavation. The second stage will be to excavate the middle part of the section. Construction works in the western part will continue during the second stage (Trafikverket, 2014b).

3.2.1 Excavation Geometry

The future station of Korsvägen will be constructed using the cut-and cover method, by first creating an excavation and subsequently constructing the concrete tunnel segment. The excavation geometry differs along the Korsvägen construction site, due to station design and soil conditions. To obtain a representative result the cross-section in the middle of the construction site was chosen, since here the clay depths are the greatest. In Figure 3.4 below is the future tunnel route presented, the chosen cross section, 461 + 040, for the model excavation is marked with a dashed line.

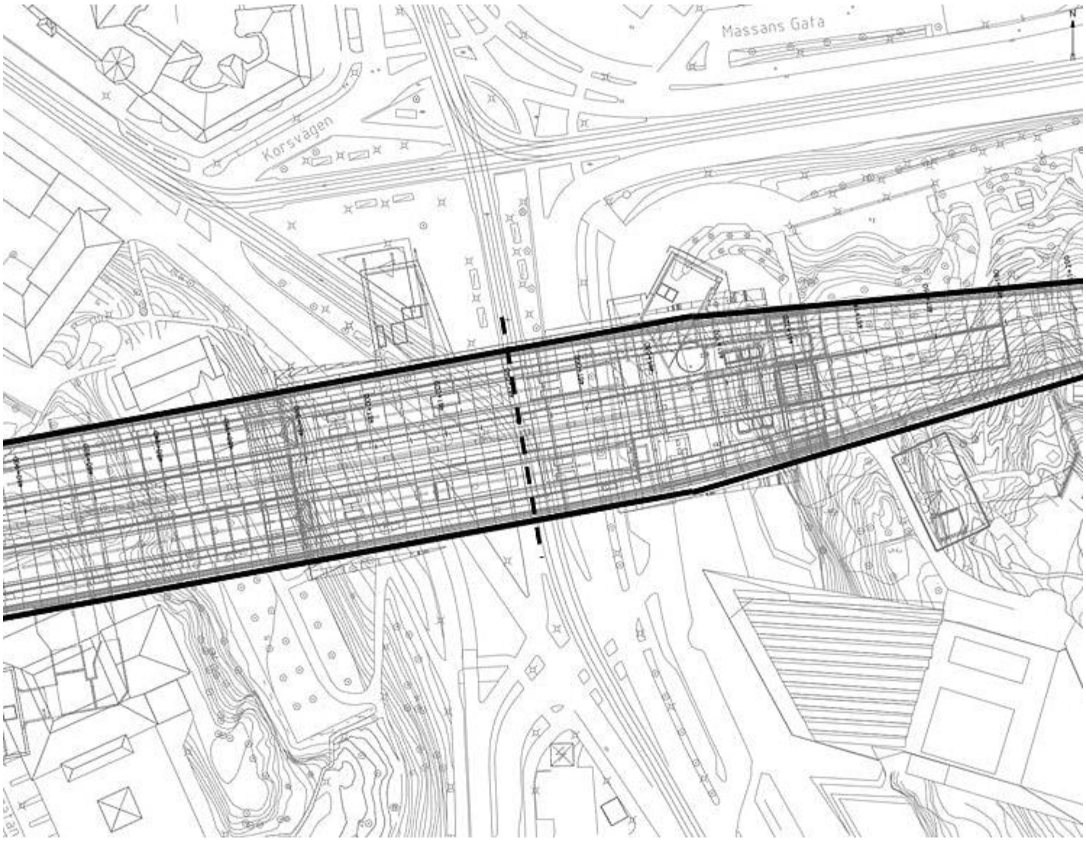

Figure 3.4. Map over planned tunnel route, the investigated cross section marked with dashed line.

A suggested design for the retaining wall is shown in Figure 3.5. Since the dimension was not given in the suggested design, therefore the diameter of the secant pile needed to be assumed. Based on data from a similar excavation was the diameter of the secant pile assumed to be 1000 mm (Powderham, 2000).

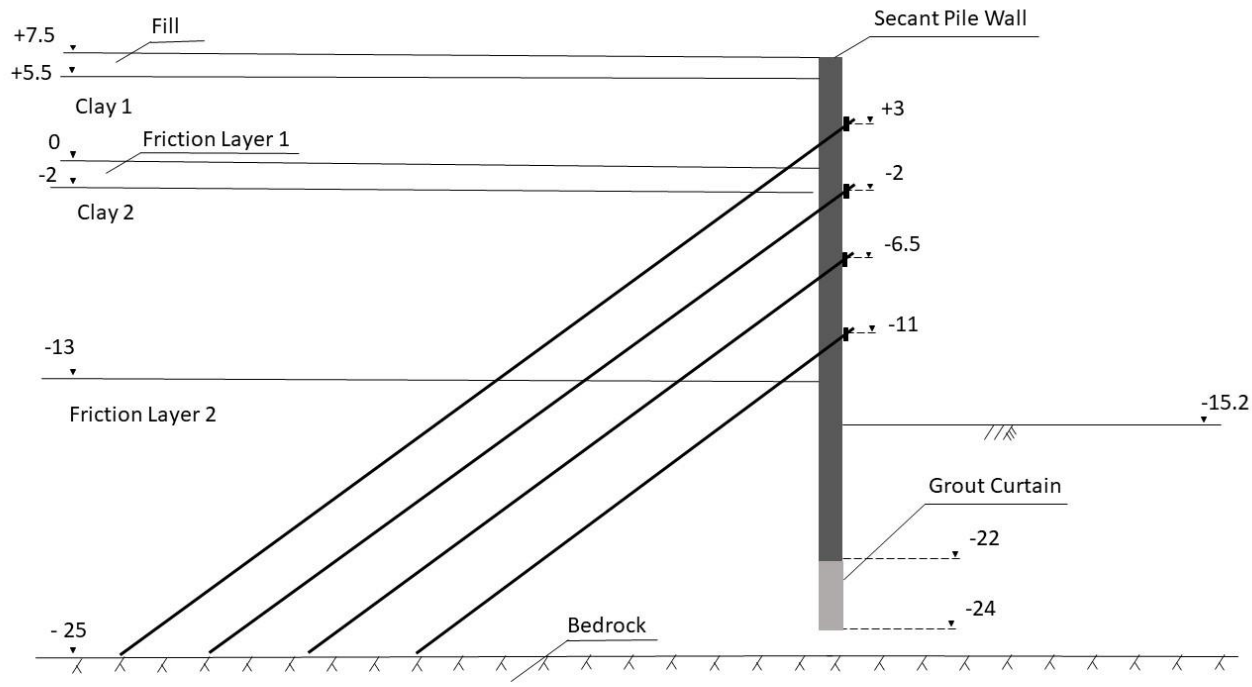

Figure 3.5. Proposed design of retaining wall for the excavation.

3.2.2 Monitoring

Due the importance of surrounding buildings and infrastructure and the sensitivity of the groundwater system is the groundwater levels closely monitored, along with settlements and wall movements. Groundwater measurements are made using standpipes and piezometers. Furthermore, the subsidence is monitored by installed ground stubs and extensometers (Sweco Civil AB, 2014).

4 Method

To analyse subsidence the finite software Plaxis was used. The approach used in analysis is in accordance to Burland's soil mechanics triangle (1987). To make the calculations and analyses in Plaxis possible to recreate a section also follows containing the settings and calculation phases used.

4.1 Soil Mechanics Triangle

A general methodology for soil mechanics related questions was proposed by Burland (1987). He forms with four key aspects the soil mechanics triangle. When forming the triangle are the three apexes represented by the ground profile, soil behaviour and applied mechanics. Empiricism is places in the middle of the triangle. All four aspects are presented below;

1) *The Ground Profile* – Obtained from site investigations.
2) *Soil Behaviour* – Produced from laboratory tests, in-situ tests and field tests.
3) *Applied Mechanics* – By idealising material behaviour can analyses be carried out.
4) *Empiricism* – Experience of material modelling.

By interlinking the four aspect together and expand each aspect can the soil mechanics triangle be produced, as seen in Figure 4.1.

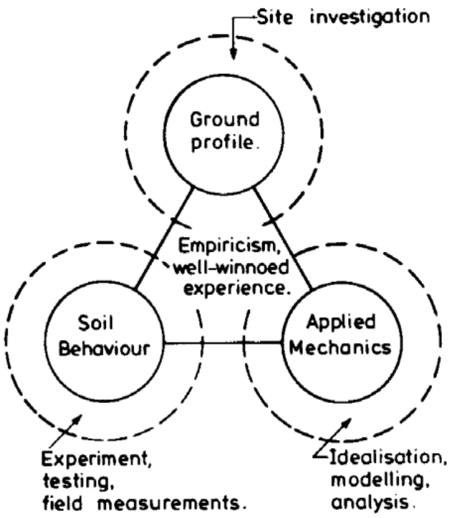

Figure 4.1. Soil Mechanics Triangle, from Burland (1987).

This methodology is applicable to all geotechnical problems, including the problem statement in this report. Producing the initial ground profile was based on site investigation, the profile was then modified when further laboratory data was received. Based on the soil propertied were their behaviour idealized, respectively, and each assigned a material model.

4.2 Plaxis Model

The finite element software Plaxis was used for analysing subsidence and how the groundwater lowering will affect the surrounding area. To enable analysis was a 2D model setup in Plaxis using model dimensions from early stage blueprints and soil data from field and laboratory investigation. The following chapter aims to present the model, including settings, dimensions, input data and the construction sequence used for calculation.

4.2.1 Model Setup

When setting up the model, only the soil conditions considered, no loads from buildings or foundations were considered. However, since sensitive foundation types can be found near the excavation was the model extended to investigate the influence radius and potential impact in the area where the wooden pile foundations are located.

Setting up the model, 15 node elements were used. Mesh and dimensions of the model can be seen below in Figure 4.2, the general mesh was set to medium with refinement surrounding the excavation. The ground surface is located at +7.5 meters, and the model bottom is located at –30 meters. Furthermore, the total length of the model is 105 meters.

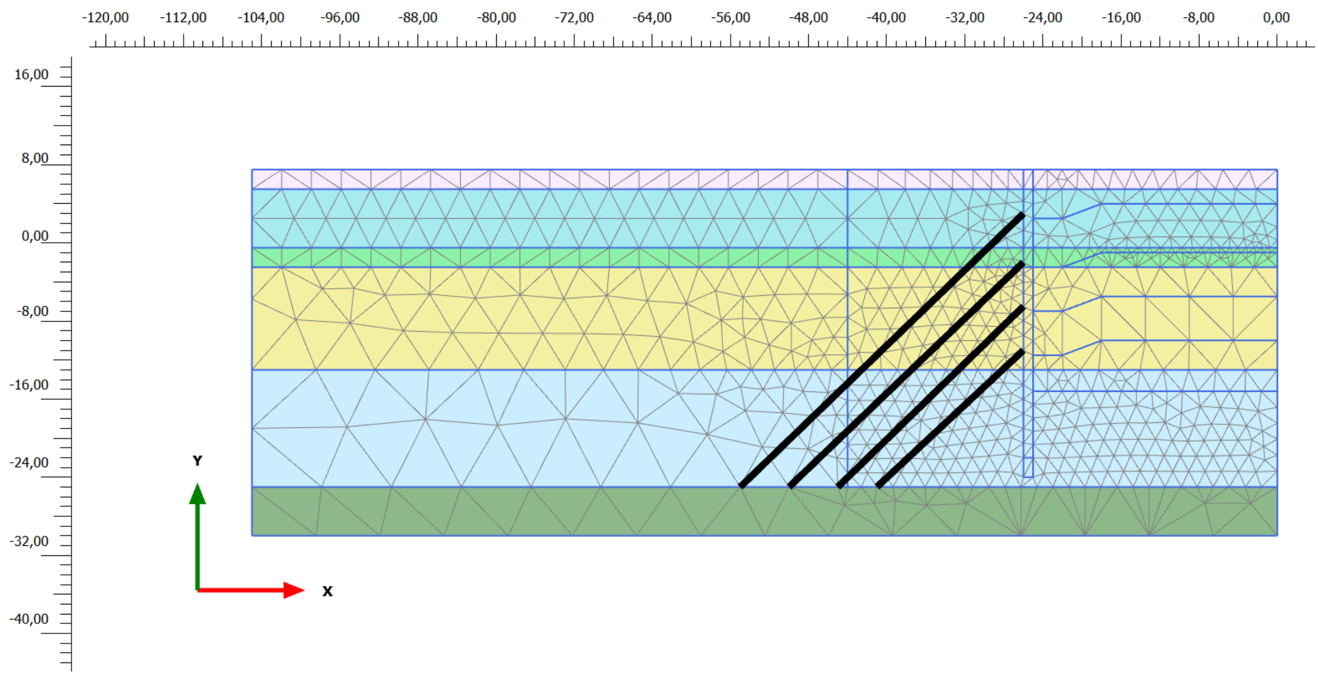

Figure 4.2. Connectivity plot generated in Plaxis 2D of mesh and model dimensions.

When modelling the consolidation analysis type was used, to ensure consideration of excess pore pressures dissipating and consolidation settlements due to this. To model the groundwater lowering the material inside the excavation was set to dry. The global water level was thus kept at the initial level of +6 m. The Groundwater Flow model conditions were set to the following;

- XMin: Open
- XMax: Closed
- YMin: Closed
- YMax: Open

4.2.2 Material Properties

Soil investigation has, as previously mentioned, been extensive in the area surrounding the future excavation. An initial determination of material properties was derived from laboratory data and field investigation. For some soil layers the available data was not sufficient. Input was then retrieved from literature or case studies and simpler material models were opted for. Summaries of the initial parameter determination are presented below in Table 4.1 to Table 4.9. Where CRS or Triaxial test is given as source, were the parameters determined by using the methodology presented in Section 2.6.6. The sources of soil data are presented along with the soil data, and a complete soil data can be found in Appendix A – Soil Properties.

Information about the fill was rather scarce, therefore was data retrieved both from site specific sources and a Case Study from a project in Gothenburg. The produced data is presented in Table 4.1.

Table 4.1. Initial parametric determination of Fill.

Fill				
Parameter		*Value*	*Unit*	*Source of Data*
-	**Material Model**	Mohr-Coulomb	[-]	-
γ_{sat}	**Saturated Unit Weight**	18	[kN/m^3]	Korsvägen Geotechnical PM (Sweco Civil AB, 2014)
γ_{unsat}	**Unsaturated Unit Weight**	11	[kN/m^3]	Korsvägen Geotechnical PM (Sweco Civil AB, 2014)
k	**Permeability**	1.9E-5	[m/s]	Slug Test (Engelbrektsson, 2016)
		1.9	[m/day]	
φ	**Friction angle**	38	[°]	Korsvägen Geotechnical PM (Sweco Civil AB, 2014)
ψ	**Dilatancy angle**	0	[°]	Case Study (De Bourgh & Jägryd, 2018)
E	**Young's Modulus**	10E3	[kPa]	Case Study (De Bourgh & Jägryd, 2018)
ν_{ur}	**Poisson's ratio for unloading/reloading**	0.30	[-]	Case Study (De Bourgh & Jägryd, 2018)
-	**Determination of K_0^{NC}**	Automatic	[-]	-
-	**Drainage Condition**	Undrained A	[-]	-

Investigation of the upper clay layer was quite extensive; several field and laboratory test have been carried out. Since no hydraulic testing was performed in the upper clay layer the permeability was retrieved from a case study carried out in Gothenburg clay. The initial determination of required parameter is shown below in Table 4.2.

Table 4.2. Initial parametric determination of Clay layer 1.

Clay 1				
Parameter		*Value*	*Unit*	*Source of Data*
-	**Material Model**	Plaxis Soft Soil	[-]	-
γ_{sat}	**Saturated Unit Weight**	16	[kN/m^3]	SDMT (COWI AB, 2018)
γ_{unsat}	**Unsaturated Unit Weight**	16	[kN/m^3]	SDMT (COWI AB, 2018)
k	**Permeability**	7E-10	[m/s]	CRS Test (Medin, 2016a)
		6E-5	[m/day]	
φ	**Friction angle**	27	[°]	Triaxial Test (Medin, 2016a)
ψ	**Dilatancy angle**	0	[°]	Assumed based on Karstunen & Amavasai (2017)
ν_{ur}	**Poisson's ratio for unloading/reloading**	0.15	[-]	Assumed based on Karstunen & Amavasai (2017)
λ*	**Modified compression index**	0.145*	[-]	CRS Test (Medin, 2016a)
κ*	**Modified swelling index**	0.0225*	[-]	CRS Test (Medin, 2016a)
c'	**Effective cohesion**	9	[kPa]	Triaxial Test (Medin, 2016a)
K_0	**Coefficient of lateral stress**	0.698	[-]	Triaxial Test (Medin, 2016a)
K_0^{NC}	**Coefficient of lateral stress for Normal Consolidated soil**	0.546	[-]	Triaxial Test (Medin, 2016a)
OCR	**Over Consolidation Ratio**	1.7	[-]	CPT (Medin, 2016b)
-	**Determination of K_0^{NC}**	Manual	[-]	-
-	**Drainage Condition**	Undrained A	[-]	-

*Calibrated according to Section 5.1.

Since the soil to excavated will be stabilized, each clay layer has a modified equivalent. The stabilizing agent consists of a combination of lime and cement. The main objective of the stabilizing action was to avoid excavating liquid material, therefore were only the strength parameters considered when modifying the soil. To simulate this strength increment, the cohesion and friction angle of the soil were increased, while all other parameters were kept the same. The cohesion was doubled, and friction angle increased with 10 degrees, parameters are given in Table 4.3.

Table 4.3. Assumed parameters for Stabilized Clay layer 1.

Clay 1 – Stabilized				
Parameter		*Value*	*Unit*	*Source of Data*
φ	**Friction angle**	37	[°]	Assumed
c'	**Effective cohesion**	19	[kPa]	Assumed

Friction Layer 1 consists of varying fractions of silt, sand and clay. In field and laboratory investigation some uncertainties regarding this layer were mentioned, the parameters presented below in Table 4.4 are therefore a generalization. To further investigate the impact of these uncertainties, will a parametric sensitivity study be carried out, focusing on this layer.

Table 4.4. Initial parametric determination of Friction layer 1.

Friction Layer 1				
Parameter		*Value*	*Unit*	*Source of Data*
-	**Material Model**	Plaxis Soft Soil	[-]	
γ_{sat}	**Saturated Unit Weight**	17	[kN/m^3]	SDMT (COWI AB, 2018)
γ_{unsat}	**Unsaturated Unit Weight**	17	[kN/m^3]	SDMT (COWI AB, 2018)
k	**Permeability**	3.2E-7	[m/s]	Slug Test (Engelbrektsson, 2016)
		2.76E-2	[m/day]	
φ	**Friction angle**	24	[°]	Triaxial Tests
ψ	**Dilatancy angle**	0	[°]	Assumed based on Karstunen & Amavasai (2017)
ν_{ur}	**Poisson's ratio for unloading/reloading**	0.15	[-]	Assumed based on Karstunen & Amavasai (2017)
λ*	**Modified compression index**	0.17*	[-]	CRS (Medin, 2016a)
κ*	**Modified swelling index**	0.021*	[-]	CRS (Medin, 2016a)
c'	**Effective cohesion**	11	[kPa]	Triaxial (Medin, 2016a)
K_0	**Coefficient of lateral stress**	0.706	[-]	Triaxial (Medin, 2016a)
K_0^{NC}	**Coefficient of lateral stress for Normal Consolidated soil**	0.593	[-]	Triaxial Test (Medin, 2016a)
OCR	**Over Consolidation Ratio**	1.2	[-]	CPT (Medin, 2016b)
-	**Determination of K_0^{NC}**	Manual	[-]	-
-	**Drainage Condition**	Undrained A	[-]	-

*Calibrated according to Section 5.1.

When determining the properties of the lower clay layer field and laboratory data was used, assumptions were only made regarding dilatancy angle and Poisson's ratio. Permeability data was retrieved from a case study of another project in Gothenburg clay. The initial parameters as presented in Table 4.5.

Table 4.5. Initial parametric determination of Clay layer 2.

Clay 2				
Parameter		*Value*	*Unit*	*Source of Data*
-	**Material Model**	Plaxis Soft Soil	[-]	
γ_{sat}	**Saturated Unit Weight**	18	[kN/m^3]	SDMT (COWI AB, 2018)
γ_{unsat}	**Unsaturated Unit Weight**	18	[kN/m^3]	SDMT (COWI AB, 2018)
k	**Permeability**	2.2E-10	[m/s]	CRS (Medin, 2016a)
		1.9E-5	[m/day]	
φ	**Friction angle**	30	[°]	Triaxial Tests
ψ	**Dilatancy angle**	0	[°]	Assumed based on Karstunen & Amavasai (2017)
ν_{ur}	**Poisson's ratio for unloading/reloading**	0.15	[-]	Assumed based on Karstunen & Amavasai (2017)
λ*	**Modified compression index**	0.0896*	[-]	CRS (Medin, 2016a)
κ*	**Modified swelling index**	0.0215*	[-]	CRS (Medin, 2016a)
c'	**Effective cohesion**	11.5	[kPa]	Triaxial (Medin, 2016a)
K_0	**Coefficient of lateral stress**	0.655	[-]	Triaxial (Medin, 2016a)
K_0^{NC}	**Coefficient of lateral stress for Normal Consolidated soil**	0.50	[-]	Triaxial Test (Medin, 2016a)
OCR	**Over Consolidation Ratio**	1.1	[-]	CPT (Medin, 2016b)
-	**Determination of K_0^{NC}**	Automatic	[-]	-
-	**Drainage Condition**	Undrained A	[-]	-

*Calibrated according to Section 5.1.

Similarly, as for the upper clay layer was the second clay layer also stabilized. The strength increment was done in accordance to clay layer 1, cohesion was doubled and friction angle increased with 10 degrees. Numerical values are presented in Table 4.6.

Table 4.6. Assumed parameters for Stabilized Clay layer 2.

Clay 2 – Stabilized				
Parameter		*Value*	*Unit*	*Source of Data*
φ	**Friction angle**	40	[°]	Assumed
c'	**Effective cohesion**	22	[kPa]	Assumed

Information about the friction layer overlying the bedrock was rather scarce. SDMT tests were indicating a sand layer, rather dense. Based on this information was data retrieved from literature, the properties of the sand are presented in Table 4.7.

Table 4.7. Initial parametric determination of Friction layer 2.

