Effect of pile installation on nearby existing structure

Sanjeev Singh[1], Siddharth Jain[1], Rahul Yadav[2],Vinay Baudh[2],Rahul[2],Himanshu Verma[2],Siddharth Bajpai[2],Abhinandan Singh[2]

[1] Faculty Supervisor, Department of Civil Engineering, KIET Group of Institutions, Delhi-NCR, Ghaziabad.

[2]Undergraduate Scholars, Department of Civil Engineering, KIET Group of Institutions, Delhi-NCR, Ghaziabad

Abstract

Pile driving may generate vibrations that harm neighbouring structures or disrupt nearby residents or activities. However, the ability of humans to perceive vibration is not a reliable indicator of the vibration's potential for harm. It takes careful design, execution, and accommodation of the most sensitive neighbours to reduce the consequences of pile drive vibrations. Both potential genuine damage and litigation based on subjective impression are subject to mitigation. A thorough site inspection is necessary to prevent settlement-related harm. It may be necessary to assess and alert anyone within 400 metres of the claim in order to prevent claims. It is vital to use easily accessible instruments to measure the real magnitude of vibration in order to prevent harm. The most effective technique to control pile driving vibrations is to put into practise a well-thought-out pile driving specification that includes vibration limits, predriving evaluations, vibration monitoring, informational activities, ground elevation surveys for settlement observation, and standard record procedures. Vibrations caused by pile driving can occur with any type of pile driver and any type of driven pile, and the key determining element is the intensity of the vibration.

keywords: Pile installation, vibration due to pile driving, Plaxis 2d

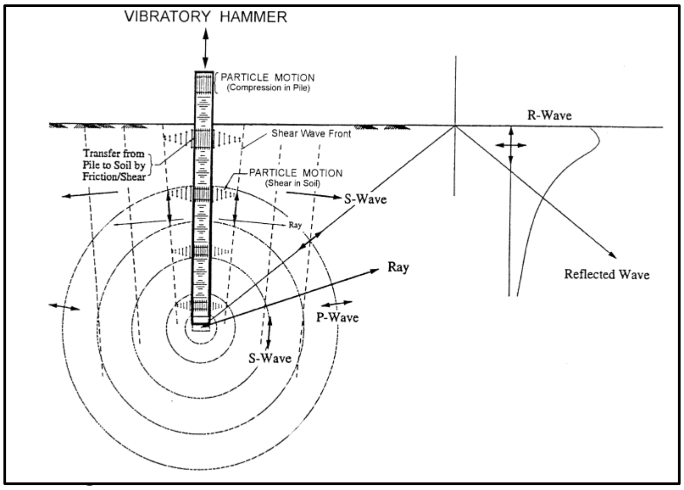
1.Introduction

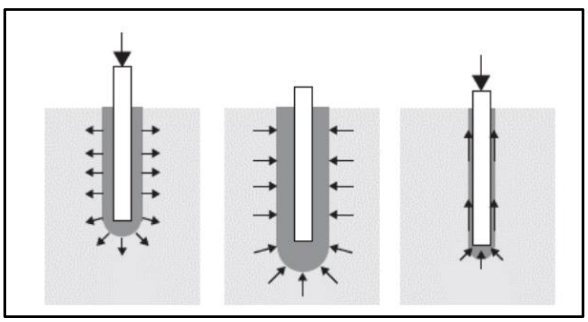
Natural events or human activity both have the potential to cause ground vibrations. Natural ground vibrations are mostly caused by earthquakes and ocean waves. When earthquakes happen, the ground shaking can seriously harm buildings and possibly result in fatalities. All facets of earthquakes and their effects on the ground and soil structures are studied in geotechnical earthquake engineering. Man-made vibrations are produced by humans and range in strength depending on the source. The seismic waves from man-made vibrations interact with constructed structures, potentially producing disturbances and even posing a threat to their integrity, despite having a lesser intensity than earthquake shaking. Soil dynamics research has spent the last two decades examining man-made vibrations and how they affect structures. There are numerous laws, regulations, and standards in place to reduce the harm that groundborne man-made vibrations cause to people and buildings. However, putting these specifications into practise can be challenging and perplexing due to the intricacy of the phenomenon and the numerous connected aspects. To link the findings with recognised structural and behavioural patterns, more field vibration measurements caused by humans are required.