Friction Layer 2				
Parameter		*Value*	*Unit*	*Source of Data*
-	**Material Model**	Mohr-Coulomb	[-]	-
γ_{sat}	**Saturated Unit Weight**	19	[kN/m^3]	Korsvägen Geotechnical PM (Sweco Civil AB, 2014)
γ_{unsat}	**Unsaturated Unit Weight**	11	[kN/m^3]	Korsvägen Geotechnical PM (Sweco Civil AB, 2014)
k	**Permeability**	3E-5	[m/s]	Slug Test (Engelbrektsson, 2016)
		2.59	[m/day]	
φ	**Friction angle**	39	[°]	Korsvägen Geotechnical PM (Sweco Civil AB, 2014)
ψ	**Dilatancy angle**	5	[°]	Case Study (Plaxis, 2019b)
E	**Young's Modulus**	45E3	[kPa]	Case Study (Plaxis, 2019b)
ν_{ur}	**Poisson's ratio for unloading/reloading**	0.2	[-]	Case Study (Plaxis, 2019b)
-	**Determination of K_0^{NC}**	Automatic	[-]	-
-	**Drainage Condition**	Drained	[-]	-

To model the grout curtain the properties of the grouted soil were kept the same, only the permeability was reduced and all other model parameters were kept the same. The assumed permeability is shown in Table 4.8.

Table 4.8. Assumed properties of Grout curtain.

Grout Curtain				
Parameter		*Value*	*Unit*	*Source of Data*
k	**Permeability**	1.16E-10	[m/s]	Assumed
		1E-5	[m/day]	

The underlying bedrock was according to SGU (n.d.) a Granitoid Metamorphic rock, based on this knowledge parameters were taken from literature see Table 4.9, since no information was given about the bedrock. When choosing material model for the bedrock was this lack of information considered and the simplest material model, the Linear-Elastic, was chosen.

Table 4.9. Retrieved properties of Bedrock (Granitoid Metamorphic rock).

Bedrock – Granitoid Metamorphic Rock				
Parameter		*Value*	*Unit*	*Source of Data*
-	**Material Model**	Linear-Elastic	[-]	-
γ_{sat}	**Saturated Unit Weight**	27	[kN/m^3]	(Alm, Hakami, Ljunggren, Mattila, & Stephansson, 1985)
γ_{unsat}	**Unsaturated Unit Weight**	27	[kN/m^3]	(Alm, Hakami, Ljunggren, Mattila, & Stephansson, 1985)
k	**Permeability**		[m/s]	Assumed
		1E-3	[m/day]	
E	**Young's Modulus**	75E6	[kPa]	Case Study (Fredriksson & Lanaro, 2005)
ν_{ur}	**Poisson's ratio for unloading/reloading**	0.20	[-]	Case Study (Fredriksson & Lanaro, 2005)
-	**Determination of K_0^{NC}**	Automatic	[-]	-
-	**Drainage Condition**	Drained	[-]	-

The suggested excavation design is a back anchored secant pile walls. To avoid seepage a grout curtain will extend the secant pile wall, to bedrock where possible. Properties of such secant pile walls were presented by Bryson & Zapata-Medina (2010), where the properties of the rebars and concrete have been combined. The properties are presented in Table 4.10.

Table 4.10. Linear-elastic material properties for secant pile wall (Bryson & Zapata-Medina, 2010).

Secant Pile Wall			
Parameter		*Value*	*Unit*
-	**Type**	Non-porous	[-]
γ	**Unit weight**	24	[kN/m^3]
$k_x = k_y$	**Permeability**	0	[m/day]
E_g	**Elastic Stiffness**	8.9E6	[kN/m^2]
ν	**Poisson's Ratio**	0.2	[-]
R_{inter}	**Interface Strength**	1	[-]
d	**Diameter**	1	[m]

Finally, will the wall, as previously mentioned, be back anchored to bedrock. Since the anchors will be connected to bedrock can they be modelled as node to node anchors in Plaxis. The properties of such anchors are presented in Table 4.11.The Lspacing is derived from experience and given in Plaxis. As there was no knowledge of the planned pre-stresses, no pre-tension was assumed to keep the results conservative.

Table 4.11. Properties of node to node anchor in Plaxis (Plaxis, i.e).

Anchors			
Parameter		*Value*	*Unit*
-	**Type**	Elastic	[-]
EA	**Normal Stiffness**	2E6	[kN]
$L_{spacing}$	**Spacing out of plane**	1	[m]

4.2.3 Construction Sequences

When defining the construction sequence and complementary construction time, was the suggested design used, along with the Construction plan (Trafikverket, 2014b).The chosen sequence is given in Table 4.12 below.

Table 4.12. Calculation phases of Plaxis 2D analysis.

Analysis type	*Phase*	*Duration [days]*	*Time past [days]*
K0 - procedure	Initial Phase	-	-
Consolidation	Phase 1 – Installation of Secant Pile Wall	84	84
Consolidation	Phase 2 – Stabilizing Clay	14	98
Consolidation	Phase 3 – Setting Soil to dry inside Exacavation	7	105
Consolidation	Phase 4 – Excavating down to first anchor + activating anchor 1	21	126
Consolidation	Phase 5 – Excavating down to second anchor + activating anchor 2	21	147
Consolidation	Phase 5 – Excavating down to third anchor + activating anchor 3	21	168
Consolidation	Phase 7 – Excavating down to fourth anchor + activating anchor 4	21	189
Consolidation	Phase 8 – Final excavation	14	203
Consolidation	Phase 9 – Open excavation	365	**568**

4.2.4 Parametric Verification

To validate the derived parameters were the Plaxis Soil Test Tool used. By plotting the data from the soil testing tool and comparing it with the laboratory data could the produced parameters be reviewed. Further potential calibration was performed by using an iterative approach.

4.2.5 Numerical Analysis

To investigate the subsidence and analyse how the subsidence was affected by the uncertainties in the parametric determination a number of different simulations were run. All models are based on a Basic Model (1.A), where all parameters are according to Section 4.2.1, with calibration of some parameters according to Section 5.1. During the parameter determination some major areas of uncertainty were identified, these were the permeability of the upper and lower Friction layers. Also, the thickness of the Friction layer 1 is somewhat undefined, therefore the stratigraphy will be varied. To further investigate and understand how these uncertainties affect the model and the final result, several simulations were run, varying permeability and thickness of the upper Friction layer. Strength and stiffness properties of Friction layer 2 was also varied, since the material was rather undefined. Lastly, a simulation was performed using the Fully Coupled Flow-Deformation analysis type to investigate how the subsidence and groundwater flows are affected by the grout curtain. To also ensure convergence, a convergence test was performed, by running the Basic Model with a finer mesh. All run simulations are listed below in Table 4.13, a more detailed description of the simulations can be found in Table B.1 in Appendix B – Numerical Simulations.

Table 4.13. Model simulations run in Plaxis 2D.

Model Name	*Property Variated*	*Analysis Type*	*Comment*
Korsvägen 1	Basic Model	Consolidation	-
Korsvägen 2	Permeability	Consolidation	Investigating impact of permeability of Friction layers 1 and 2
Korsvägen 3	Stratigraphy	Consolidation	Investigating impact of thickness of Friction layer 1
Korsvägen 4	Strength and Stiffness	Consolidation	Investigating impact of strength and stiffness of Friction layer 2
Korsvägen 5	Grout Curtain	Fully Coupled Flow-Deformation	Investigating the impact of the Grout Curtain

The investigated aspects of the models are the settlements at the surface, the wall displacements and the excess pore pressures. Cross-sections were generated in order to evaluate the chosen aspects, the coordinates and investigated parameters are given in Table B.2 in Appendix B – Numerical Simulations. When choosing intervals of variance for each parameter were literature consulted, to investigate what values are reasonable to assume.

4.2.6 Sensitivity Analysis

Sensitivity analyses were performed to investigate what influenced the model. The model parameters investigated were the permeability, the Over Consolidation Ratio, the Modified Swelling index and the Modified Compression index. Each parameter was varied 10 %, except for the permeability, which was either decreased or increased times 10, i.e. one order of magnitude.

5 Result

The result of the report is divided into two parts, firstly the calibration of the parameters and secondly the results of the numerical analysis in Plaxis.

5.1 Parametric Determination

Following the initial parametric determination, several soil test simulations were run to calibrate and validate the chosen parameters. Simulations of CRS tests and Undrained Triaxial tests were performed at three separate depths, representing the three soil layers modelled with the Soft Soil model. To validate and potentially calibrate the modified compression and swelling indices the CRS test were modelled in the Plaxis Soil Test tool. After plotting the laboratory results and the initially determined parameters further calibration of the modified compression and swelling indices was deemed necessary. A summary of the results from the modelling and calibration can be seen in Table 5.1 and the corresponding graphs from the calibration are presented in Appendix C – Soil Test.

Table 5.1. A summary of calibrated soil properties, data from CRS test performed in laboratory and in Plaxis Soil Test tool.

		Initial Value	*Final Adaptation*	*Unit*
Clay 1				
λ^*	**Modified compression index**	0.143	0.145	[-]
κ^*	**Modified swelling index**	0.0115	0.0225	[-]
Friction Layer 1				
λ^*	**Modified compression index**	0.186	0.17	[-]
κ^*	**Modified swelling index**	0.0144	0.021	[-]
Clay 2				
λ^*	**Modified compression index**	0.0896	0.0896	[-]
κ^*	**Modified swelling index**	0.022	0.0215	[-]

When viewing the result can it also be seen that the Plaxis Soft Soil model simulates the soil behaviour reasonably well. It can also be seen that the Modified Swelling indices were greatly modified in the calibration. This could be due to lacking soil data, fewer measuring points yield a more uncertain initial determination. The available data for unloading were lacking, since the Triaxial and CRS tests only were performed for loading and unloading was not considered.

To produce valid values of friction angle and cohesion, additional calibrations using the Plaxis Soil Test tool was performed, using laboratory data from Triaxial tests. The results from the simulation of the soil behaviour of the soil layers are depicted in Appendix C – Soil Test, a summary of the calibrated parameter can be seen in Table 5.2.

Table 5.2. A summary of calibrated soil properties, data from laboratory of Undrained Triaxial test and in Plaxis Soil Test tool.

		Initial Value	*Final Adaptation*	*Unit*
Clay 1				
φ	**Friction Angle**	27	28	[°]
c	**Cohesion**	9	10	[kPa]
Friction Layer 1				
φ	**Friction Angle**	24	25	[°]
c	**Cohesion**	11	11	[kPa]
Clay 2				
φ	**Friction Angle**	30	30	[°]
c	**Cohesion**	11.5	11.5	[kPa]

In the result it can be seen that the initial values were not altered significantly. Furthermore, a general comment; a softening can be seen in the laboratory tests, this behaviour can however not be modelled using the Soft soil model. Softening is not allowed, as explained in Section 2.6.6, since no stress states above the Mohr-Coulomb failure line are tolerable. Only half of the curve can thus be modelled. Lastly, modelling the second clay layer proved difficult, as calibrating the model by changing the values of cohesion and friction angle did not significantly improve the modelled behaviour. However, the laboratory data shows an irregular behaviour, which could be due to unexpected soil content or cracks in the sample. The image of the sample also shows some disturbance in the sample, which could affect the result. The input parameters were thus not altered, since calibrating the model further did not yield any improvement.

5.2 Numerical Analysis

The following section consists of the results from numerical analyses carried out in Plaxis 2D, along with sensitivity analyses.

5.2.1 Convergence Test

To ensure the analysis converges, a convergence test was carried out, by running the same model using different mesh sizes, first was the model run with medium sized mesh as described in Section 4.2.1 and thereafter with mesh size *Fine.* A comparison of the two simulations can be seen in for settlement trough and wall displacements in Appendix D – Convergence Test. The mesh size was seen to yield an insignificant difference in result, therefore will the analysis be carried out using medium sized mesh to save computational time.

5.2.2 Varying Permeability of Friction layer 1

To investigate the impact of the intermittent friction layer, as number of simulations were run, and the results are presented in the following section.

Figure 5.1 presents the surface settlements for the final phase, i.e. after 1 year of open excavation, when varying the permeability of the upper friction layer. Four different values were used in the analysis and all curves are plotted in the graph. When viewing the result can it be seen that the permeability influences the settlement trough. However, the shape and magnitude of the settlement is rather constant.

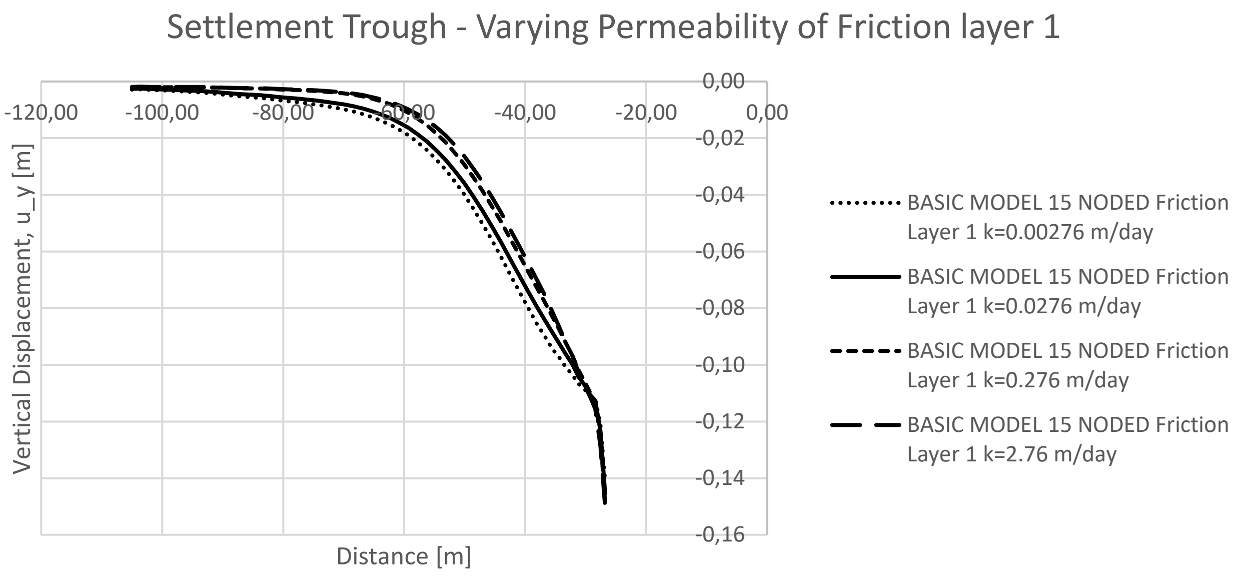

Figure 5.1. Settlement trough based on varying permeability of Friction layer 1.

Further investigation was carried out, in Figure 5.2 can the horizontal wall displacement be seen for Phase 9. The result shows that the varying permeability mainly affect the displacement close to the surface.

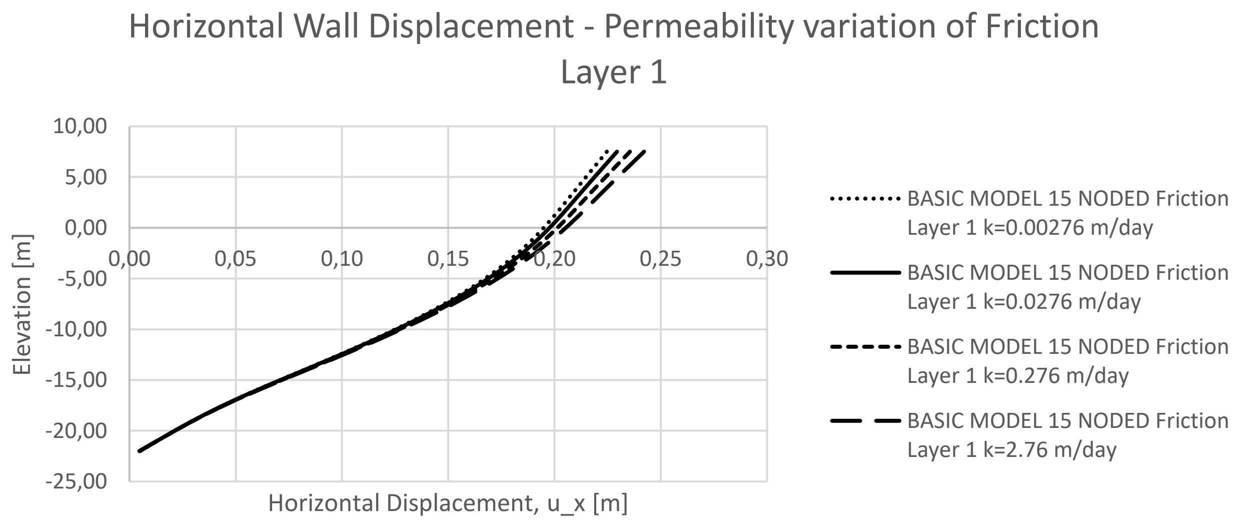

Figure 5.2. Horizontal wall displacement, varying the permeability of Friction layer 1.

Moreover, the heave of the excavation bottom was analysed. The displacement curves for all four permeabilities used were similar, which can be seen in Figure E.1 in Appendix E – Numerical Results.

When increasing the permeability higher dissipation rates can be seen for Friction layer 1. In Friction layer 1 also lower peak excess pore pressures are recorded. On the contrary, when decreasing the permeability lower dissipation rates are observed and higher peak excess pore pressures are noted for Friction layer 1. In Clay layer 1 and 2 are no significant differences seen, the curves for the four permeabilities are very similar. However, some small change can be seen when reviewing the dissipation rates. No excess pore pressures are recorded in the lowest Friction layer. The result is shown in Appendix E.1 Varying Permeability of Friction Layer 1.

Furthermore, a cross section was investigated to analyse the distribution of pore pressures. The results from three of the nine phases are presented below in Figure 5.3, Figure 5.4 and Figure 5.5 respectively. These three phases were chosen, since they represent the peak excess pore pressures and the pore pressures residual after 1 year of open excavation.

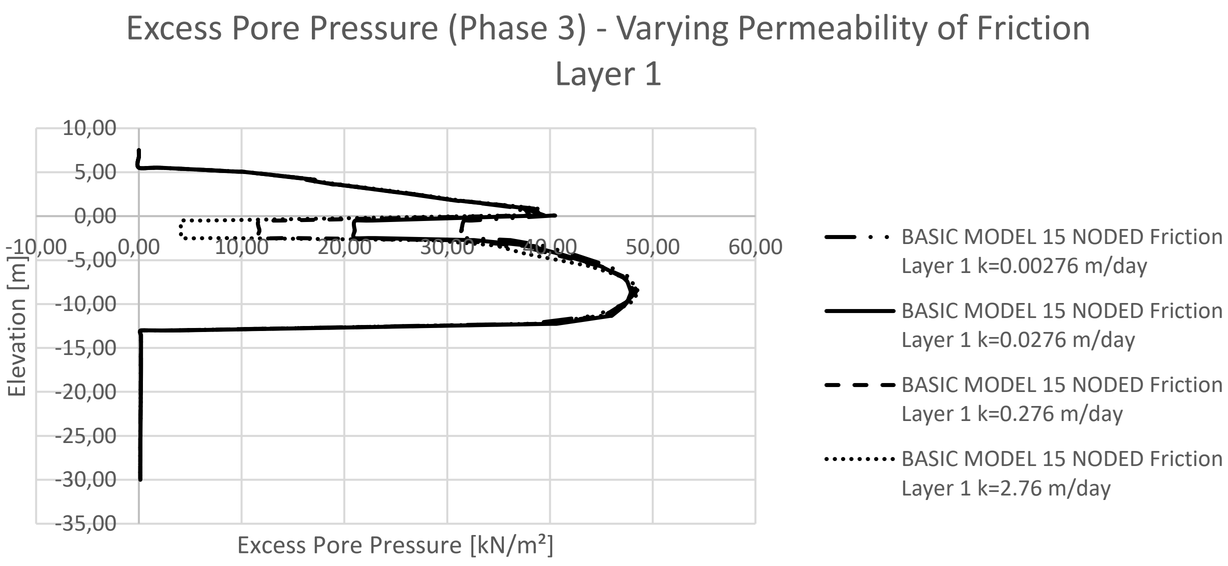

Figure 5.3. Excess pore pressure of cross section close to the wall, Phase 3 – Setting Soil to dry inside Excavation.

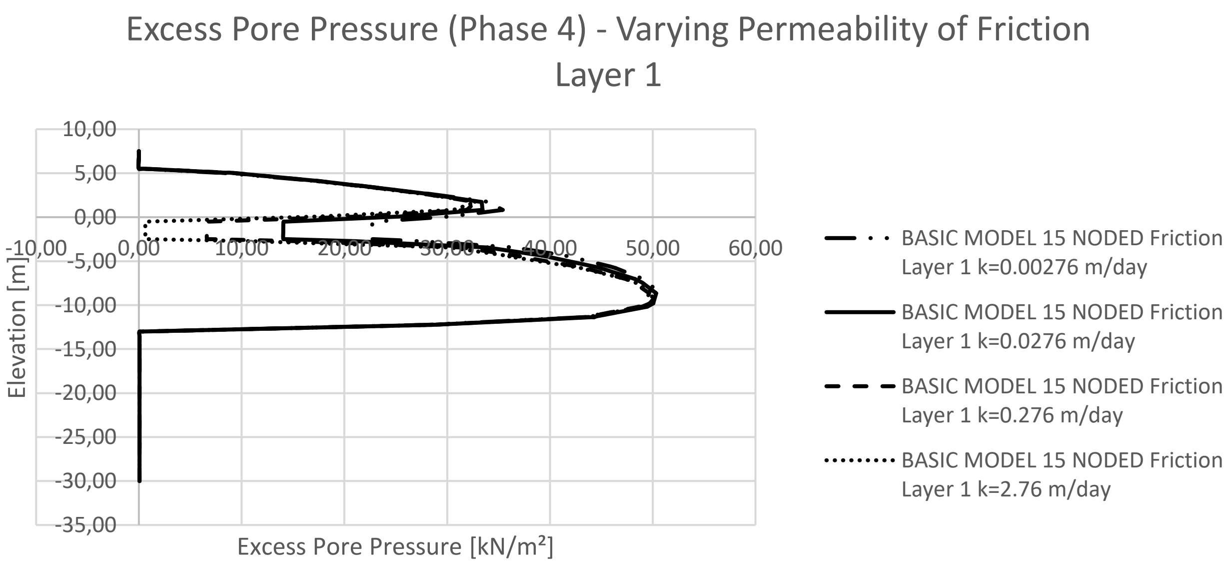

Figure 5.4. Excess pore pressure of cross section close to the wall, Phase 4 - Wall Installation.