This study gives a general review of man-made vibrations, in particular pile-driving vibrations, and the results of ground vibration measurements coming from vibratory sheet pile driving in metropolitan settings. The ground surface and higher floors of reinforced concrete buildings close to sheet pile drive were used to measure the vibration levels at various distances from the source. The values of the recorded vibration intensity were connected with the impacts on the structural integrity of the nearby structures as well as the requirements of the current codes, standards, and specifications. Additionally, based on the responses of the authors and occupants, the measurement findings were examined in terms of human response and compared to relevant standards.

2.Background

The analysis of ground vibrations relies heavily on understanding the process of energy transmission from the hammer to the pile and eventually to the surrounding soil. This plays a critical role in the entire vibration transmission process. To explain this concept, it can be simplified as follows: when the hammer strikes the pile head, the energy is transmitted to the body of the pile through a compressional body wave. The pile shaft generates S-waves, which propagate conically from the pile. Additionally, P-waves and S-waves are generated in the pile toe and propagate spherically in all directions. When in amplitude proportionate to the square root of distance, as depicted in Fig. 2.1



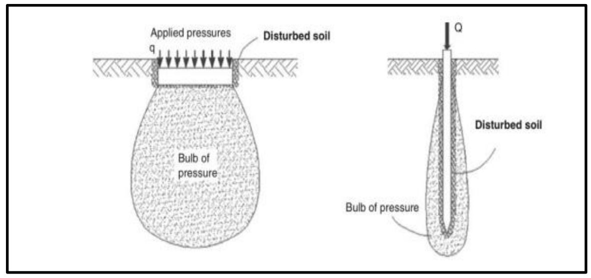


The installation phase involves large strains, mass displacement, soil disturbance, and excess pore water pressure due to rapid changes in void ratio and stresses in the soil, leading to the destruction of the initial soil structure and stress history. During the equalization phase, excess pore pressure induced during installation dissipates, and the soil moves back towards the pile, leading to the recovery of the soil's bearing capacity, influenced by hydraulic conductivity and stiffness of the clay, as well as thixotropic effects and creep. The set-up period for equalization, during which bearing capacity recovers, is usually between three to six months in Gothenburg clay, according to Fellenius (1972). The decrease of void ratio

these waves reach the ground surface, a portion of them is transformed into R-waves, resulting in a decrease

<u>Pile installation and its effect on surrounding soil</u>

Deep foundations are being studied, which are one of two types of foundations, the other being shallow foundations, as shown in Figure 1.1.



Deep foundations are characterized by their extension to a large depth into the ground while occupying a small area in plan. Piles are the most common type of deep foundation, as stated by Knappett and Craig (2012). During the lifetime of a pile, there are three phases: installation, equalization, and loading, illustrated in Figure 1.2, according to Ottolini et al. (2014).

in the clay adjacent to the pile shaft leads to an increase in undrained shear strength. In the loading phase, the load on the pile head is transmitted through the pile into the soil, with the pile's bearing capacity being proportional to the interface friction angle at the pile-soil interface and the normal effective stresses, according to Randolph et al. (2011).

<u>Pile types and installation techniques</u>

The type of pile and piling method used can have varying effects on the displacement of soil mass [Knappett and Craig, 2012]. Soil displacement occurs when the volume of soil pushed aside by the pile is equivalent to the volume of the pile installed [Olsson and Holm, 1993]. To mitigate soil displacement, the soil can be removed through drilling, a technique known as pre-augering.

There are different piling techniques available, with driven and bored piles being the two main categories [Fleming et al., 2008]. The choice of piling method depends on various factors such as soil characteristics, groundwater conditions, and potential consequences of soil displacement. Driven piles can be further categorized into drop hammer, explosive, vibration, and jacking against a reaction. On the other hand, bored piles are installed by rotary augering machines, which remove soil and cause no soil displacement. In cases where the pile head is out of reach from the piling machine, a pile-block can be used to extend the pile head area. Once the pile is installed, the pile-block is removed, leaving a cavity where soil can fall back and reduce the amount of displaced soil.

Piles can be made from various materials, including concrete, steel, and wood [Fleming et al., 2008]. Piles can also be classified as displacement or non-displacement piles. Displacement piles have a larger cross-sectional area, causing more soil displacement upon installation, while non-displacement piles have smaller cross-sectional areas or hollow cross-sections, resulting in less soil displacement. The chosen pile type in this thesis is precast concrete piles, which are displacement piles and the most common pile type in Sweden [Edstam, 2011].

Moreover, piles can be end-bearing or floating. End-bearing piles rely on the pile toe resting on a solid bedrock or a stiff soil layer with adequate strength. Floating piles, on the other hand, rely on adhesion between the soil and pile shaft, which can be friction or cohesion, depending on the soil type. In this thesis, floating cohesion piles are studied.

Soil mechanics interpreted with constitutive models

Soil is a complex material composed of three main components: grains, water, and air in the voids. When subjected to stress and stress change, such as during pile installation, the behavior of soil can be highly non-linear, anisotropic, and time-dependent. Different constitutive soil models can be used to capture this behavior, but their accuracy depends on the complexity of the model and the amount of input data available. Basic field tests often provide limited data, making it difficult to select all the necessary parameters for more advanced models. The simplest constitutive models assume that the soil behaves linear-elastically and only shows reversible deformations, which implies that the soil is infinitely strong. However, this assumption is not accurate since soil is a highly non-linear function of shear strain and effective confining stress. To model the failure of soil, the Mohr-Coulomb failure criterion can be used, which is based on the assumption that soil is a frictional material exposed to three-dimensional states of stress. This model is sometimes called the linear elastic perfectly plastic model because the soil is assumed to be elastic until it reaches a defined failure condition. To capture the undrained response of soil, which is necessary when studying the short-term response of soft clays, the model must be adapted since the shear strength of the soil is different compared to drained conditions. The Soft Soil (SS) model and Soft Soil Creep (SSC) model are two additional models that adapt the Mohr-Coulomb failure criterion. Both models consider the elastic and plastic deformations of the soil, reversible and irreversible deformations, in contrast to the MC model. The SSC model further aims to capture the secondary consolidation, creep, and is a development of the SS model. However, both models have drawbacks, such as assuming soil is isotropic and falsely modeling creep strains, which can lead to inaccurate predictions of soil strength.

Empirical and analytical calculation methods

To estimate and forecast the soil's reaction to pile installation, such as mass displacement, excess pore pressure increment, and strain response, various computation techniques can be employed, including empirical, analytical, and numerical methods.

Rhenman's method

The use of empirical calculations for predicting soil behavior due to pile installation relies on observations made in the field and laboratory, but they do not account for small variations that may occur, resulting in rough predictions at best. Rhenman's method is an example of an empirical calculation method used to estimate mass displacement. However, this method makes several simplifications and assumptions, such as assuming that the vertical mass displacement of the ground surface occurs within an area of one pile length away from the piling area, assuming the ground surface is horizontal, and assuming that the volume of vertical mass displacement is proportional to the volume of the piles installed. Equation 2.1 is used to calculate the vertical mass displacement of the ground surface.

$$x = \eta(V_p - \Delta V_{pr}) / d[(\alpha + \beta)(l^2 + d^3) + (\gamma + \delta)(b^2 + d^3) + bl\,d]$$

Where: x = Mass displacement within the piling area

d = Depth of the pile below the ground surface or excavation bottom

b = Width of the piling area

l = Length of the piling area

η = Heave factor, ranging between 0.5 to 1.0, often 0.75

V_p = Pile volume

ΔV_{pr} = Volume of removed clay with pre-augering

$\alpha, \beta, \gamma, \delta$ = Relative weight of constructions A, B, C and D, account for soil behavior. It relies heavily on volumetric ratios and geometry, and thus, is suitable for quick and approximate predictions. However, if greater accuracy and insight into additional soil processes is required, the method cannot be applied.

Shallow Strain Path Method and Cavity Expansion Method

Various analytical methods are available for modeling and predicting the displacement and soil disturbance caused by piling. The cavity expansion method (CEM) and the strain path method (SPM) are two such methods. The Sagaseta's method, based on SPM, is known as the shallow strain path method (SSPM). SPM assumes that soil deformations and strains caused by deep pile installation in undrained clays are independent of shear strength. The method models the mass displacement that occurs due to the irrational flow of an ideal fluid. However, this assumption ignores that different soils can have highly different penetration resistance and soil stresses. Empirical evidence shows a link between pile driving and ground heave, whereas SPM analysis of pile penetration calculates that all soil elements undergo a net downward movement. Therefore, SPM is suitable for calculating strains near the pile toe but not for far-field conditions where the ground surface can affect soil deformations. The SSPM analysis addresses these limitations by incorporating the effects of a stress-free ground surface.