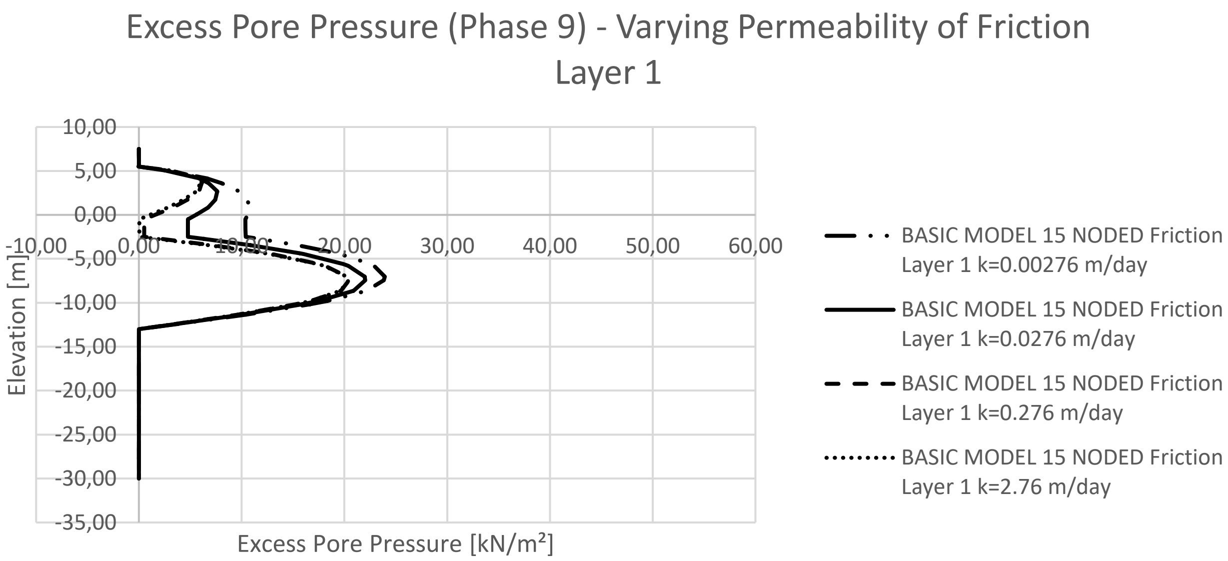

Figure 5.5. Excess pore pressure of cross section close to the wall, Phase 9 - Open Excavation.

When reviewing the excess pore pressure distribution over depth, a very symmetric behaviour can be seen in all phases. The differing factor is the excess pore pressure in the Friction layer 1, when decreasing the permeability are greater excess pore pressures built up in Friction layer 1. Subsequently, when increasing the permeability lower pore pressures are observed. In phase 3 and 4 the excess pore pressure in Clay layer 1 and 2 are practically identical. However, in the final phase a larger portion of the excess pore pressures have dissipated for increasing permeabilities, especially in Clay layer 1.

5.2.3 Varying Permeability of Friction layer 2

Secondly, the permeability of the lower friction layer was varied and analysed using three values of permeability. The result from modelling the surface settlements can be seen for Phase 9 below in Figure 5.6. When evaluating the result, it can be noticed that increasing the permeability yields a very small difference. Furthermore, decreasing the permeability slightly influences the magnitude of settlements. Close to the wall can no significant difference be seen, but when moving further away from the wall an increasing difference is recorded.

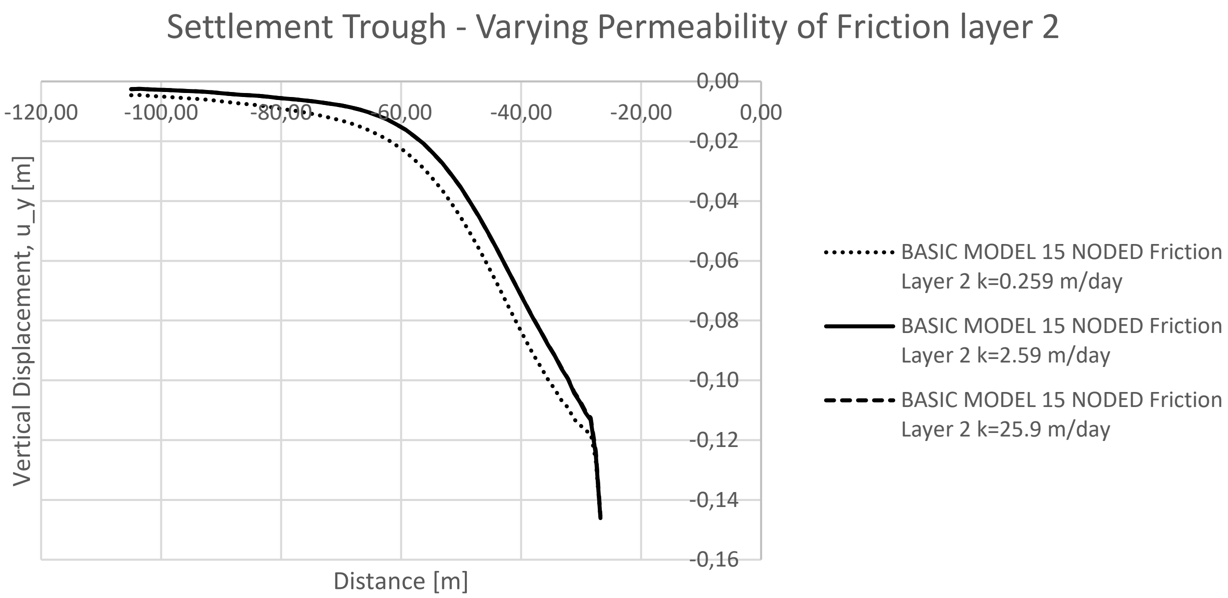

Figure 5.6.Settlement trough based on varying permeability of Friction layer 2.

Additionally, the horizontal wall displacements were plotted, for all three permeabilities, the results are plotted for the final Phase in Figure 5.7. Varying the permeability for the lower friction layer does not influence the horizontal displacement in a noteworthy way, the displacements curves and the extreme values are practically identical.

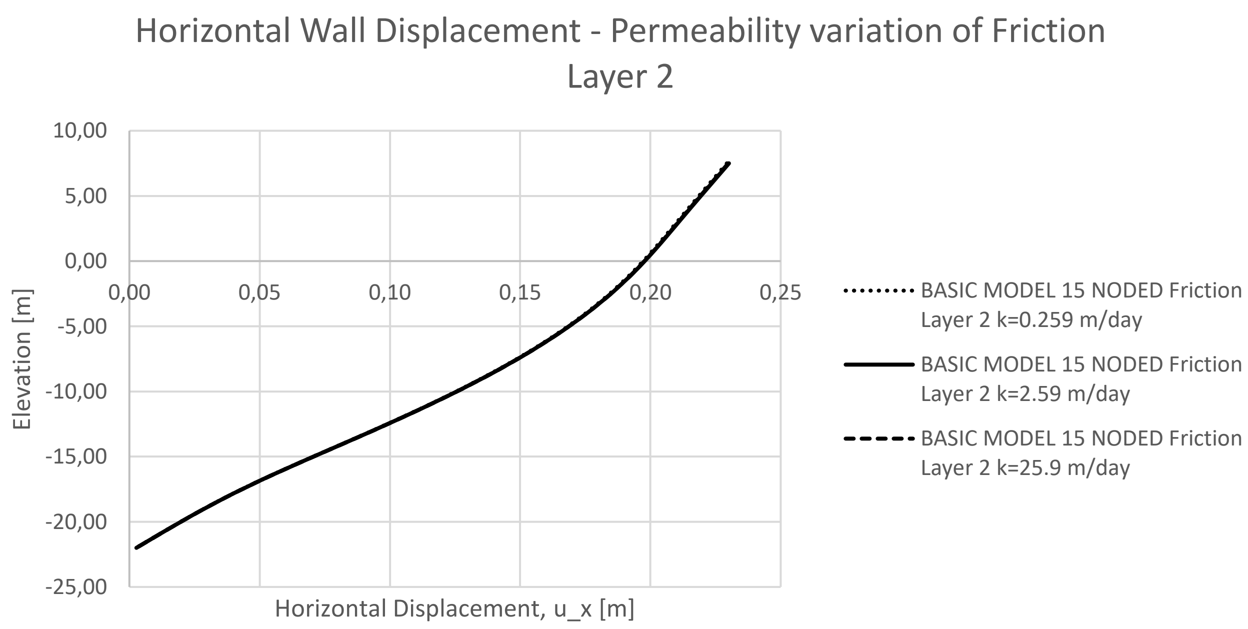

Figure 5.7. Horizontal displacement of secant pile wall plotted with variated permeabilities.

No impact on the heave of the excavation bottom was recorded when varying the permeabilities.

Varying the permeability of Friction layer 2 yield no result when plotting the excess pore pressures over time. A very small peak can be seen in Friction layer 2 when decreasing the permeability, but apart from that is the results very homogeneous. Lastly, it can be concluded that varying the permeability of Friction layer 2 yield little result for the excess pore pressure distribution. In Phase 3 a slight increase in excess pore pressures can be seen for Friction layer 2 when decreasing the permeability. When reviewing the other layers no significant change can be seen. The plots are shown in Appendix E.2 Varying Permeability of Friction Layer 2

5.2.4 Varying Thickness of Friction layer 1

The thickness of the upper friction layer was also varied, the settlement trough is for Phase 9 plotted in Figure 5.8. A noteworthy change in settlements were obtained when decreasing the thickness of the friction layer, significantly higher settlement values were then recorded close to the secant pile wall.

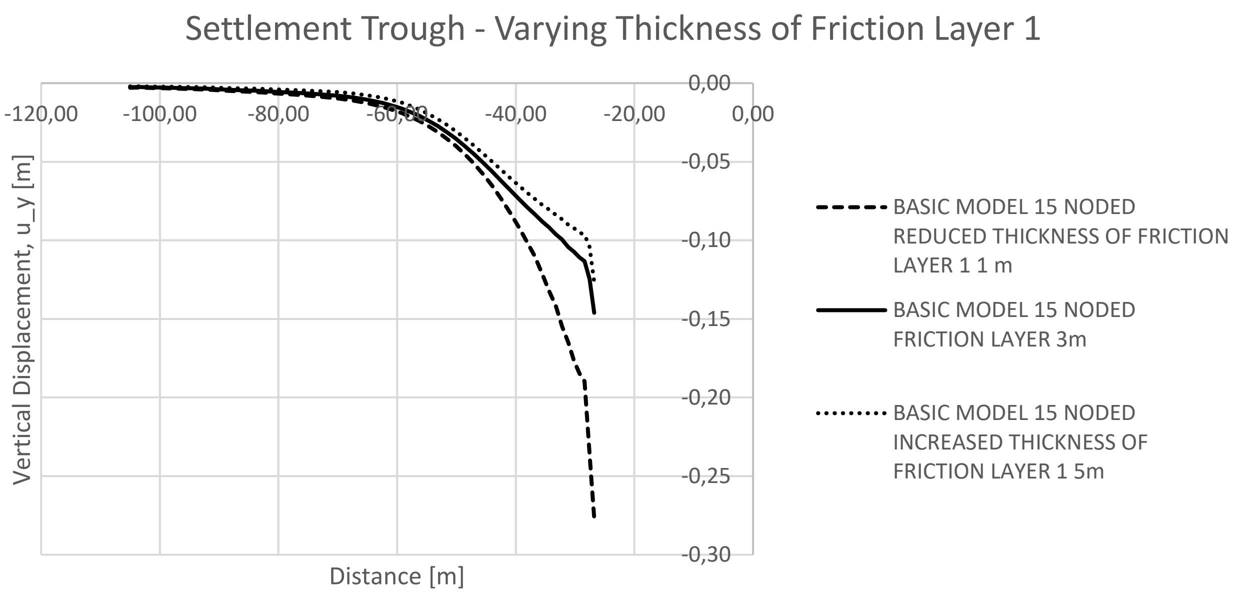

Figure 5.8. Settlement trough based on varying the thickness of Friction layer 1.

Horizontal displacements for the wall were also plotted, based on the three different thicknesses, for the last Phase. The plot is presented in Figure 5.9. It can be seen that reducing the thickness yields a significant increase in displacement.

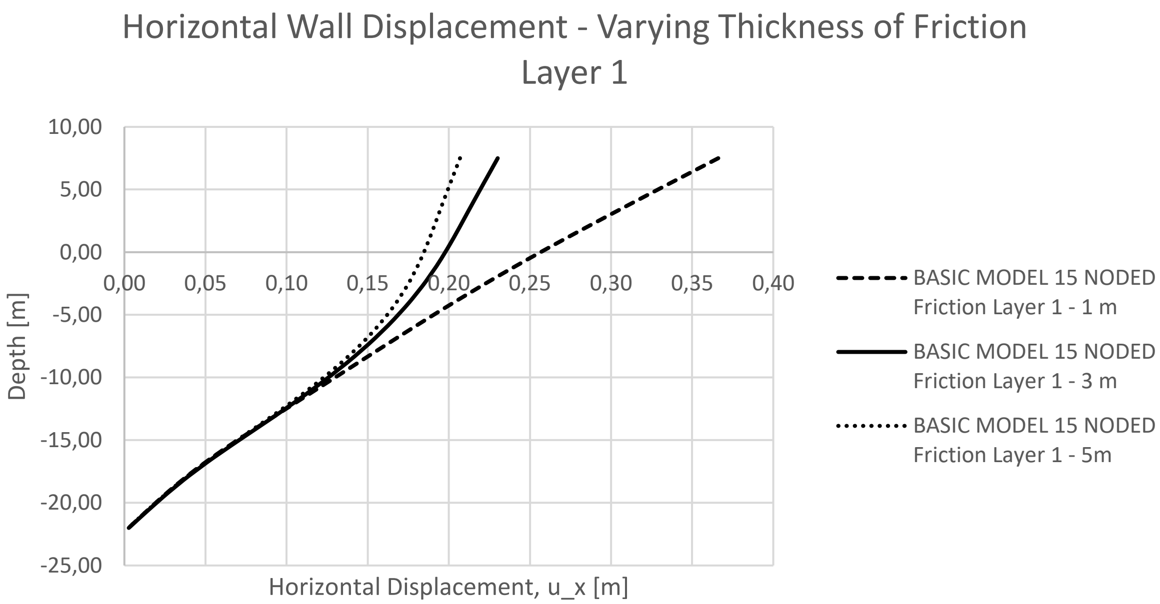

Figure 5.9. Horizontal wall displacements plotted using three separate thicknesses of the upper friction layer.

Furthermore, the heave of the excavation bottom was investigated, and no significant variance in the results could be seen.

To comment on the time dependent behaviour, some variance can be seen in the result for the different thicknesses. For Clay layer 1 this does manifest in slightly lower dissipation rates, when reducing the thickness of the layer. This change is quite logical, since the permeable and therefore also draining layer is reduced. Friction layer 2 is also impacted, the dissipation rates also varies. When increasing the thickness, the rate is also increased and in a similar manner the rate is decreased when reducing the thickness. A final comment on the result of varying the thickness of Friction layer 1: In Phase 3 and 4 the variance mainly influences the distribution of excess pore pressures, but not the magnitude. The thickness of the Friction layer can clearly be seen in in the plot, as the distribution inside the friction layer is quite linear. In the final phase, Phase 9, can a similar distribution still be seen. However, higher excess pore pressures are recorded when reducing the thickness, which could be explained by the increased clay depth.

5.2.5 Varying Strength and Stiffness of Friction layer 2

To further investigate how the lower Friction layer affects the result, the strength and stiffness parameters, Young's Modulus and Friction angle, were varied. The results presented are the surface settlements, the wall displacement and the heave of excavation bottom. In Figure 5.10 the settlement trough is plotted for the four cases, plus the basic model, with all other parameters unchanged, in the final Phase.

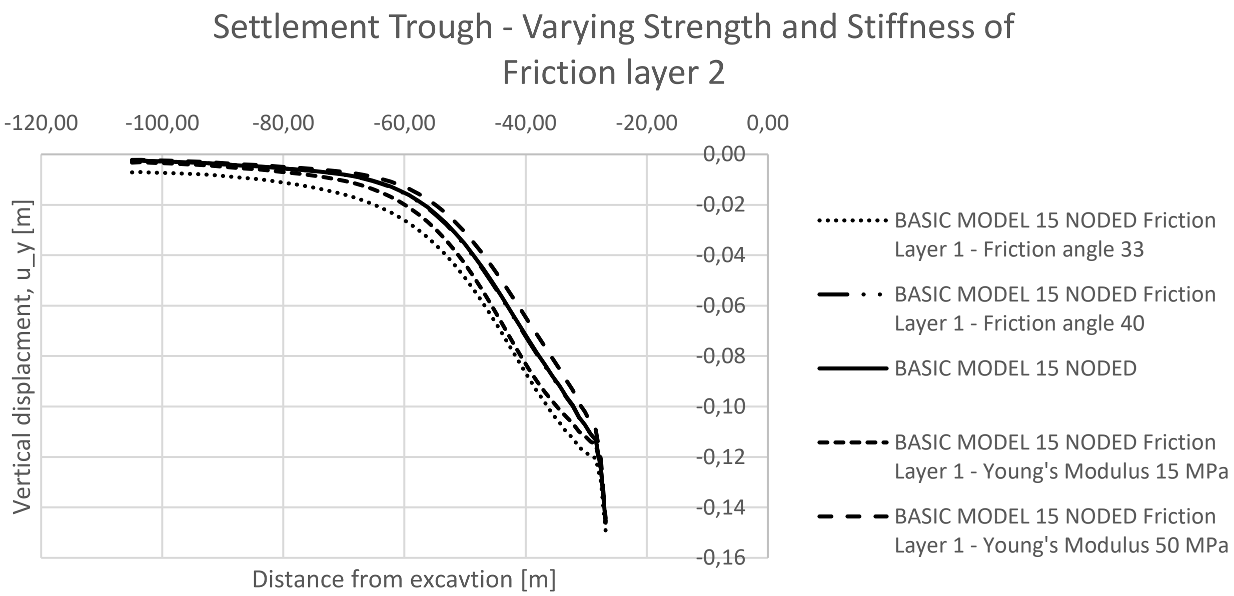

Figure 5.10. Settlement trough plotted for variated strengths and stiffnesses of Friction layer 2.

Furthermore, was the wall displacement plotted over depth for Phase 9. The result can be seen in Figure 5.11.

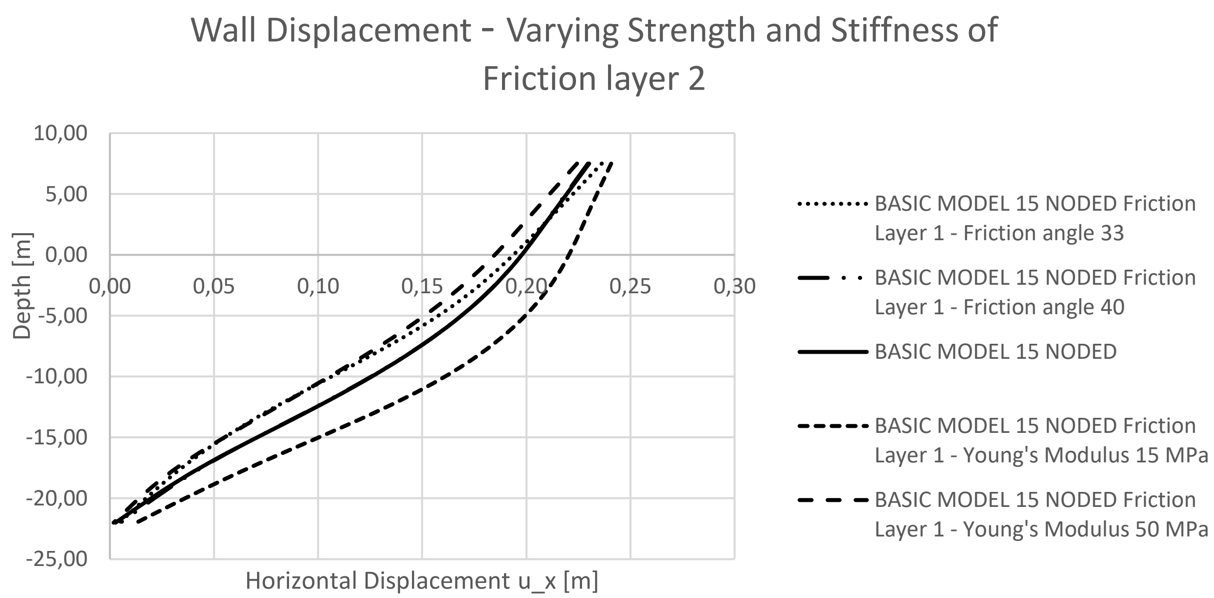

Figure 5.11. Wall displacement plotted for variated strengths and stiffnesses of Friction layer 2.

The most significant change in results could be seen when reviewing the heave of the excavation bottom. Below in Figure 5.12 presented from varying strength and stiffness parameters, in Phase 9.

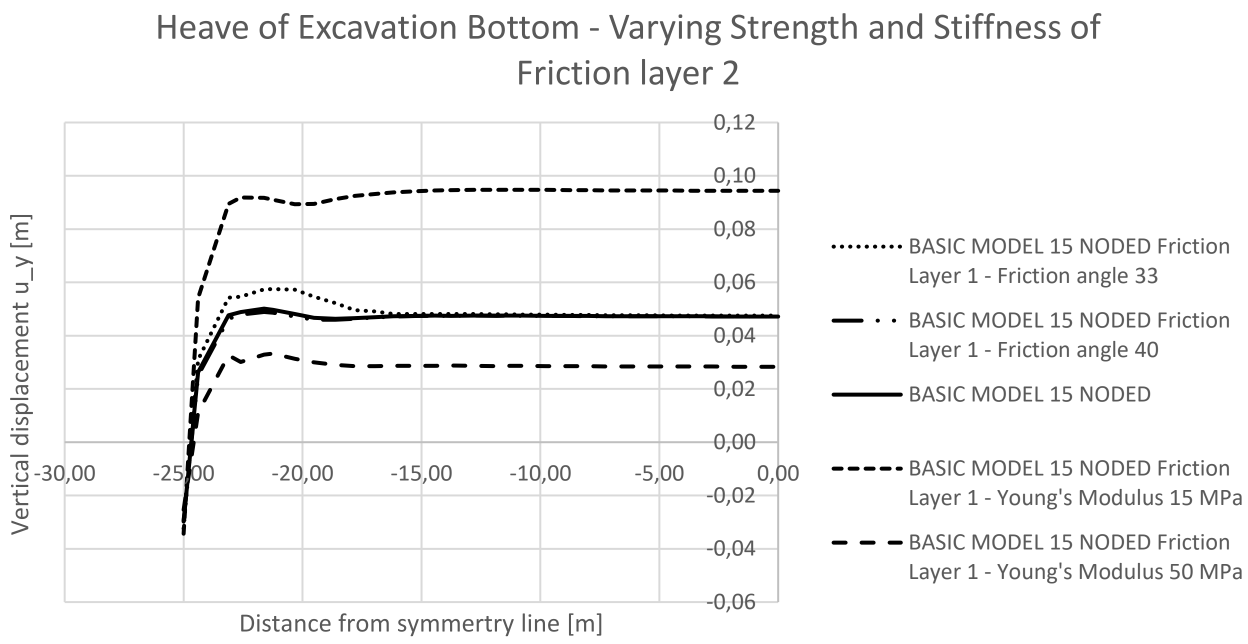

Figure 5.12. Heave of excavation bottom plotted for variated strengths and stiffnesses of Friction layer 2.