SSPM calculates mass displacement in four steps, simulating the installation of a single pile in three steps. The results from these simulations are then combined for a final analysis of the mass displacement. The first step involves assuming a point source (S) that penetrates the soil throughout the pile length and discharges an ideal fluid throughout the penetration, creating a spherical flow that causes mass displacement. The second step introduces a mirror image sink (S') that moves in the opposite direction, canceling out normal stresses while doubling shear stress. However, this does not correspond to an unloaded ground surface. To counteract this, a set of corrective radial shear forces is added in the third step. The method is The method of calculation focuses solely on the mass displacement of the ground surface, and does not suitable for predicting floating cohesion piles in deep deposits of clay but cannot be applied to end-bearing piles or floating cohesion piles driven close to the bedrock, as the displacements will behave differently.

CEM studies the soil's reaction to pile installation through the use of an expanding or contracting cylindrical cavity, instead of an ideal fluid as in SPM and SSPM. When using the CEM, constitutive models such as linear elastic (LE), elastic-perfectly plastic, or strain hardening/softening plasticity can be used to model the spherical or cylindrical cavity. LE models assume the soil has infinite strength, while elastic-perfectly plastic models have constant strength during both loading and unloading, not considering the variation of the soil's strength due to deformation history. As pile installation is a rapid process, and soft clays have low permeability, the undrained response of the soil is modeled. The CEM and SPM can be applied with numerical methods, which are necessary when advanced constitutive material models are used, due to the complexity of the problem.

Numerical calculation methods

Various numerical calculation methods in combination with constitutive soil models can be used to predict the response of the soil due to pile installation. Numerical calculation methods use algorithms and the discretization of continua which allows for calculating non-linear and time-dependent material behavior, arbitrary geometries, initial or in situ conditions, multiphase media, and different types of loading such as static and cyclic loading. Numerical modelling can be used for geotechnical problems, such as pile installation and mass displacement, which can be complex to solve using empirical or analytic methods. The finite element method, boundary element method, discrete element method, and finite difference method are some numerical methods that can be used for these problems. PLAXIS 2D and PLAXIS 3D are two FEM-based software programs that can be used for the modeling of geotechnical problems, such as mass displacement. Analytical calculations can also be adapted with numerical methods, such as SPM and CEM.

In PLAXIS 2D, either a plane strain or an axisymmetric model can be used, where the main difference between the two models is the geometry. The plane strain assumption should be used for models with a uniform cross-section, whereas the axisymmetric model is used for circular structures. Both models have advantages and disadvantages when modeling pile installation. A prescribed line displacement can be used to model pile installation with the use of CEM, where the prescribed line displacement function is called in PLAXIS. The expansion of a soil cluster in PLAXIS can also be modeled through volumetric expansion by using the function volumetric strain. The prescribed line displacement is more numerically stable than volumetric expansion.

Prediction of mass displacement due to pile installation, using PLAXIS 2D

PLAXIS 2D and PLAXIS 3D offer various methods for modeling pile installation, but none have been fully established or verified. In order to determine the most effective method for modeling pile installation, different approaches were evaluated in PLAXIS 2D, in line with the objective of the thesis. All models were created with a pre-installed pile in the soil, which was then expanded using various functions. The pile was either a cavity or consisted of a linear elastic material, and the models were based on the CEM. The impact of pile installation disturbances, such as installation technique, was not examined; only the volume change in the soil caused by the pile installation was studied. Linear-Elastic, Mohr-Coulomb, Soft Soil, and Soft Soil Creep were the constitutive models used. The reason for testing several constitutive models was that simpler models like LE and MC could handle greater deformations before collapse, so experiments began with them before moving on to more advanced models.

Investigation and chosen modelling technique

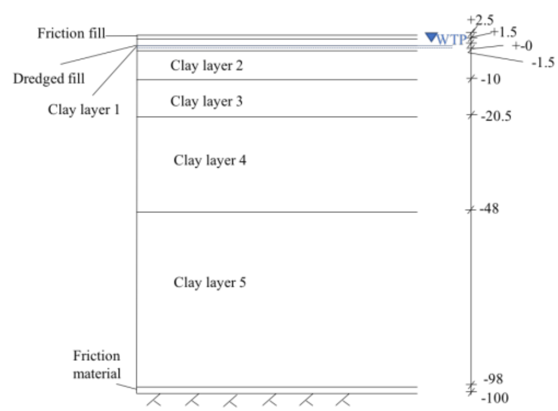
A variety of models were created in PLAXIS to determine the optimal model and modelling technique for the calculations. An axisymmetric model was the first to be constructed, with a general soil profile and model parameters, and three different versions were used to model mass displacement. The initial model contained a cavity with a prescribed line displacement, but it had issues with soil collapse and inaccurate mass displacement distribution due to tugging in the nodes. Additionally, there was no interaction between the soil and the pile. A soil cluster with a LE material was then modelled to address these limitations, which allowed for interaction between the soil and the pile, resulting in smoother deformation of the mesh and more plausible mass displacement.

A plane strain model was also constructed with a general soil profile, which allowed for more than one pile to be modelled. The influence of the pile order was investigated using prescribed line displacement and volumetric strain, and three soil clusters were constructed where the prescribed line displacement or volumetric strain were activated upon. One pile was activated at a time, and different measures were implemented to counteract problems like deformed piles before activation. Some unsymmetrical deformations that could be linked to the mesh as well as the pile order were observed, and volumetric strain was deemed uncertain due to overruled parameters in the model.

Prescribed line displacement was then investigated, and a plane strain model with a superpile and prescribed line displacement was chosen as the optimal model technique. The use of prescribed line displacement in the horizontal direction with free or fixed movement in the vertical direction, with or without interfaces, allowed for the interaction between the surrounding soil and the pile to be somewhat captured. Deformation differences resulting from individual piles and pile order were negligible, and displacement was more numerically stable than volumetric strain. The chosen model technique was compatible with unsymmetrical geometries, and the scenarios investigated in the Thesis corresponded well with the plane strain assumption.

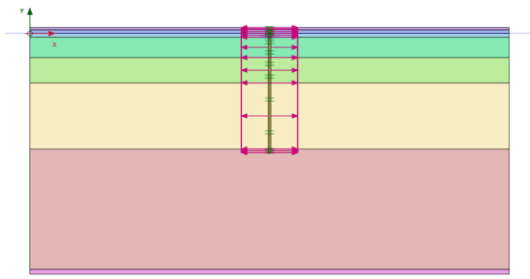
Construction of Base Model in PLAXIS 2D

A 15-node SS model was used to construct a Base Model with a plane strain model. Figure 3.3 shows the chosen soil profile, starting with a layer of friction material at level +2.5 meters, followed by a layer of dredged material at level +1.5 meters, five clay layers with different sets of model parameters, and a final layer of friction material before the bedrock.



At a depth of 20.5 meters, there are two different geological deposits between clay layer 3 and 4 with significantly different model parameters, including κ ∗, λ ∗, OCR, effective cohesion (c 0), and effective friction angle (φ 0 c).

The Base Model had a horizontal ground surface, was 200 meters wide, and contained a superpile structure in the center, as shown in Figure 3.4.



The prescribed line displacement was added to the soil cluster of the superpile, which had a width of one meter from the beginning. The pile was not included in the Base Model due to variations in the superpile design depending on the pile design. Both sides of the superpile were modeled to allow for different unsymmetrical cases. A borehole was placed at origo, with the layers of fill, clay, and friction material, and the water table was set to level ±0, the top of the first clay layer. The horizontal boundaries (xmin and xmax) were closed for ground water flow in the clay layers, while seepage was allowed in the friction material and fill. The minimum vertical boundary (ymin) was also closed for groundwater flows, and the maximum vertical boundary (ymax) was open. The bedrock was not initially closed for groundwater flow, as it typically consists of crushed or cracked areas. However, since a friction material layer allowing for groundwater flow was placed on top of the bedrock, this was deemed a valid assumption.