When increasing the Young's Modulus, significantly lower values of heave are predicted, the opposite is identified when decreasing the modulus. This result is also quite intuitive, a stiffer material is less prone to heave.

Excess pore pressures were not plotted, since only the strength and stiffness of the lowest layer was varied.

5.2.6 Influence of Grout Curtain

To investigate how the Grout curtain influences the groundwater flow and excess pore pressures Fully coupled flow-deformation analyses were carried out. The result is presented in the following section. For clarity, the model named 'Basic Model' uses Consolidation as analysis type and is only included as reference. Firstly, was the settlement trough plotted for Phase 9 in Figure 5.13 to study the impact of the grout curtain. It can be noted that the dotted line is hardly visible, since both plots for the fully coupled flow-deformation analysis are practically identical.

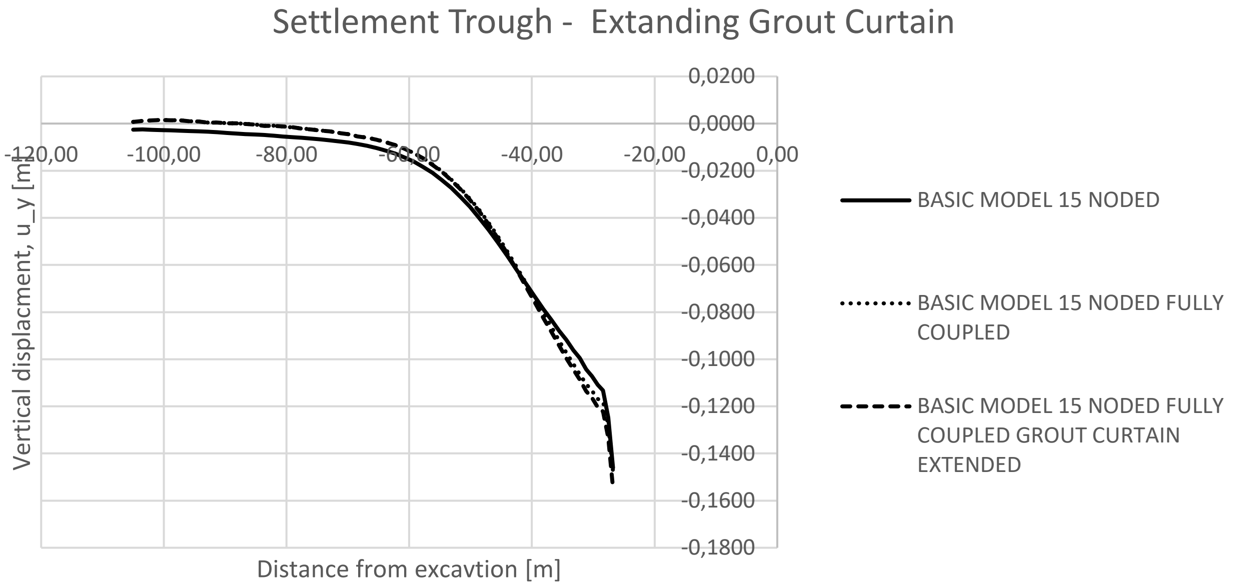

Figure 5.13. Settlement trough plotted for various extensions of the grout curtain.

Moreover, the wall displacement was investigated, the result can be seen in Figure 5.14 for Phase 9. No significant variance in the result can be seen.

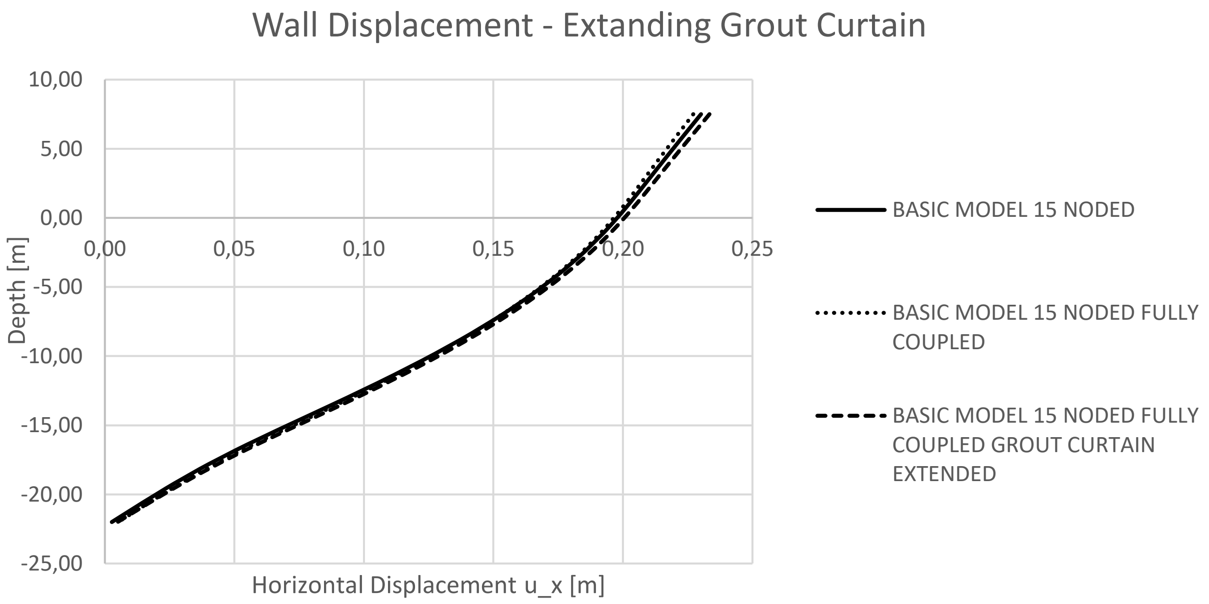

Figure 5.14. Wall displacement plotted for various extensions of the grout curtain.

Furthermore, the heave of the excavation bottom was studied. The result is depicted in Figure 5.15 for the final Phase. The difference seen in the plot is mainly between the two analysis types. No significant difference can be observed when implementing a prolonged grout curtain.

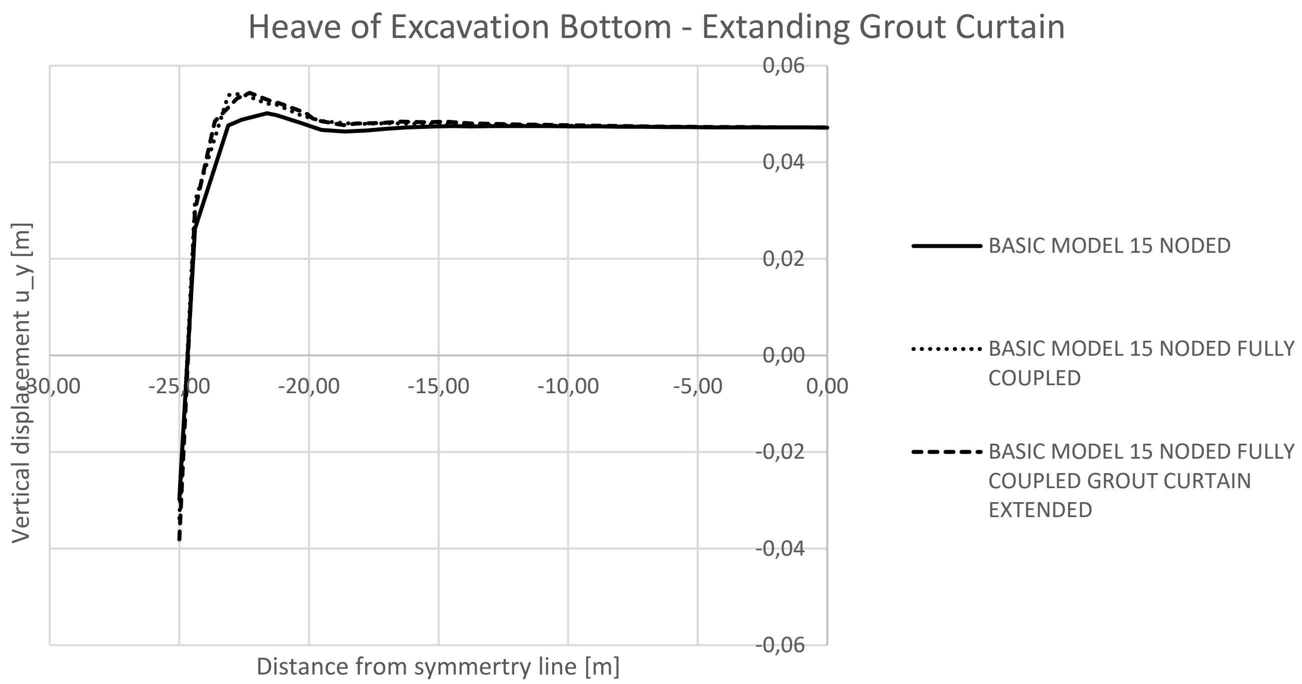

Figure 5.15. Heave of excavation bottom plotted for various extensions of the grout curtain.

Furthermore, the groundwater flows beneath the wall was investigated, the result can be seen in Figure 5.16 and Figure 5.17. Extending the grout curtain is efficiently stopping any groundwater from flowing beneath the wall.

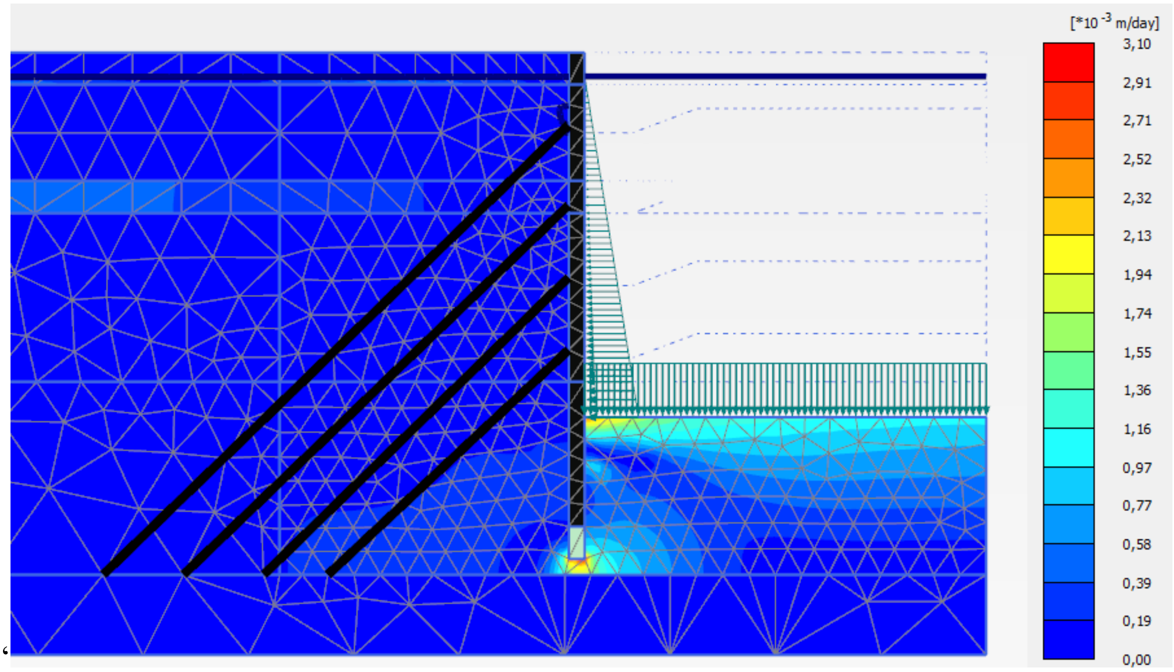

Figure 5.16. Groundwater flow analysis, grout curtain extended 2 meters below the wall.

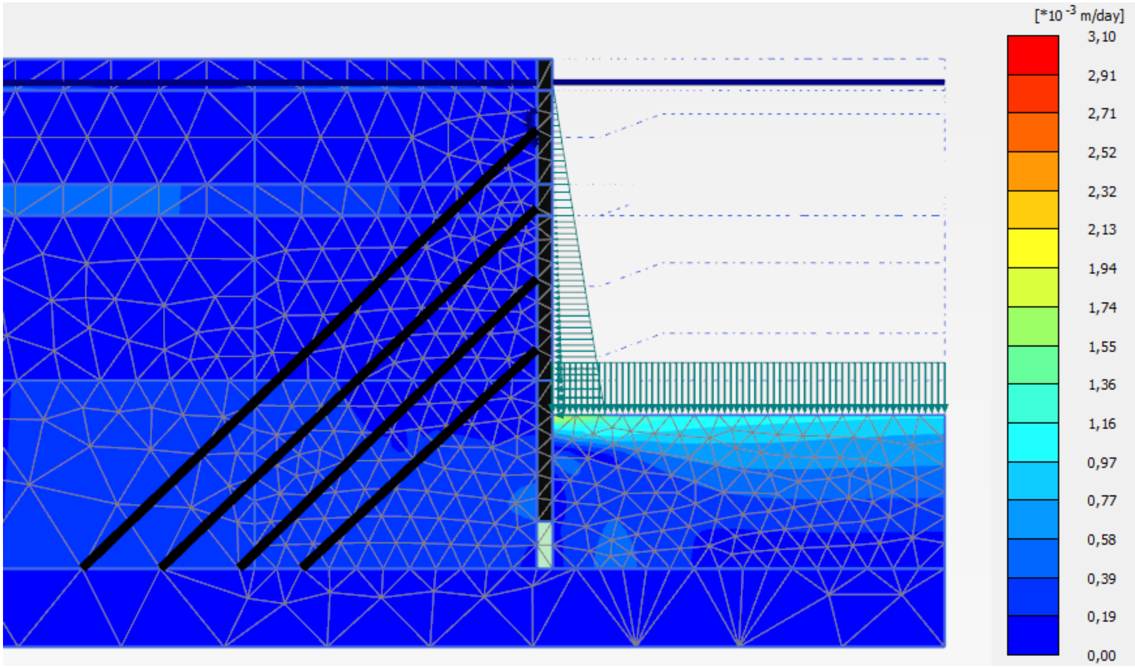

Figure 5.17. Groundwater flow analysis, grout curtain extended to bedrock.

5.3 Sensitivity Analysis

The following section consists of the performed sensitivity analyses, varying OCR, permeability, Modified Compression index and lastly the Modified Swelling index, separately. The result is presented in Table 5.3, Table 5.4 Table 5.5 and Table 5.6, respectively where the reference values are marked in grey.

Table 5.3.Sensitivity analysis based on variation of OCR, values marked in grey represent the values used as reference in previous analyses.

Soil Layer	*OCR*	*Max. Vertical Displacement [m]*	*Max. horizontal Displacement [m]*	*Change [%]*
Clay 1	1.7	0.1838	0.2294	-
	1.5	0.2352	0.2791	+28/+22
	1.9	0.1499	0.1988	-18/-11
Friction Layer 1	1.2	0.1838	0.2294	-
	1.1	0.1873	0.2318	+2/+1
	1.3	0.1825	0.2279	-1/-1
Clay 2	1.1	0.1838	0.2294	-
	1.0	0.197	0.241	+7/+5
	1.2	0.1644	0.2094	-11/-9

Table 5.4. Sensitivity analysis based on variation of permeability values marked in grey represent the values used as reference in previous analyses.

Soil Layer	*k [m/day]*	*Max. Vertical Displacement [m]*	*Max. horizontal Displacement [m]*	*Change [%]*
Clay 1	6E-5	0.1838	0.2294	-
	6E-4	0.1898	0.239	+3/+4
	6E-6	0.1822	0.2249	-1/-2
	2.76E-2	0.1838	0.2294	-

Friction Layer 1	2.76E-1	0.1879	0.2355	+2/+3
	2.76E-3	0.1785	0.2355	-3/-2
Clay 2	1.9E-5	0.1838	0.2294	-
	1.9E-4	0.1836	0.23	+0/+0
	1.9E-6	0.1815	0.2282	-1/-1
Friction Layer 2	2.59	0.1838	0.2294	-
	25.9	0.184	0.2296	+0/+0
	0.259	0.1844	0.2286	-0/-0

Table 5.5. Sensitivity analysis based on variation of Modified Compression index values marked in grey represent the values used as reference in previous analyses.

Soil Layer	λ^*	*Max. Vertical Displacement [m]*	*Max. horizontal Displacement [m]*	*Change [%]*
Clay 1	0.145	0.1838	0.2294	-
	0.1305	0.1811	0.2262	-1/-1
	0.1595	0.1865	0.2319	+1/+1
Friction Layer 1	0.167	0.1838	0.2294	-
	0.153	0.1851	0.2301	+1/0
	0.187	0.1847	0.2294	+0/+0
Clay 2	0.0896	0.1838	0.2294	-
	0.0806	0.1696	0.2161	-8/-6
	0.0986	0.1968	0.2413	+7/+5

Table 5.6. Sensitivity analysis based on variation of Modified Swelling index values marked in grey represent the values used as reference in previous analyses.

Soil Layer	κ^*	*Max. Vertical Displacement [m]*	*Max. horizontal Displacement [m]*	*Change [%]*
Clay 1	0.022	0.1838	0.2294	-
	0.0203	0.1818	0.2249	-1/-2
	0.0248	0.1857	0.2336	+1/+2
Friction Layer 1	0.021	0.1838	0.2294	-
	0.0189	0.1844	0.2292	+0/+0
	0.0231	0.1842	0.2301	+0/+0
Clay 2	0.0215	0.1838	0.2294	-
	0.0194	0.1823	0.2254	-1/-2
	0.0237	0.1854	0.2324	+1/+1

6 Discussion

In order to analyse how the uncertainties in the parameter determination impacted the results several simulations were run. The uncertainties of the model were mainly related to Friction layer 1 and 2, specifically the permeability and stratigraphy. Furthermore, the strength and stiffness of the bottom friction layer were varied. Lastly, the impact of the grout curtain was studied, a fully coupled flow-deformation analysis was then carried out to investigate the groundwater flows, which is not possible when using the consolidation analysis type.

Varying the permeabilities influenced the excess pore pressures, mainly in Clay layers 1 and 2 and Friction layer 1. For lower values of permeability in Friction layer 1 a slower dissipation rate was observed especially in the Clay layer 1. When increasing the permeability faster dissipation rates were recorded, as expected. When investigating consolidation settlements dissipation rates could be of interest, since the consolidation is highly dependent on excess pore pressures. However, when decreasing the permeability, higher settlement values were obtained. This could be explained by further observing the excess pore pressures. For lower permeabilities higher excess pore pressures are built up, and the effective stresses are thus further reduced. To investigate this phenomenon, it would be interesting to study the vertical settlements on several depths, to see which layer contributes to the surface settlements. Varying the permeability of Friction layer 2 did not yield any notable changes in wall displacement. However, when studying the surface displacement an increase in displacements could be seen when lowering the permeability. No significant changes in either excess pore pressure distribution or dissipation rates could be seen. The change in surface settlement is thus the only factor of the investigated aspects that change in a noteworthy way.

When varying the stratigraphy, i.e. the thickness of Friction layer 1 significant changes in the settlements and the wall displacement were noticed. When decreasing the thickness larger displacements were observed. Clay then “replaced” the friction material, which likely caused the increase since the magnitude of the subsidence is highly dependent of the thickness of the compressible layer. In reality, the material was not either a friction material or clay but a combination of the two, with unique material properties. The lower Clay layer, which replaced Friction layer 1 when decreasing the thickness, was described as stiff. However, the determined Modified Compression index (λ^*) is lower in the lower Clay layer compared to the upper. In similar cases with Gothenburg clay, an increase with depth can be seen. The properties used for the Korsvägen might be invalid, and when increasing the thickness of Clay layer 2 are these properties used on even thicker deposit and could thus add to the increased settlements. However, the correlation of Modified Compression index over depth is complicated, further evaluation of laboratory data and literature is recommended to validate the parameter. Subsequently, when increasing the thickness of the Friction layer 1, settlements and displacements were lowered. Excess pore pressures were influenced, since the permeable layer was increased or reduced. The distribution was somewhat altered, but the magnitude of the pore pressure remained the same. In the plots of excess pore pressure over depth, the thickness of the friction layer could clearly be seen, since the Friction layer contains much lower excess pore pressures. When increasing or decreasing the thickness, the pore pressure distribution is thus altered.

Lastly, the influence of the Grout curtain was studied. When reviewing the groundwater flows, it could be seen that the curtain efficiently seals the gap between wall and bedrock. Since the excavation will be carried out in dry conditions, the seal could simplify the dewatering since groundwater flows are then limited.

When performing the sensitivity analyses, the greatest change in results are seen when varying the OCR, especially in Clay Layer 1. The Plaxis Soft Soil model is known of being sensitive to OCR and the determination of the OCR should therefore be performed with great care. Changing the Modified Swelling and Compression indices yield very little change in maximum displacements. Worth mentioning is also that sensitivity analysis was performed by varying one parameter at a time, to isolate the effect of each parameter. However, in reality are the parameters likely to be connected, which could affect the result even more.

6.1 Sources of Error

When analysing geotechnical problems sources of error are included in every step, from soil sampling, laboratory testing to parametric determination and analysis. First and foremost, the stratigraphy and soil conditions were idealized, to enable and simplify the numerical analysis. The stratigraphy clearly varies along the stretch at Korsvägen, however only one cross section was analysed, the result is thus biased. A safe or more optimized construction could be verified by creating several cross sections along the stretch. Many simplifications and assumptions were made during the creating of the model, which all influence the model. When reviewing and determining the permeabilities of the soil layers, it was assumed that the horizontal and vertical permeabilities were equal, when in reality a significant difference is likely.