A coarseness factor of 0.5 was applied to the model to construct the mesh of the Base Model, which was then refined to produce a quality mesh. Two elements had a size under 0.5, and 13 elements were under 0.6, resulting in a symmetrical mesh without excessive calculation time. The mesh was capable of accommodating large strains when activating the prescribed line displacement. A finer mesh could lead to model collapse, as smaller deformations would be allowed. To test the model's ability to handle deformations, stresses, and strains due to mass displacement, the OCR was set to 10 for all clay layers, making the model behave falsely Linear Elasto-Plastic. This was done to determine if the model could handle the deformations before using the chosen OCR values for SS and SSC.

Calculation of prescribed line displacement

The amount of prescribed line displacement was determined based on the number of piles installed. The assumption made was that there would be no volume change in the soil during pile installation due to the clay's low permeability. Therefore, the volume of piles installed was considered to be equal to the mass displacement. The equation used for this calculation took into account the piling area, number of pile rows, and number of piles, among other factors. The reason for calculating the cross-sectional area of the piles and piling area from above, instead of the volume of the piles and piling area, was that the ratios were the same, given that the pile and piling areas' depth and height were equal. Figure 3.5 provides a schematic illustration of a typical piling area viewed from above.

The equation used to calculate the prescribed line displacement can be seen in Equation 3.1

$$\textit{Prescribed line displacement} = \frac{(w_{sp} * \sqrt{(\frac{a_{sp}}{a_{pa}} + 1)} - w_{sp}}{2}$$

Where: wsp - width of superpile; number of pile rows ∗ pile width

asp - area of superpile; (pile diameter) 2 ∗ number of piles

apa - piling area; wsp ∗ length of piling area

The calculation involved determining the ratio of the area of the superpile to the total piling area, including the soil. This allowed for the calculation of the percentage of piles relative to the total area, assuming that the area increase would increase the total percentage calculated. Additionally, it was assumed that each pile would increase uniformly in all directions. Since the investigation was based on a two-dimensional scenario, where the mass displacement could only occur in two directions, the square root of the percentage was taken to obtain the percentage increase for each side of the pile, which was then multiplied by the width of the superpile. From this, the width of the superpile (i.e., the number of pile rows installed in that step) was subtracted to obtain the total prescribed line displacement required for that step in the model. Since the prescribed line displacement was applied in two directions, the total prescribed line displacement was divided by two.

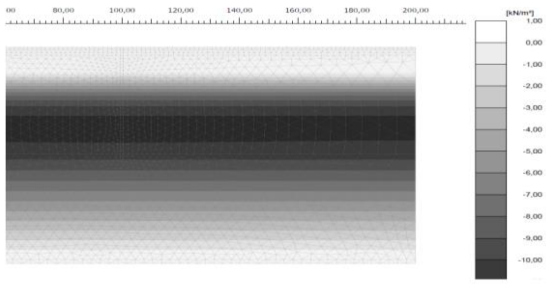
Calculation Phase	Procedure	Activated structures
Inital phase	K0	Generates the initial stresses, the two fill layers were deactivated. No structures were activated.
Phase 1	Plastic	Activation of the dredged fill.
Phase 2	Consolidation	Consolidation for 10 years, letting the excess pore pressure from the activation of the fill dissipate.
Phase 3	Plastic	Activation of the friction fill.
Phase 4	Consolidation	Consolidation for 10 years, letting the excess pore pressure from the activation of the fills dissipate.
Phase 5	Consolidation	Consolidation for 178 years, until present day, letting the excess pore pressure from the activation of the fills dissipate.
Phase 6	Plastic	No additional structures were activated, the displacements that had occured during the previous steps were reset to zero. The nil-step furthermore made the stress field be in equilibrium, and made the stresses obey the failure condition.
Phase 7	Consolidation	Consolidation for one year in order to be able to validate the excess pore pressure and settlements with values measured in the area.

<u>Calculation phases</u>

The Base Model underwent several calculation phases, where the pore pressure was set to phreatic and suction was disregarded. The mesh was updated in each phase and various structures were either enabled or disabled, as indicated in Table 3.1. In general, the plastic procedures corresponded to undrained calculations and responses, while the consolidation phases corresponded to drained calculations and responses.