In the analyses simple constitutive soil models were used, to better describe the soil behaviour could higher order models be used. However, with the information available the simple models was deemed the best. Using higher order models would entail more assumptions and estimations. It must be decided in during the process how precise the constitutive model needs to be and what accuracy is necessary in the analyses. Furthermore, can it be questioned if the Plaxis Soft Soil model is suitable for modelling the friction layer. The Soft Soil model is created to simulate the behaviour of clays and clay silts, whereas the Friction layer 1 mainly consists of sand and silt fractions.

The construction sequence and construction times were only estimated. The sequence used in the thesis is a common approach, but due to the complicated nature of the excavation could another method of excavation be used. Also, the time of construction and open excavation was not with certainty determined. If, in reality, a longer time span is expected, and creep should thus be considered.

Lastly, a conservative approach was used when modelling the anchors, since no pre-tensioning was assumed. If some pre-stressing would have been included, the horizontal wall displacements most likely would have been reduced.

7 Conclusion

To conclude, in this thesis subsidence has been investigated by using geometries and soil conditions from an excavation performed at Korsvägen as part of the West Link project. The excavation was further studied by analysing the surface settlements, the wall displacement, the heave and the excess pore pressures.

Strongest influence on surface settlements and wall displacement were obtained when varying the stratigraphy, namely the thickness of Friction layer 1. Excess pore pressures were generally most impacted when varying the permeability. Heave of the excavation bottom was not significantly influenced by any factor, except stiffness of the lower Friction layer. Furthermore, the sensitivity showed that the OCR value highly influenced the model.

Using a numerical tool greatly simplified the analysis, enabling fast and efficient varying of soil properties. However, the obtained result proves the importance of using validated input. Care must be taken when creating the model, to ensure correct geometries and parametric determination. The predictions can only be as good as the input data. When creating a valid prediction the assumptions, simplifications and correlations used to produce the model parameters must be closely considered.

In an urban environment, a small variance in settlements or differential settlements can cause severe damages. Subsidence could therefore be an issue at Korsvägen, mainly since the area round the future excavation is so densely built-up. During construction time will trams and traffic still run within the area, close to the excavation which could be problematic since the potential subsidence is largest close to the excavation.

8 Further Investigation

To suggest further investigation on the subject of subsidence, the list below was created;

- Site specific;
  - Model dewatering using the method used during construction, to make the groundwater lowering as close to reality as possible.
  - Vary properties of secant pile wall, how would a semi-permeable wall affect the results.
  - Explore a longer time span and take creep into consideration.
  - Compare results to measured data when the excavation has been constructed.
  - Focus on Groundwater flows analysis, to see how surrounding sensitive foundations are affected.
  - Create a Plaxis 3D model to take the complex geological, and hydrological, conditions into account.

- General;
  - Further analyse the impact of subsidence using the Fully coupled flow-deformation analyses.

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Appendix A – Soil Properties

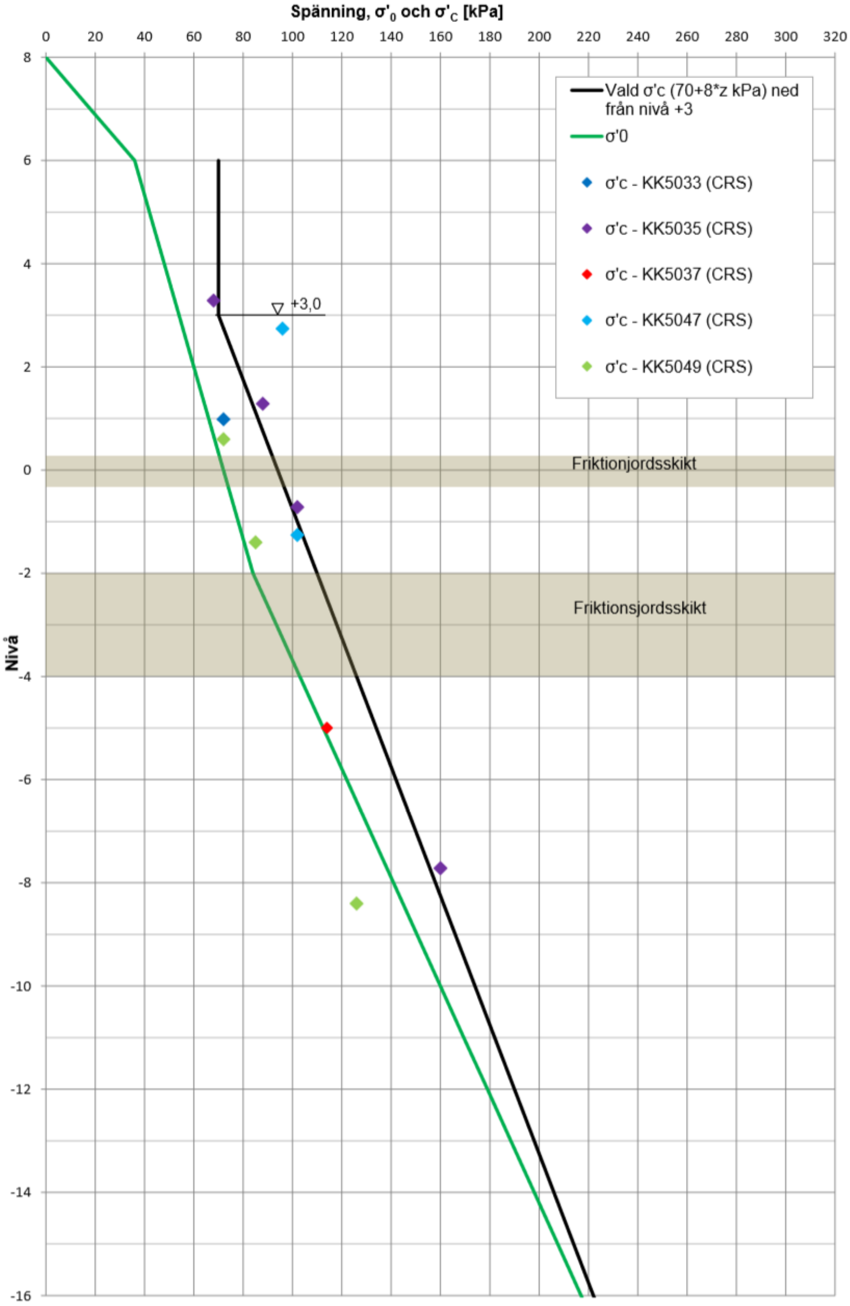

Figure A.1. Effective stresses and Pre-consolidation stresses from Sweco Civil AB (2014).

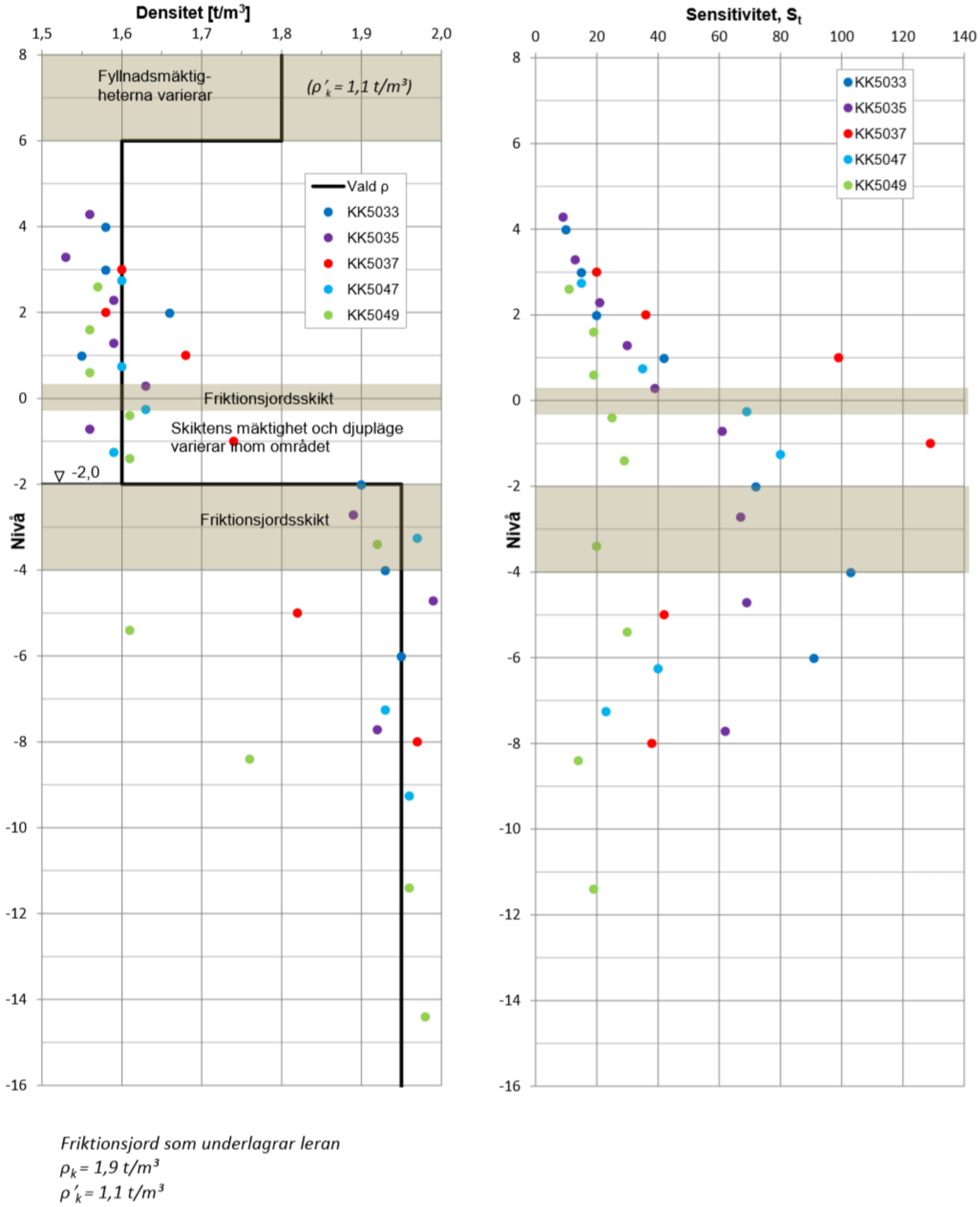

Figure A.2. Soil Density (left) and Sensitivity (right) from Sweco Civil AB (2014).

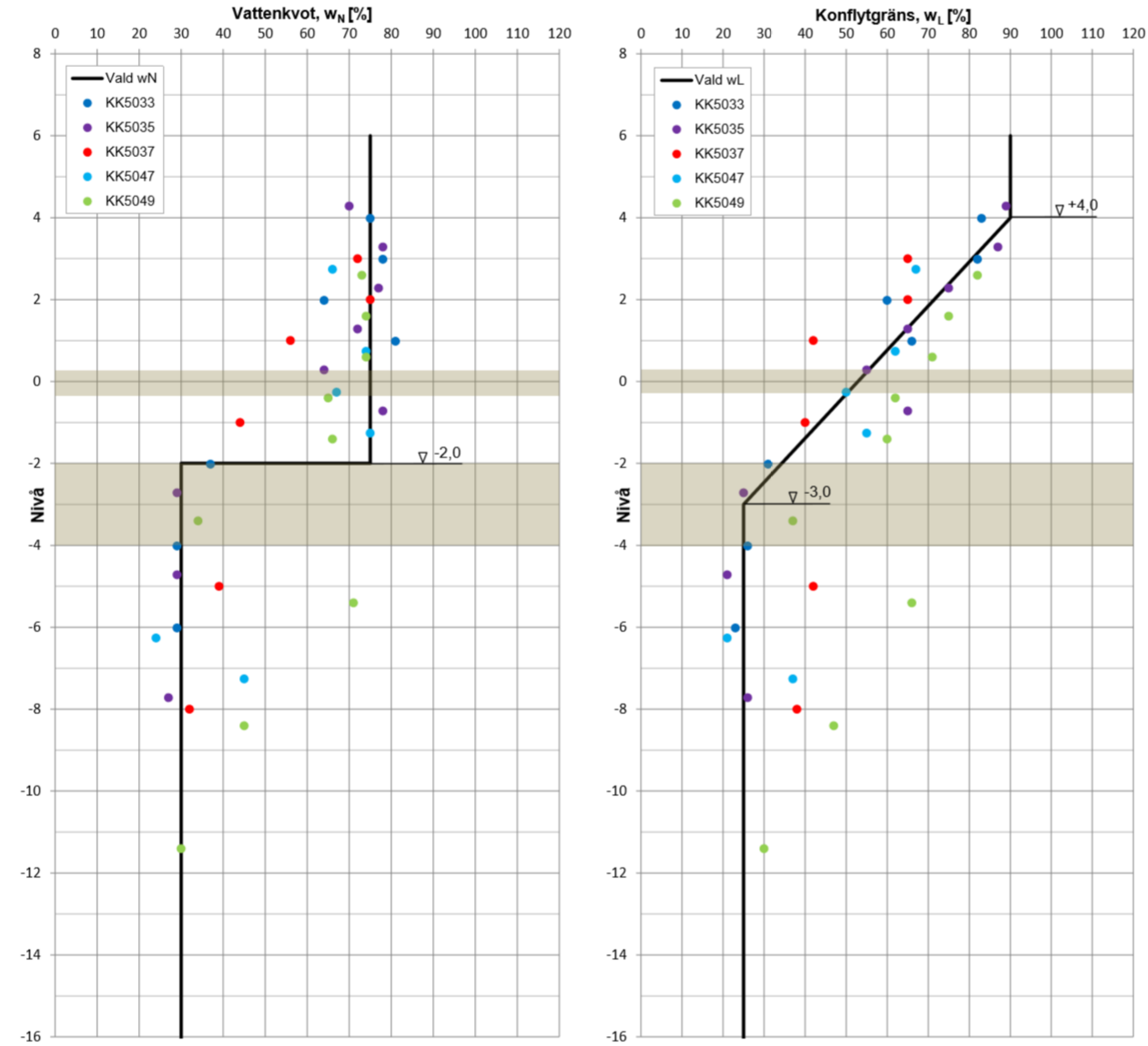

Figure A.3. Water Content (left) and Liquid Limit (right) from Sweco Civil AB (2014).

Appendix B – Numerical Simulations

The following Appendix consists of Table B.1, where the simulations run in Plaxis 2D are tabulated. Furthermore, are the coordinated of each investigation point or cross section given in Table B.2.

Table B.1. Simulations run in Plaxis 2D presented and explained.

Model Name	*Properties*	*Nodes*	*Analysis Type*	*Comment*
Korsvägen 1	Basic – 6 nodes	6	Consolidation	-
Korsvägen 1. A	Basic – 15 nodes	15	Consolidation	-
Korsvägen 1.2	Fine Mesh	15	Consolidation	Convergence Test
Korsvägen 2.1	k= 3.2E-6 m/s k= 0.276 m/day	15	Consolidation	Investigating impact of permeability of Friction Layer 1
Korsvägen 2.2	k= 3.2E-5 m/s k=2.76 m/day	15	Consolidation	Investigating impact of permeability of Friction Layer 1
Korsvägen 2.3	k= 3.2E-8 m/s k=0.00276 m/day	15	Consolidation	Investigating impact of permeability of Friction Layer 1
Korsvägen 2.4	k= 3E-4 m/s k= 25.9 m/day	15	Consolidation	Investigating impact of permeability of Friction Layer 2
Korsvägen 2.5	k= 3E-6 m/s k= 0.259 m/day	15	Consolidation	Investigating impact of permeability of Friction Layer 2
Korsvägen 2.6	Grout Curtain extended to Bedrock	15	Consolidation	Investigating the impact of the Grout Curtain
Korsvägen 3.1	Increased thickness of Friction Layer 1 to 5 m	15	Consolidation	Investigating impact of stratigraphy
Korsvägen 3.2	Reduced thickness of Friction Layer 1 to 2 m	15	Consolidation	Investigating impact of stratigraphy
Korsvägen 4.1	Friction Angle of Friction layer 2 φ= 33°	15	Consolidation	Investigating impact of strength and stiffness
Korsvägen 4.2	Friction Angle of Friction layer 2 φ= 40°	15	Consolidation	Investigating impact of strength and stiffness
Korsvägen 4.3	Young's Modulus of Friction layer 2 E=15 MPa	15	Consolidation	Investigating impact of strength and stiffness
Korsvägen 4.4	Young's Modulus of Friction layer 2	15	Consolidation	Investigating impact of strength and stiffness

	E=50 MPa			
Korsvägen 5.1	Basic	15	Fully Coupled Flow-Deformation	Basic Model
Korsvägen 5.1	Basic –Grout Curtain extended to Bedrock	15	Fully Coupled Flow-Deformation	Investigating the impact of the Grout Curtain

Table B.2.Coordinates and purpose of investigation points and cross section used in analysis.

Model Aspect	*Investigated Parameter*	*Abbreviation*	*Coordinates of Cross Section*
Settlement Trough	Vertical Displacement	u_y	x= 105 to –26 y= +6.5
Wall Displacement	Horizontal Displacement	u_x	x= –25.5 y= +7.5 to –22
Excess Pore Pressure (next to wall)	Excess Pore Pressure	p_excess	x= –27 y= +7.5 to –30
Heave	Heave of Excavation Bottom	u_y	x= –25 to 0 y= –16
Excess Pore Pressure (below excavation)	Excess Pore Pressure	p_excess	x= –0.5 y= –15.2 to –30
Excess Pore Pressure Clay 1	Excess Pore Pressure in the middle of the layer	p_excess	x= –27.93 y= 2.88
Excess Pore Pressure Friction layer 1	Excess Pore Pressure in the middle of the layer	p_excess	x= –27.14 y= –1.21
Excess Pore Pressure Clay 2	Excess Pore Pressure in the middle of the layer	p_excess	x= –27.66 y= –6.79
Excess Pore Pressure Friction layer 2	Excess Pore Pressure in the middle of the layer	p_excess	x= –27.19 y= –18.06

Appendix C – Soil Test

A parametric calibration was performed, using the Plaxis Soil test Tool. In the following Appendix is the result from the simulation and the laboratory data. Firstly, were the soil parameters obtained from the CRS tests evaluated and by calibrating the parameters was a satisfactory parametric determination obtained, the result is presented in Figure C.1, Figure C.2 and Figure C.3.

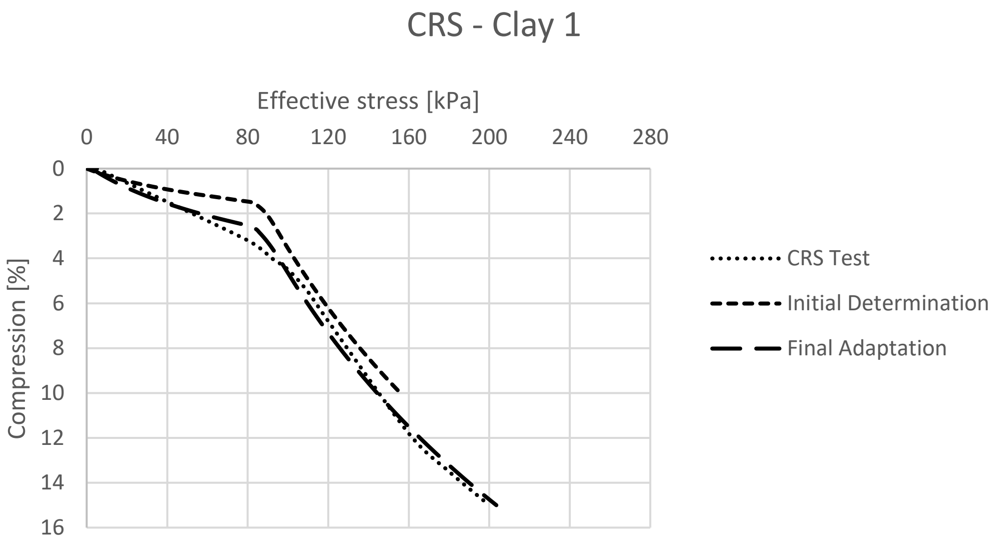

Figure C.1. Modelled behaviour of Clay 1, data from CRS test performed in laboratory and in Plaxis Soil Test tool.

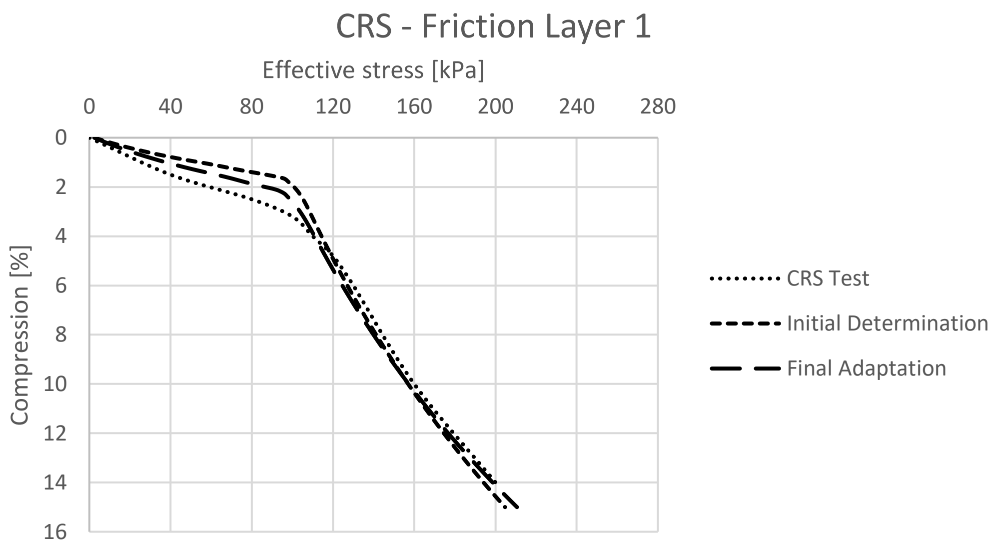

Figure C.2. Modelled behaviour of Friction Layer 1, data from CRS test performed in laboratory and in Plaxis Soil Test tool.