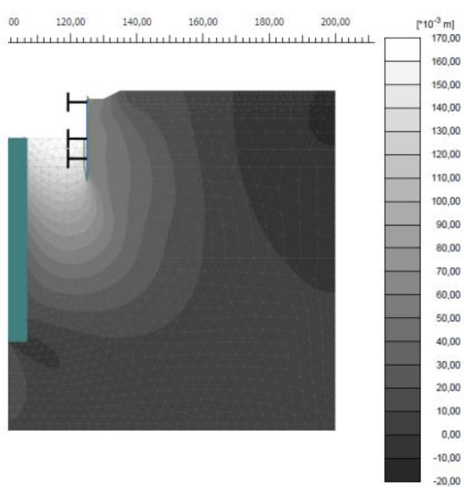
The purpose of modeling the previous soil history, even though the model parameters are based on the current soil, is to determine the stress and strain state of the soil, which is a result of the soil's history. Additionally, excess pore pressure is also generated by modeling the soil history. However, after the Initial Phase, there is no excess pore pressure in the model, and the nil-step merely resets the displacements without affecting the stresses, strains, or pore pressures.

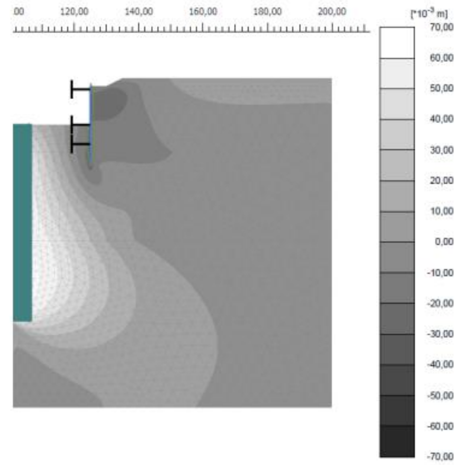
To verify the accuracy of the Base Model prior to conducting calculations, the excess pore pressures in the model during the control year and the settlements observed in the field were compared. The settlement rate in the area surrounding Gothenburg Central Station is no more than two millimeters per year, according to Wood (2014). The model produced a vertical settlement of approximately 1.5 millimeters, which means that the settlement rate was accurate since the model was able to capture the behavior. The excess pore pressure in the area varies due to the fill's placement, as shown in Appendix A, Figure A.7. The pore pressures generated by the Base Model were compared to those measured in the field. The maximum excess pore pressure in the Base Model occurred around level -30 to -42 meters, with a magnitude of 12 kPa, which was reasonably consistent with the measured excess pore pressures at that depth in the area (see Appendix A, Figure A.7). This validation was not possible for the SSC model since the model generates significant pore pressures to simulate creep, resulting in excessive excess pore pressures during the control year.



Result and analysis

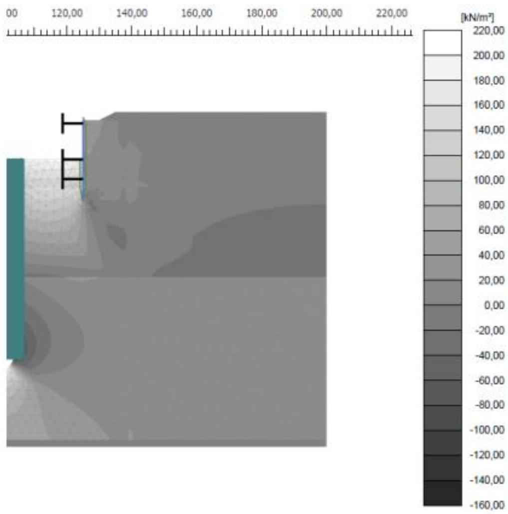
The maximum vertical mass displacement varied among the three models after the final phase of pile installation. In Model 1, it was approximately 16.5 centimeters, while in Models 2 and 3, it was approximately seven centimeters, with Model 3 having a slightly lower vertical mass displacement of eight millimeters. The maximum vertical mass displacement occurred within the support structures for all three models, either on the excavation bottom, as for Model 1, or at level ±0 meter in Models 2 and 3. In Models 2 and 3, an increase of approximately seven centimeters in the maximal vertical mass displacement was observed directly after the excavation was carried out, which was linked to the bottom heave occurring in the excavation when the soil was removed.