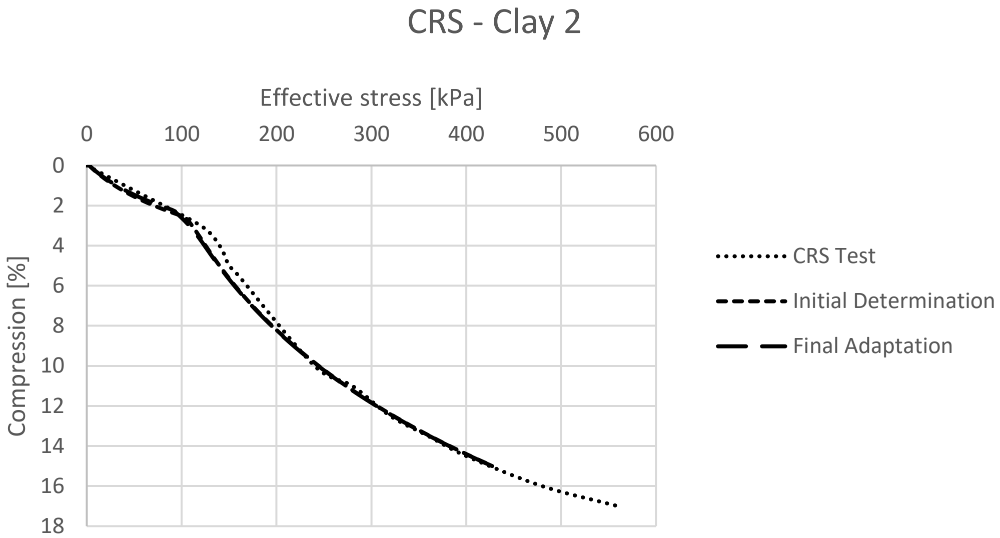

Figure C.3. Modelled behaviour of Clay 2, data from CRS test performed in laboratory and in Plaxis Soil Test tool.

Furthermore, were the soil parameters retrieved from the Triaxial test calibrated, the result is shown in Figures C.4 through C.6.

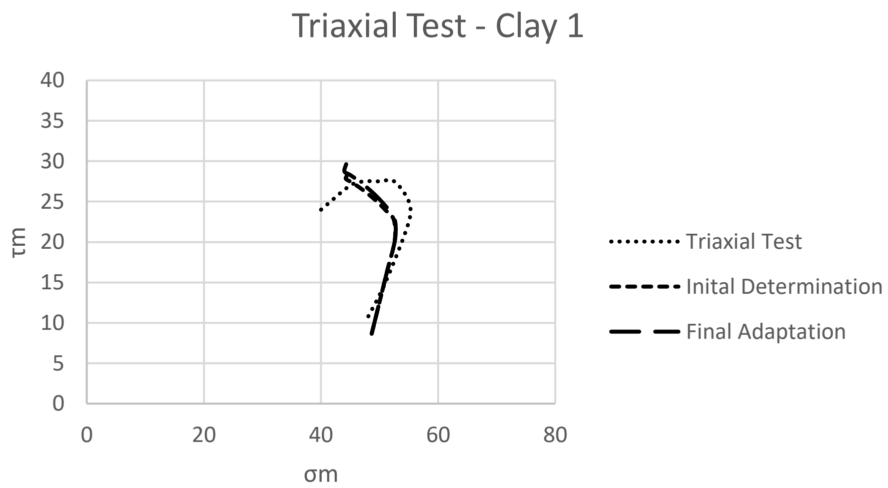

Figure C.4. Modelled behaviour of Clay 1, data from Undrained Triaxial test performed in laboratory and in Plaxis Soil Test tool.

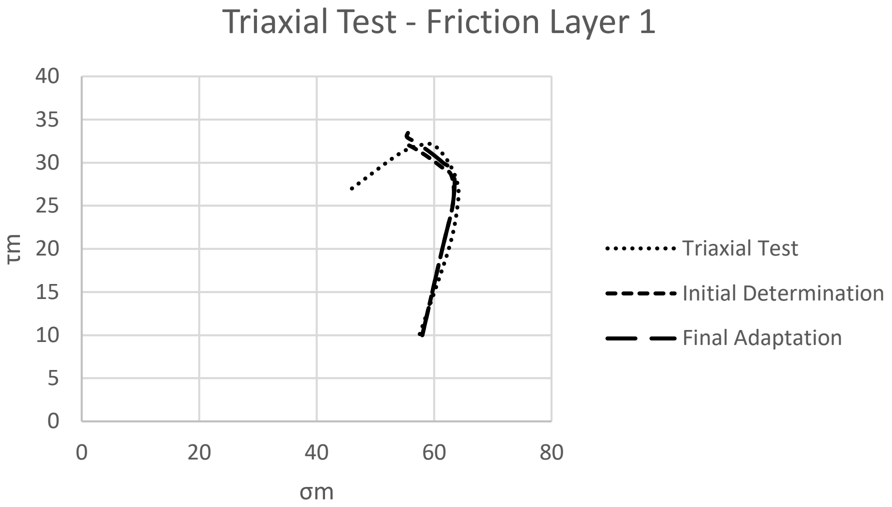

Figure C.5. Modelled behaviour of Friction layer 1, data from Undrained Triaxial test performed in laboratory and in Plaxis Soil Test tool.

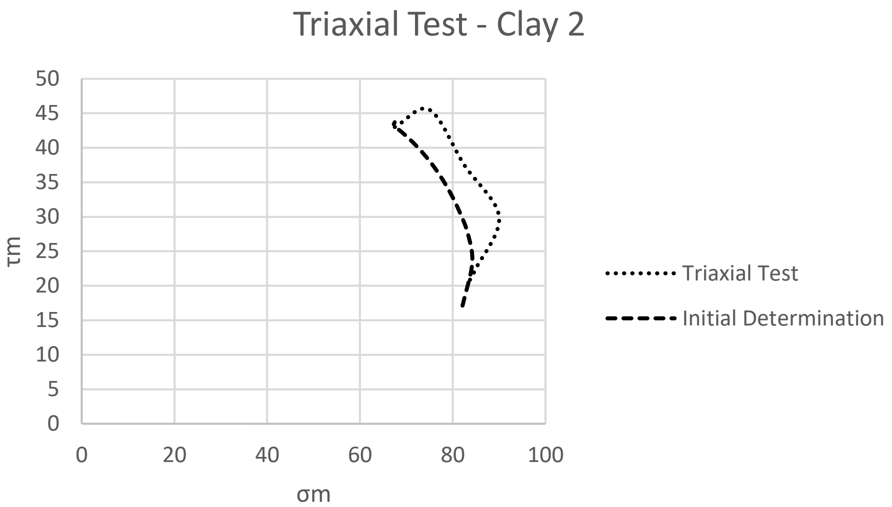

Figure C.6. Modelled behaviour of Clay 2, data from Undrained Triaxial test performed in laboratory and in Plaxis Soil Test tool.

Appendix D – Convergence Test

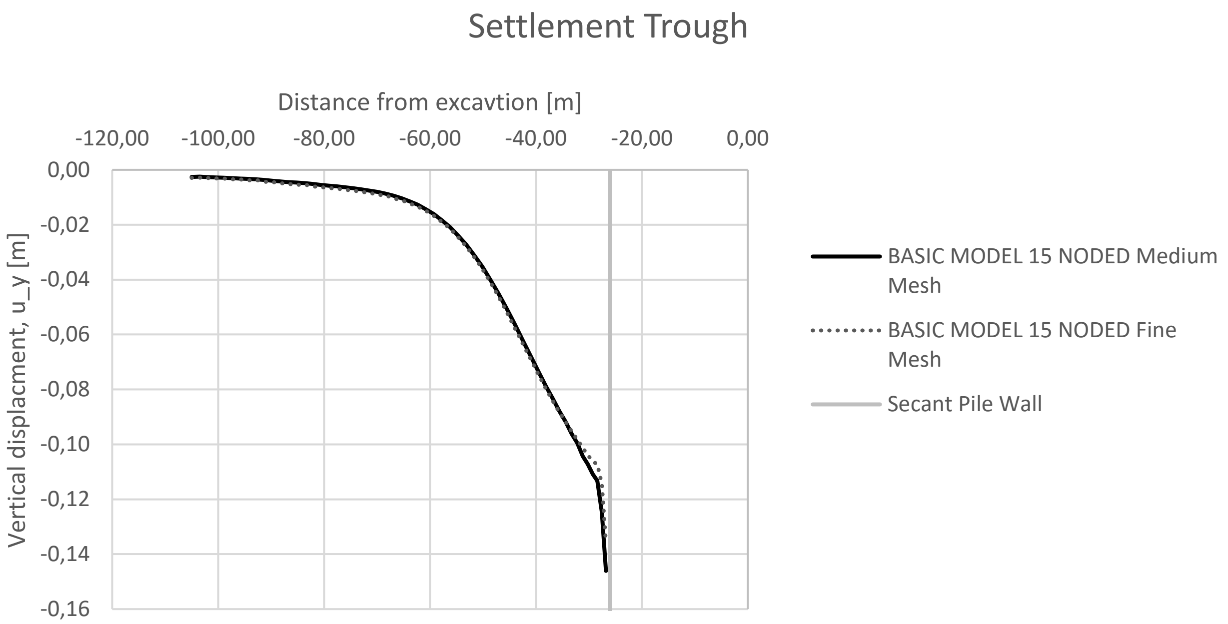

Figure D.1. Comparison of mesh settings for simulation of settlement trough, using Medium or Fine as Mesh setting.

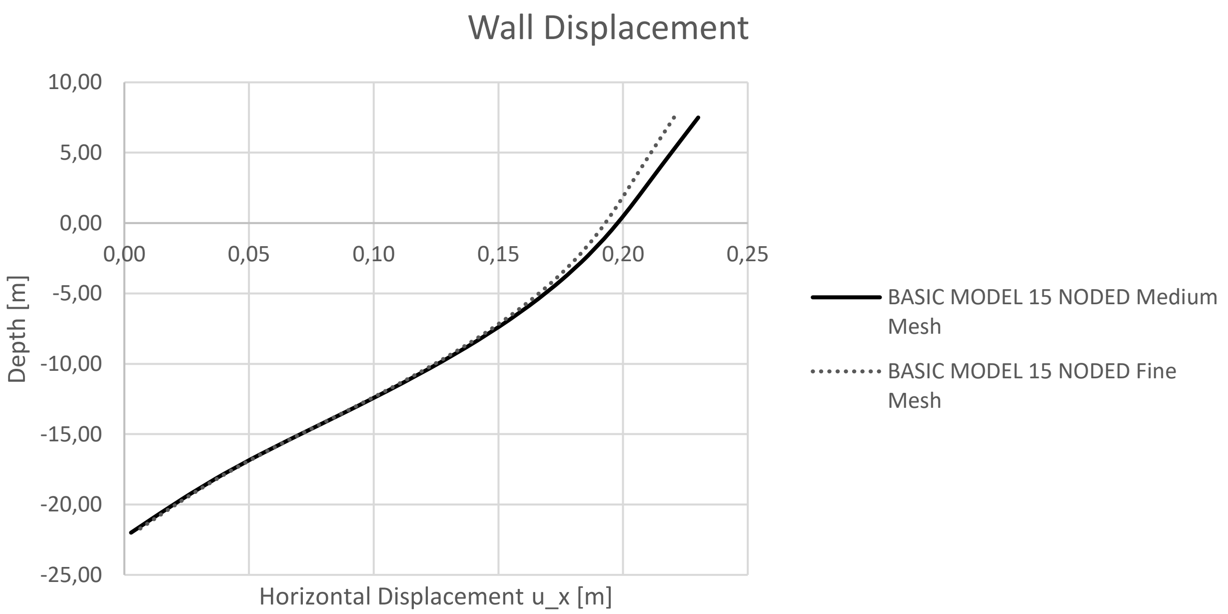

Figure D.2. Comparison of mesh settings for simulation from wall displacement, using Medium or Fine as Mesh setting.

Appendix E – Numerical Results

E.1 Varying Permeability of Friction Layer 1

Heave of excavation bottom is plotted in Figure E.1.

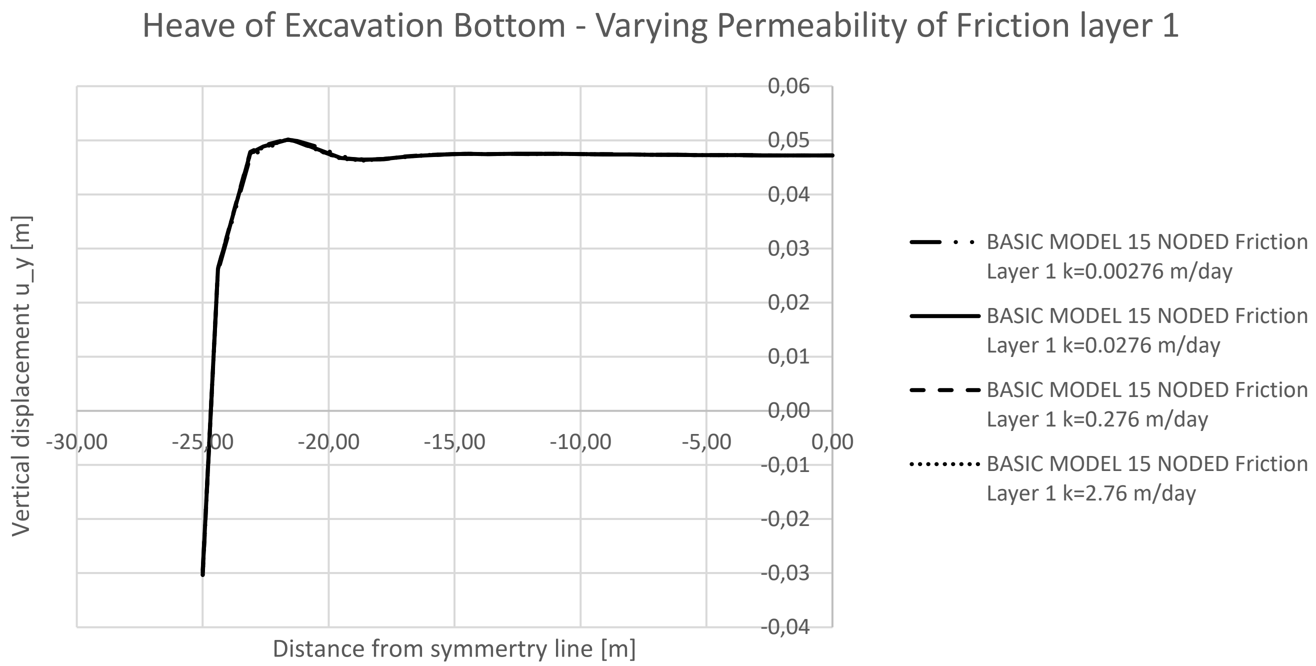

Figure E.1. Heave of excavation bottom plotted with varying permeabilities for Friction layer 1.

To analyse the excess pore pressures, how the pressures build up and dissipate, was the excess pressures plotted over time for each layer. A point was chosen in the middle of each layer, according to Table B.1 Appendix B – Numerical Simulations**Fel! Hittar inte referenskälla.**. The result of varying the permeability of Friction layer 1 can be seen for each layer in Figures E.2 through E.5.

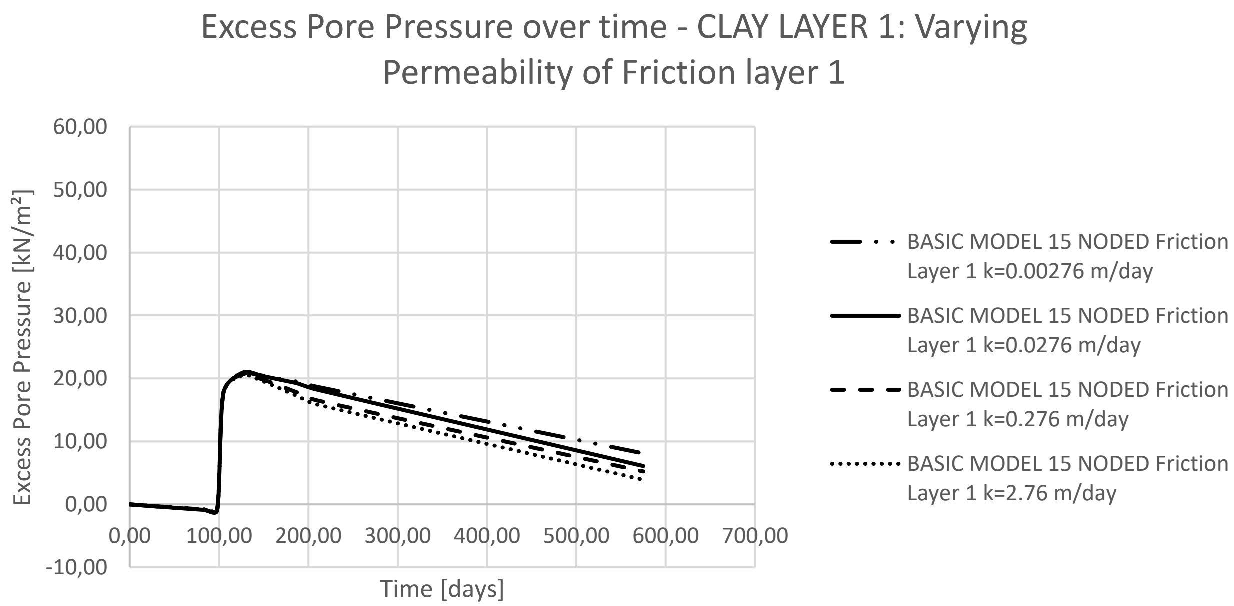

Figure E.2. Excess Pore Pressures plotted for a point in the middle of Clay layer 1, varying the permeability of Friction layer 1.

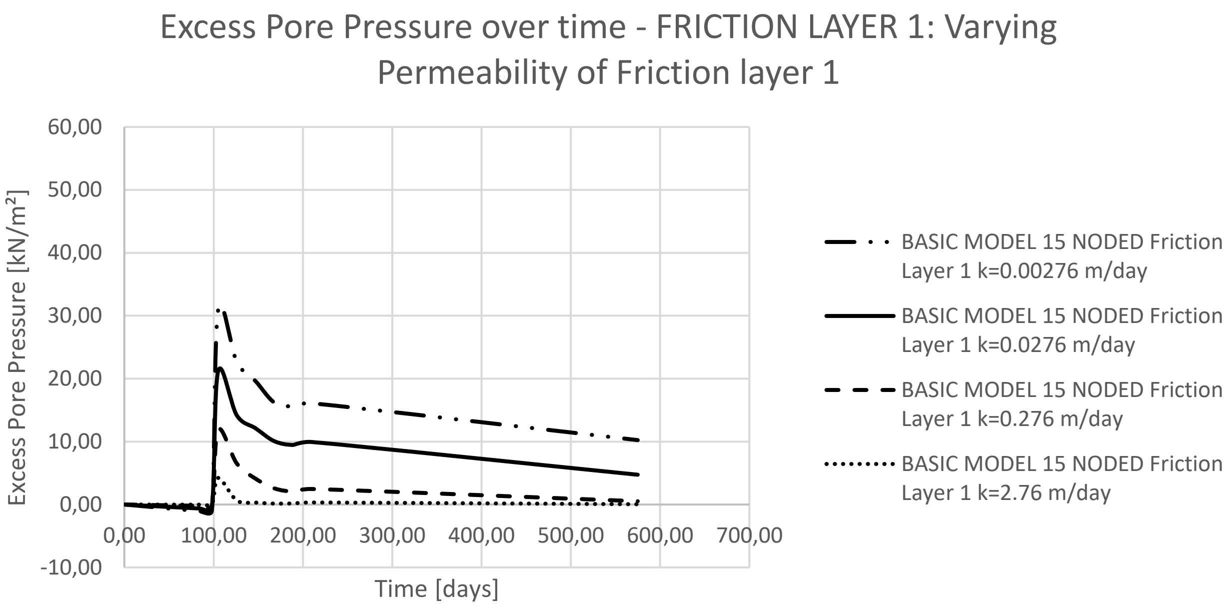

Figure E.3. Excess Pore Pressures plotted for a point in the middle of Friction layer 1, varying the permeability of Friction layer 1.

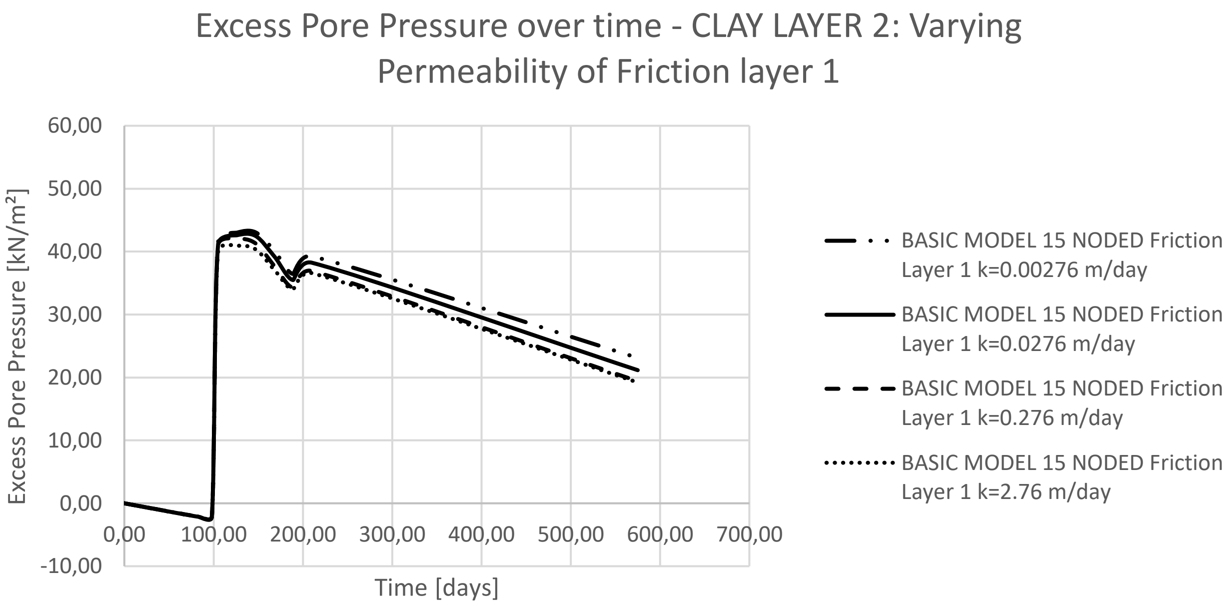

Figure E.4. Excess Pore Pressures plotted for a point in the middle of Clay layer 2, varying the permeability of Friction layer 1.

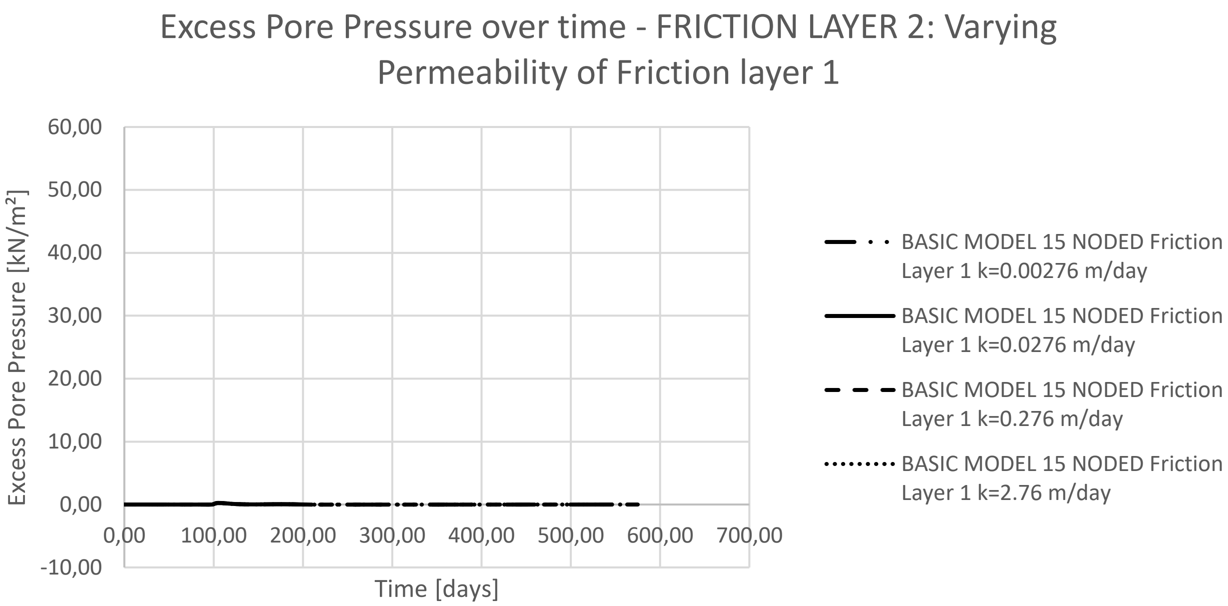

Figure E.5. Excess Pore Pressures plotted for a point in the middle of Friction layer 2, varying the permeability of Friction layer 1.

E.2 Varying Permeability of Friction Layer 2

Heave of excavation bottom is presented in Figure E.6 below, for varied permeabilities of Friction layer 2.

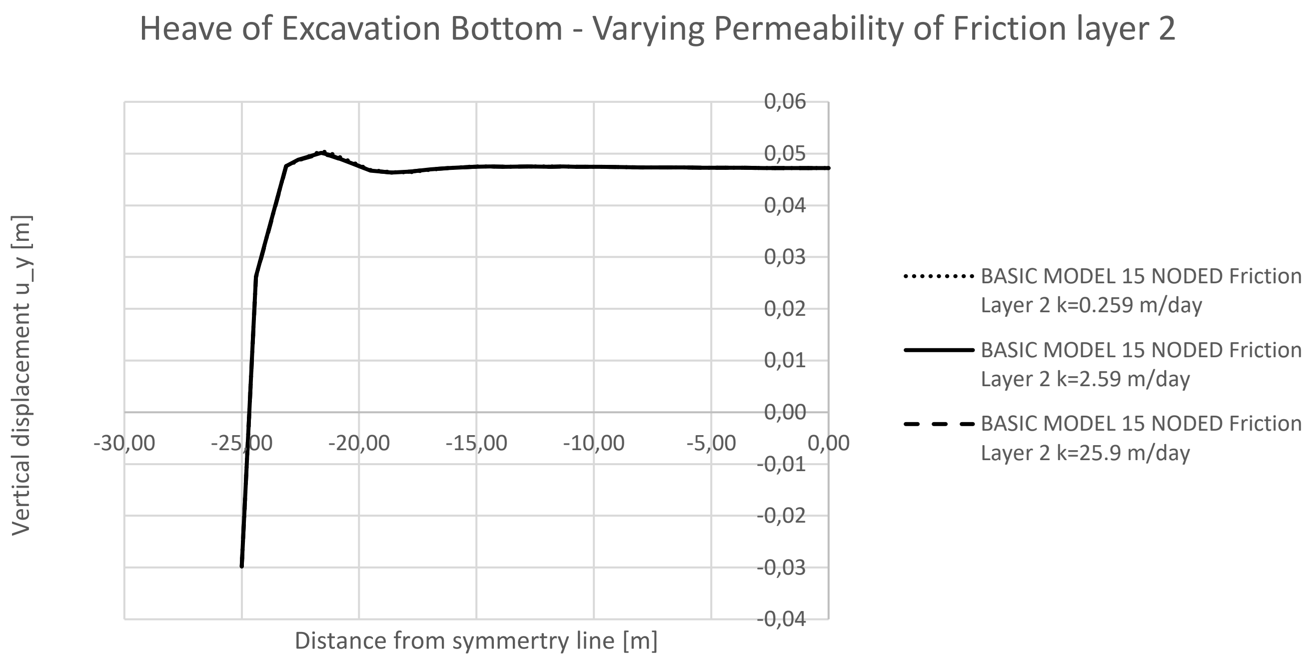

Figure E.6. Heave of excavation bottom plotted with varying permeabilities for Friction layer 2.

Moreover, were the excess pore pressures analysed by plotting the excess pore pressures over time. The result for each layer is presented separately in Figures E.7 through E.10.

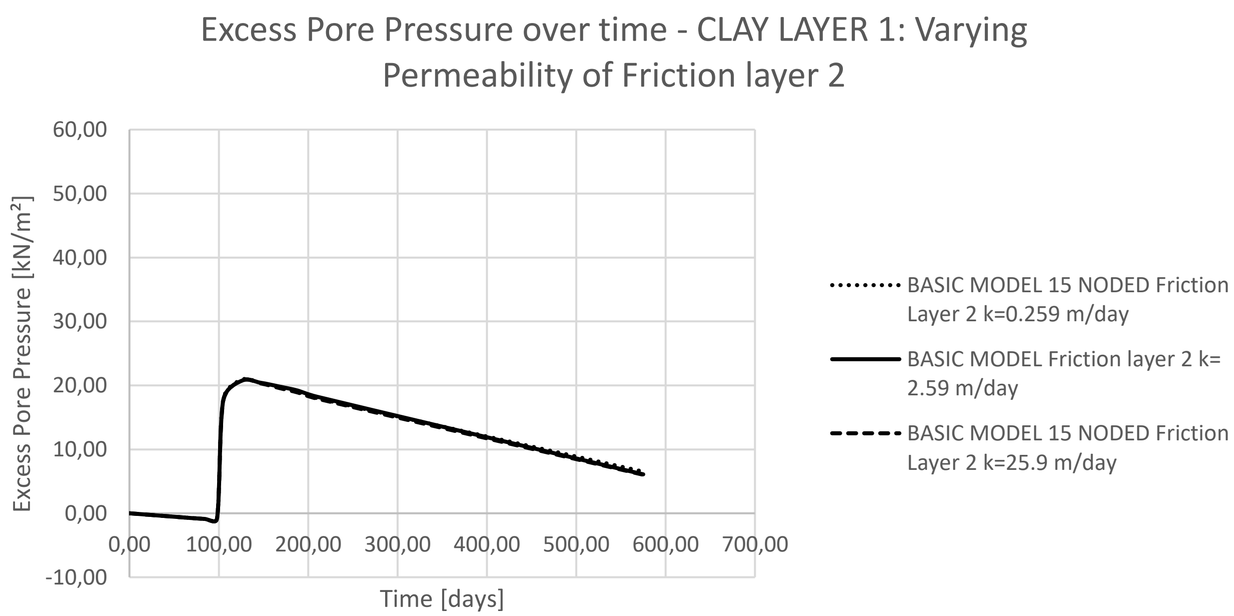

Figure E.7. Excess Pore Pressures plotted for a point in the middle of Clay layer 1, varying the permeability of Friction layer 2.

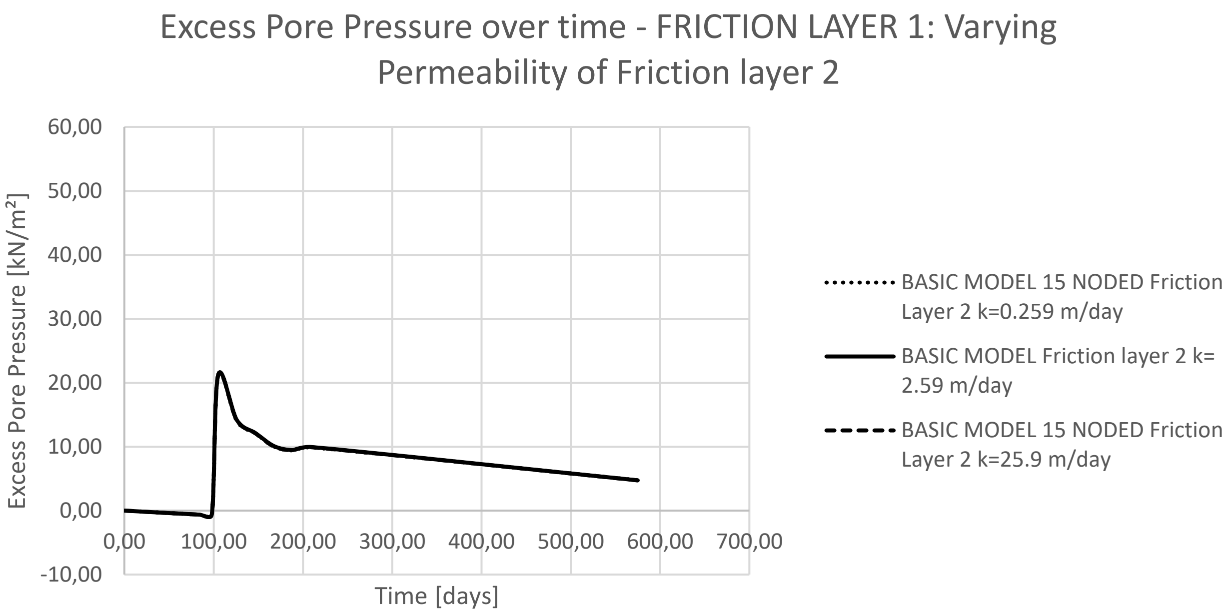

Figure E.8.Excess Pore Pressures plotted for a point in the middle of Friction layer 1, varying the permeability of Friction layer 2.

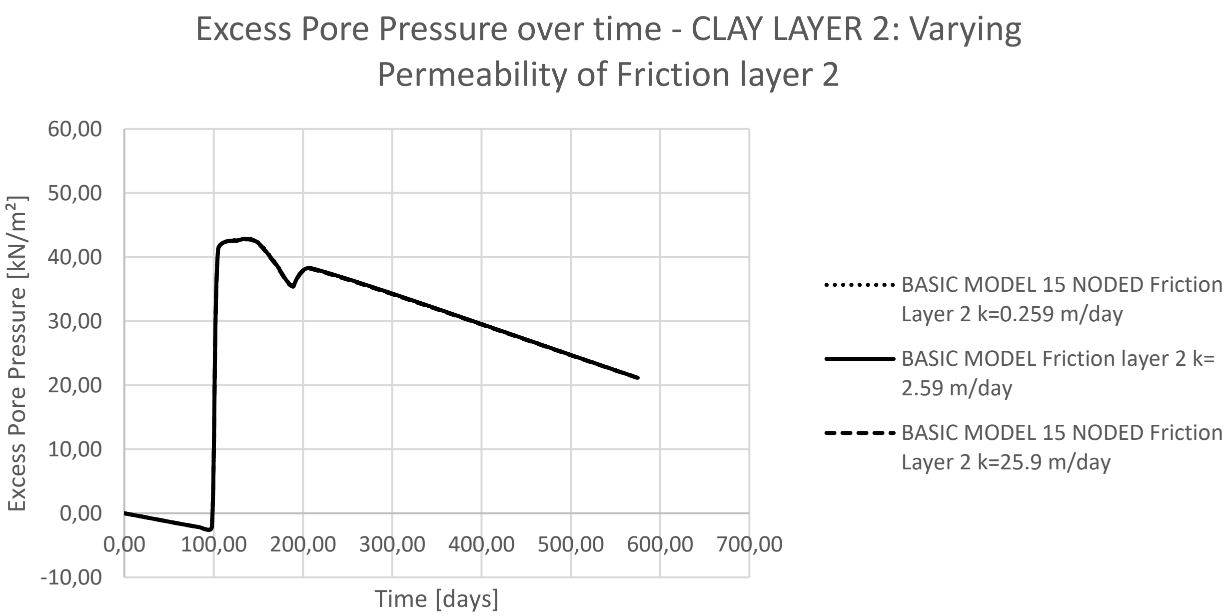

Figure E.9. Excess Pore Pressures plotted for a point in the middle of Clay layer 2, varying the permeability of Friction layer 2.

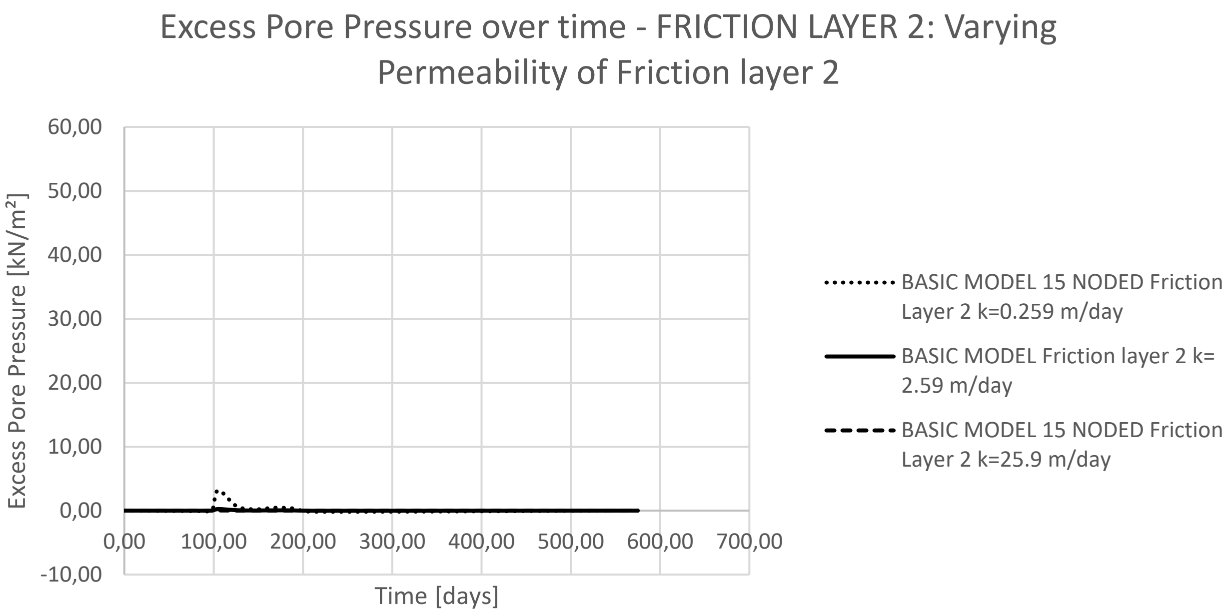

Figure E.10. Excess Pore Pressures plotted for a point in the middle of Friction layer 2, varying the permeability of Friction layer 2.

Finally, were the excess pore pressure plotted over depth for three different phases, in Figure E.11, E.12 and E.13, respectively.

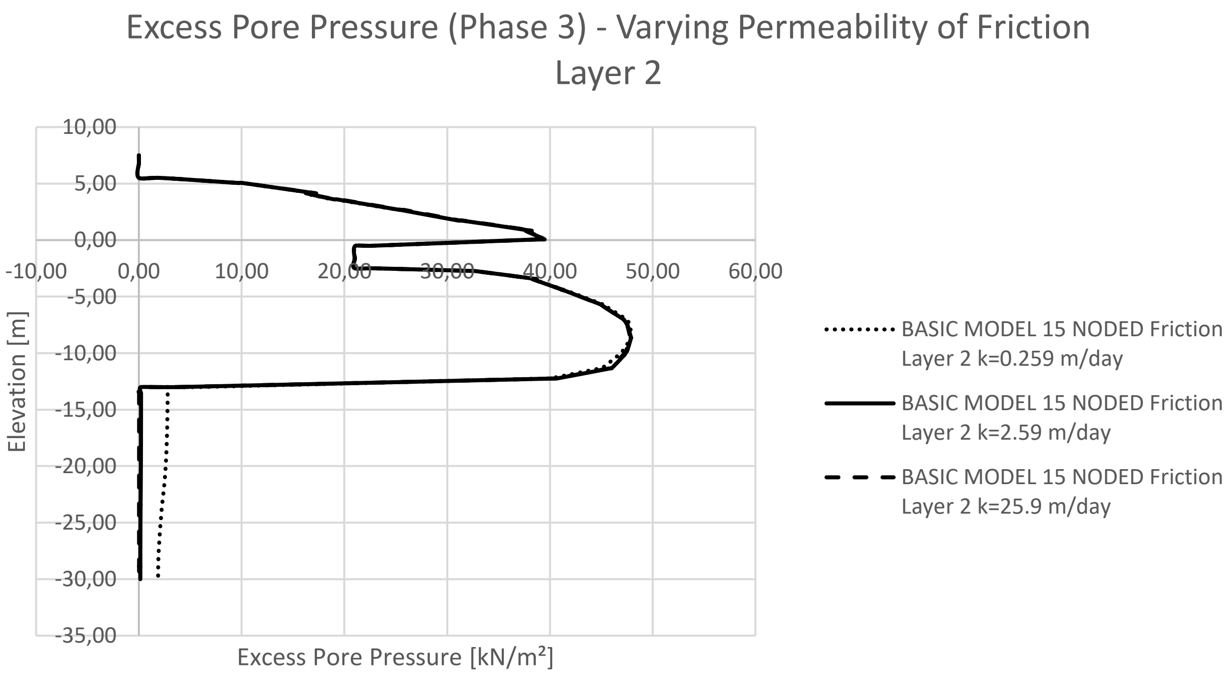

Figure E.11. Excess pore pressure of cross section close to the wall, Phase 3 – Setting Soil to dry inside Excavation.

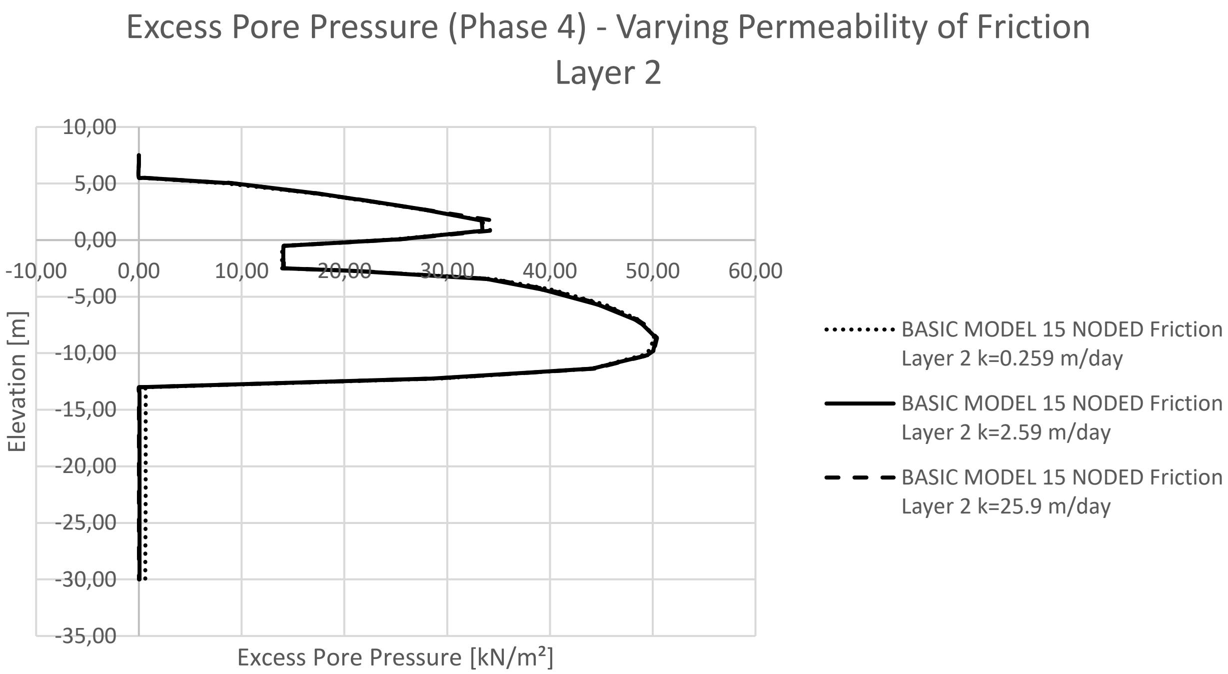

Figure E.12. Excess pore pressure of cross section close to the wall, Phase 4 - Wall Installation.

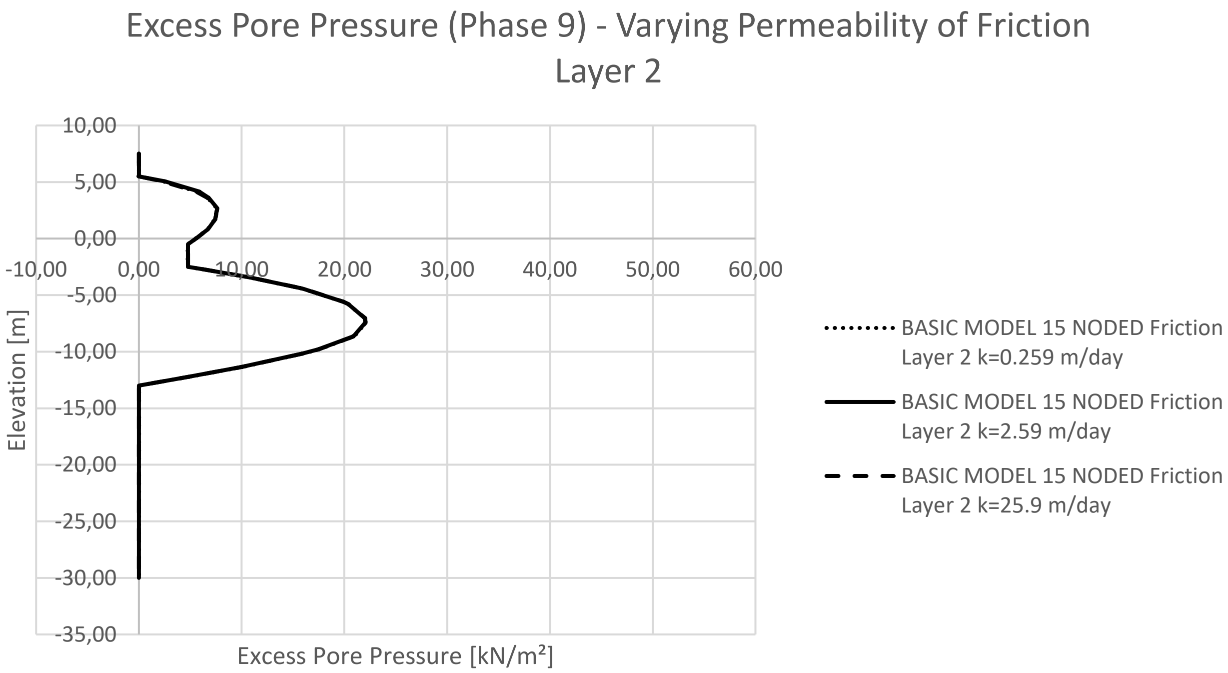

Figure E.13. Excess pore pressure of cross section close to the wall, Phase 9 - Open Excavation.