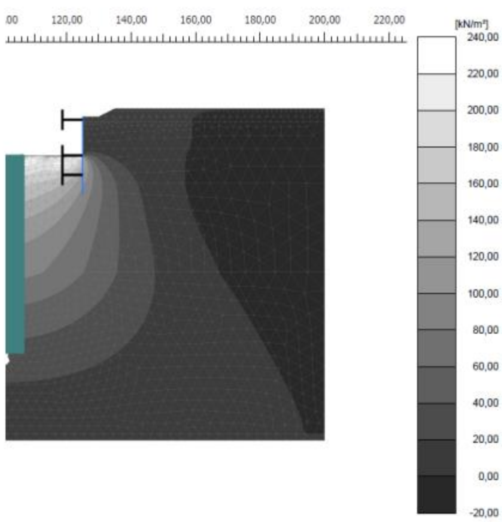


During the pile installation from the excavation bottom, the maximum vertical mass displacement was approximately six centimeters higher than when the pile installation occurred before the excavation. This difference may be due to the fact that in Model 1, there was less load present within the support structures and that bottom heave of the excavation bottom had already started to occur due to the unloading. The distribution of the vertical mass displacement differed distinctly between Model 1, Model 2, and Model 3, with Model 1 resulting in a distribution that subsided faster towards the boundaries in the horizontal direction.

The maximum horizontal mass displacement was approximately seven centimeters for all three models directly after the final pile installation phase, and it occurred adjacent to the superpile, approximately at level -50 meters. The distribution of horizontal mass displacement throughout the soil varied between Model 1 and the other models, which were nearly identical, with Model 2 having horizontal mass displacement up to level ±0 meters in the model. In Model 1, horizontal mass displacement occurred towards the superpile from outside the excavation, which was a consequence of the support structures bending due to the large horizontal mass displacements occurring within the support under the excavation bottom.



The largest excess pore pressures directly after the last pile installation phase occurred adjacent to the superpile for all three models, but they wandered away from the superpile and could be found at the boundaries in the horizontal direction after 80 years, according to Figure 5.7.



Conclusion and further investigations

The purpose of the thesis was to examine how the installation of precast concrete displacement piles impacts neighboring areas, specifically in terms of soil mass displacement and the generation of excess pore water pressures. The study focused on the Västlänken project's Central Station, using a numerical plane strain cavity expansion method with a 2D Finite Element code and a suitable constitutive model for soft soils that was validated against laboratory and field data. The validation process used nearby projects, including Partihallsbron support and lowering of the E45 highway. The thesis analyzed the effect of time and construction sequence on the Västlänken project and concluded that it's possible to make reasonable short-term predictions of mass displacements using the proposed method. However, the modeling results are most sensitive to soil stiffness, especially the unloading/reloading stiffness. The study found that pile installation from the excavation bottom results in larger magnitudes of mass displacement and shorter drainage lengths due to larger local pore pressures generated. Still, pile installation before constructing the building pit can significantly reduce magnitudes of vertical displacement close to the piling area. The thesis also found that excess pore pressures from pile installation dissipate over time, and the maximum pore pressures reduced to those in the control year only after 80 years of consolidation. Finally, the study found that mass displacement and stresses generated in the soil during pile installation lead to deformation of adjacent constructions such as support structures, affecting additional deformations behind the wall. The study suggests investigating the validity of the derived equation for the prescribed line displacement in combination with an improved constitutive model and evaluating excess pore pressure with consideration to soil homogeneity.

References

- [GILA EDVARDSSON PAULA MELIN-NYHOLM, (2018)] The influence pile installation with precast concrete displacement piles has on adjacent areas
- [Dungca J.R., Acosta D.Y(2016)] THE PROPAGATION BEHAVIOR OF PILE-DRIVING-INDUCED VIBRATION DONE ON SOIL AT VARYING DISTANCES AND ITS EFFECTS ON EXISTING STRUCTURES
- [G.A. Athanasopoulos*, P.C. Pelekis,(2000)] Ground vibrations from sheetpile driving in urban environment: measurements, analysis and effects on buildings and occupants
- [Woods, R.D. (1997)] Dynamic Effects of Pile Installations on Adjacent Structures. NCHRP Synthesis 253, National Cooperative Highway Research Program, Transportation Research Board, National Academy Press, Washington D.C., USA.

- [Adhilla Ainun Musir1,(2013)]PILE DRIVING EFFECTS ON NEARBY BUILDING.