E.3 Varying the thickness of Friction Layer 1

Heave was plotted in Figure E.14.

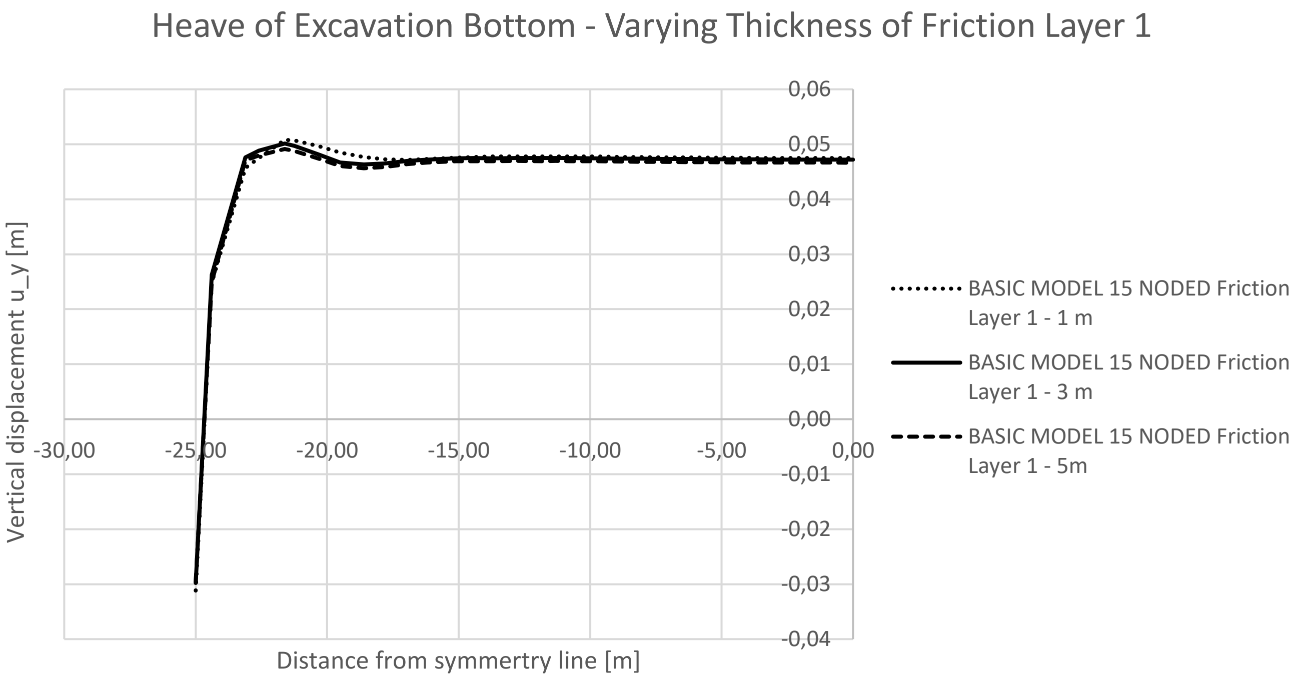

Figure E.14. Heave of excavation bottom plotted with varying thicknesses of Friction layer 1.

Additionally, were the excess pore pressures plotted over time for each layer separately in Figure E.15 to E.18.

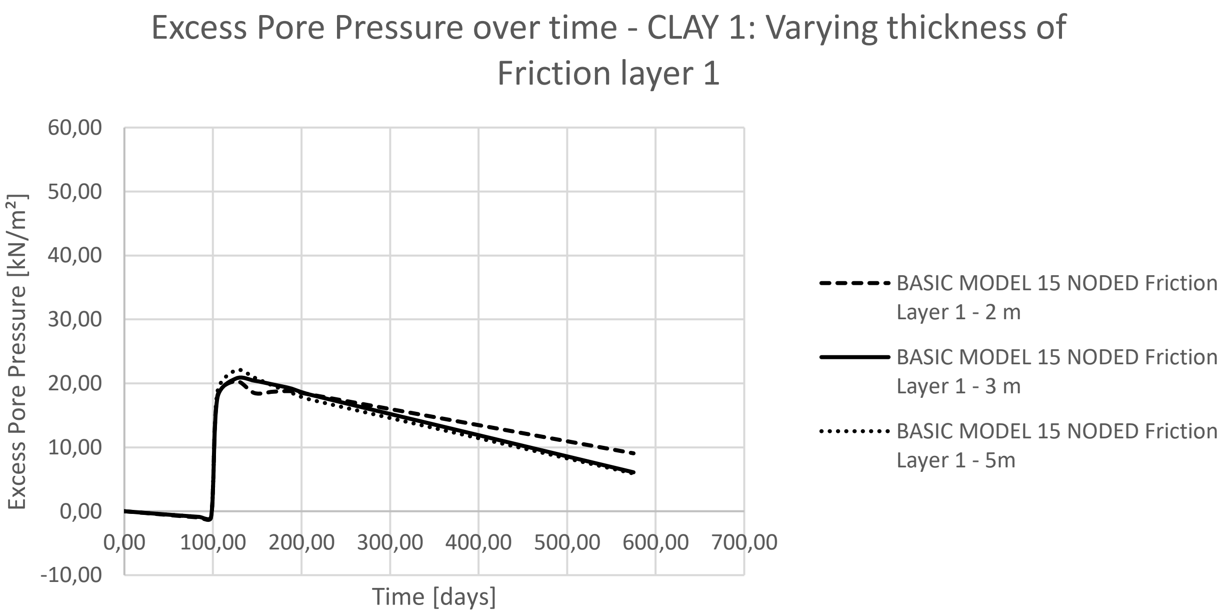

Figure E.15. Excess Pore Pressures plotted for a point in the middle of Clay layer 1, varying the thickness of Friction layer 1.

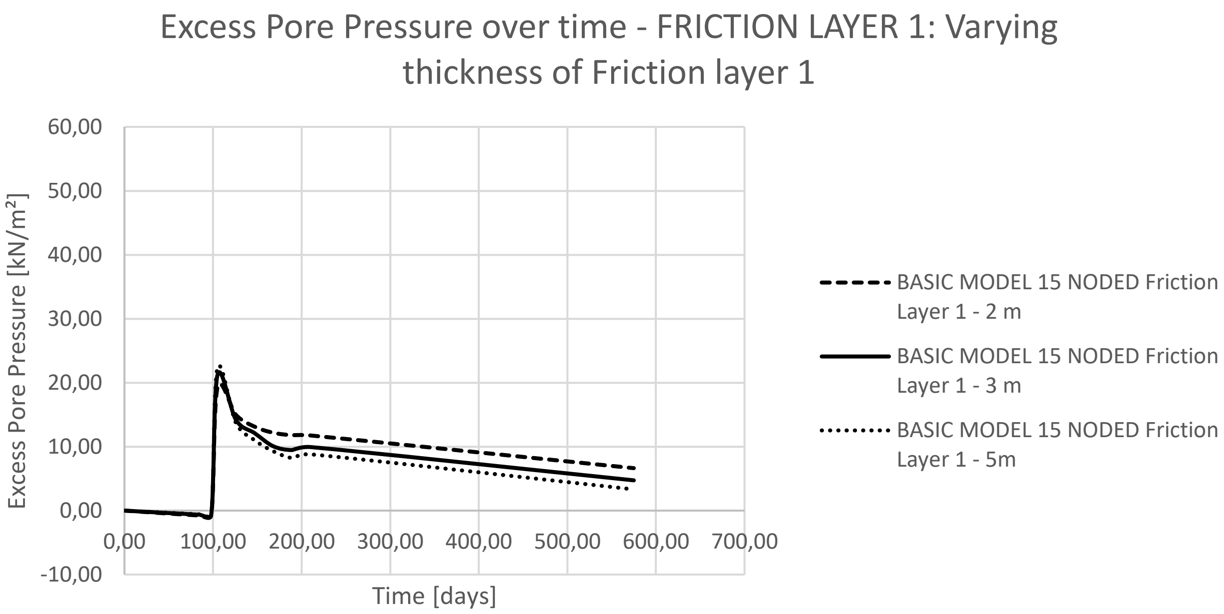

Figure E.16. Excess Pore Pressures plotted for a point in the middle of Friction layer 1, varying the thickness of Friction layer 1.

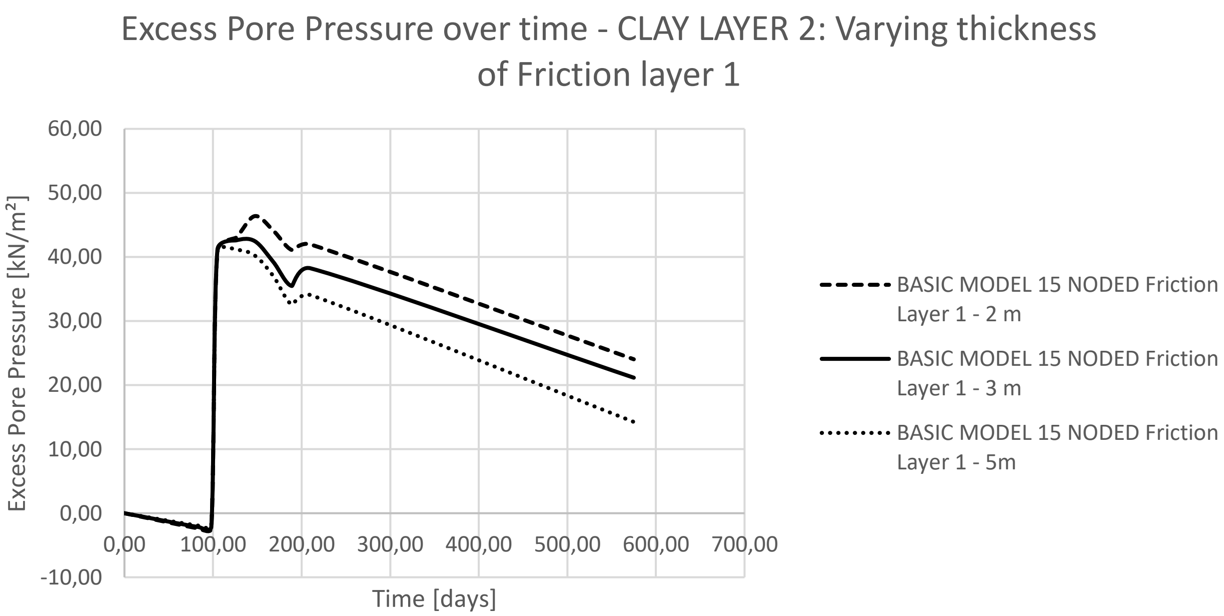

Figure E.17. Excess Pore Pressures plotted for a point in the middle of Clay layer 2, varying the thickness of Friction layer 1.

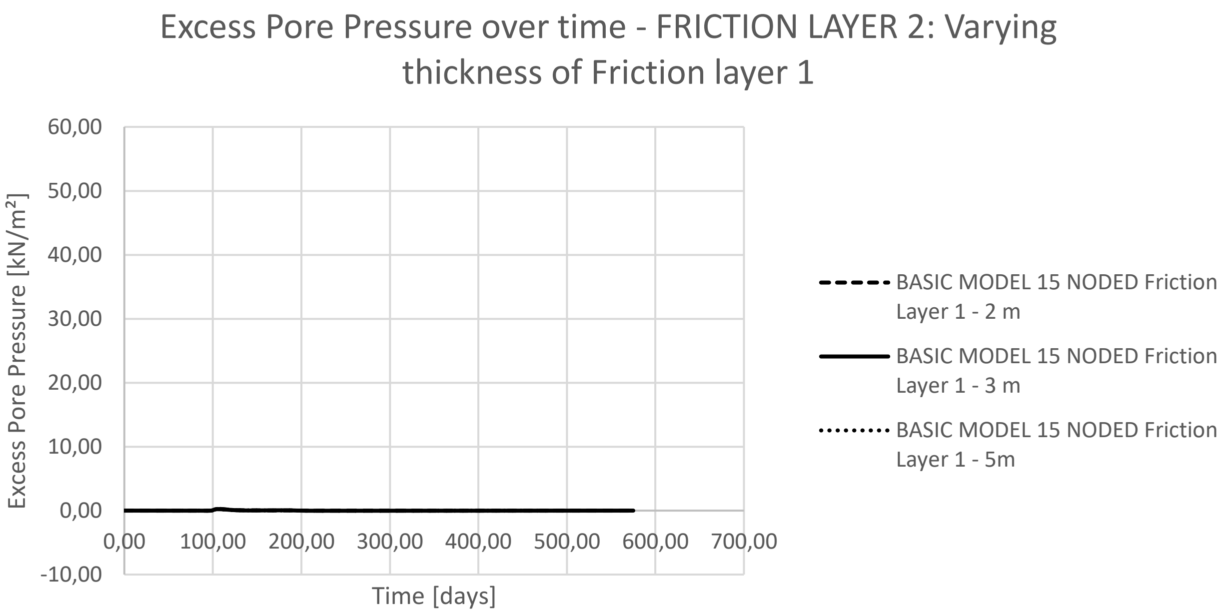

Figure E.18. Excess Pore Pressures plotted for a point in the middle of Friction layer 2, varying the thickness of Friction layer 1.

Lastly, excess pore pressures were plotted over depth in three separate phases in Figure E.19, E.20 and E.21.

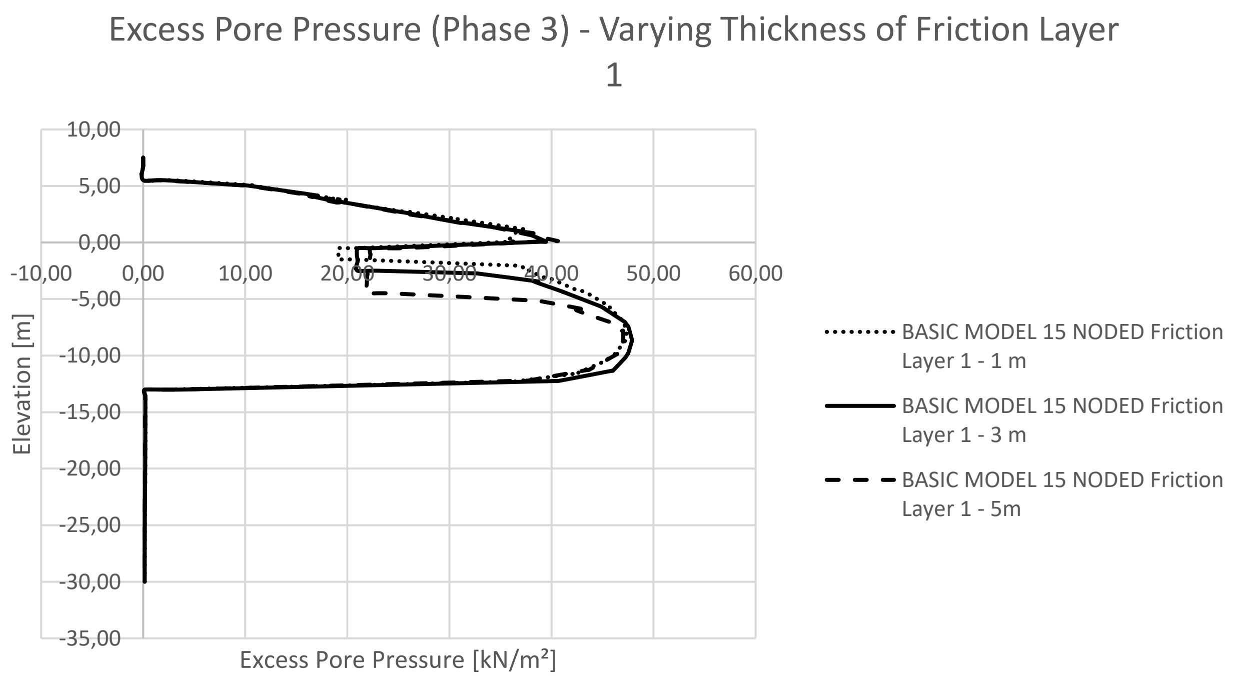

Figure E.19. Excess pore pressure of cross section close to the wall when varying the thickness of Friction layer 1, Phase 3 – Setting Soil to dry inside Excavation.

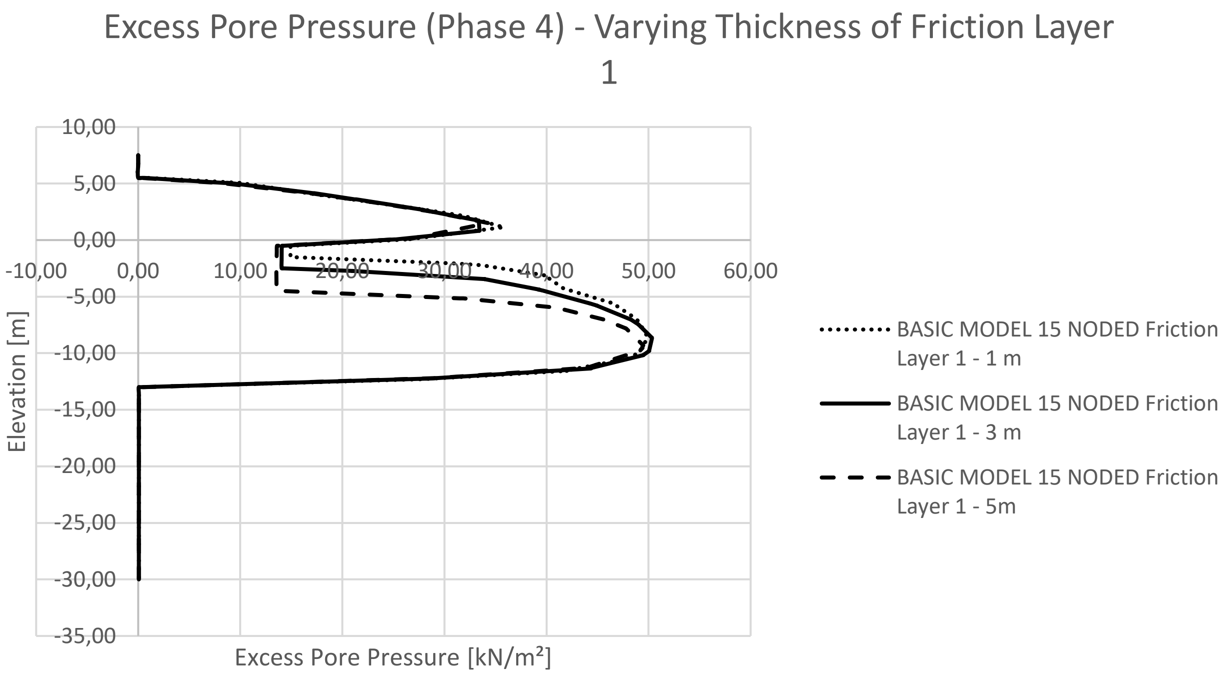

Figure E.20. Excess pore pressure of cross section close to the wall when varying the thickness of Friction layer 1, Phase 4 - Wall Installation.

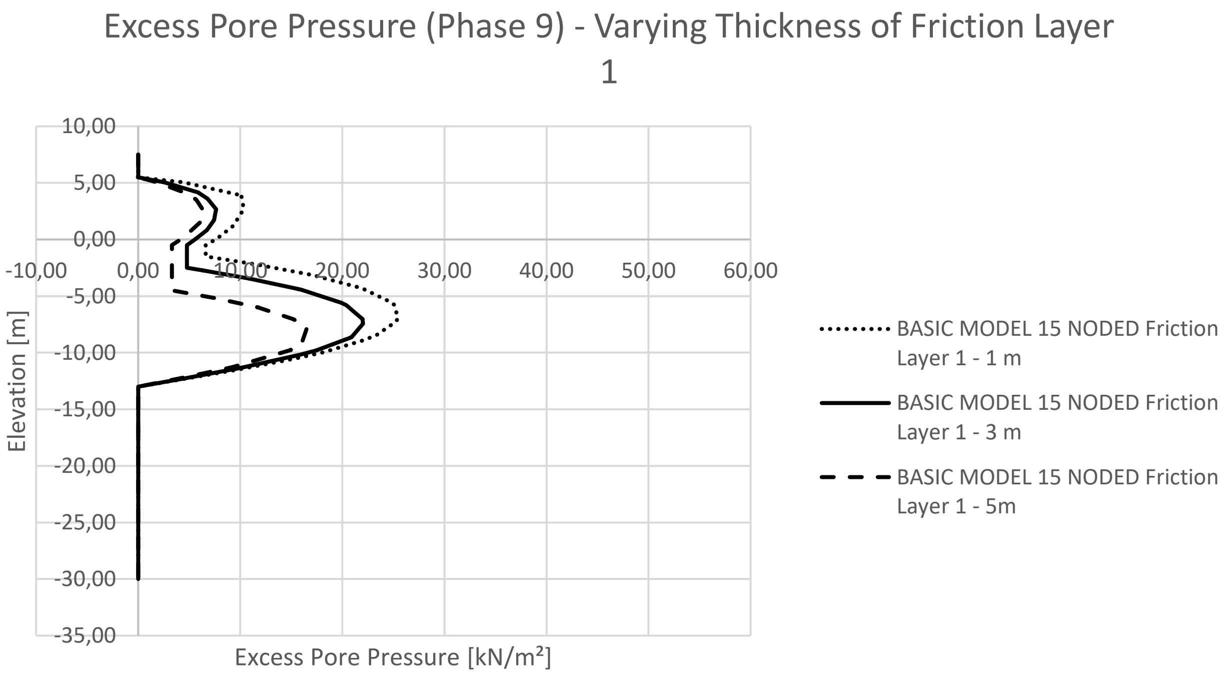

Figure E.21. Excess pore pressure of cross section close to the wall when varying the thickness of Friction layer 1, Phase 9 - Open Excavation.