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FINITE ELEMENT ANALYSIS OF SIGNIFICANT GROUND DEFORMATION DUE TO PILE DRIVING IN SILTS

Christoph Schulze
University of Rhode Island, mail@schulze-christoph.de

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**FINITE ELEMENT ANALYSIS OF SIGNIFICANT GROUND
DEFORMATION DUE TO PILE DRIVING IN SILTS**

BY

CHRISTOPH SCHULZE

**A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE
REQUIREMENTS FOR THE DEGREE OF**

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MASTER OF SCIENCE THESIS
OF
CHRISTOPH SCHULZE

APPROVED:

Thesis Committee:

Major Professor: Christopher D.P. Baxter
Aaron S. Bradshaw
David G. Taggart
Nasser H. Zawia

DEAN OF THE GRADUATE SCHOOL

UNIVERSITY OF RHODE ISLAND

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ABSTRACT

Movement of adjacent ground and support-of-excavation structures due to pile driving in non-plastic silts is a significant issue in urban areas of Rhode Island. There have been several cases in which such movements have damaged historic structures and transportation infrastructure. The objective of this research is to perform a finite element analysis of a particular case study involving movement of a sheetpile wall-supported excavation due to the excavation and pile driving activities.

The case study involved construction of a pile-supported gate and screening structure that is part of the combined sewer overflow project by the Narragansett Bay Commission. The structure was built by first driving sheetpiles around the site, then excavating in stages to the desired elevation, and then driving piles at the base of the excavation. Geotechnical instrumentation at the site included three inclinometers located behind the sheetpile walls and two piezometers in the excavation. Deformations of the wall were observed during each stage of excavation. Additional significant movements of the wall and elevated pore pressures were measured during pile driving.

A 2-Dimensional finite element analyses was performed to model the deformation of the sheetpile walls using the commercial software PLAXIS version 7. Soils at the site were modeled with either a linear elastic, perfectly plastic Mohr-Coulomb constitutive model or a non-linear hyperbolic model. The excavation sequence was taken from construction records and simulated directly by removing soil in the model. Properties of the soils (strength and stiffness) were varied around values from the literature until the predicted wall movements matched observations.

There was good agreement between the modeled displacements and observations for the first two stages of excavation using reasonable values of strength and stiffness for Rhode Island silts. These parameters would be a good place to start in future modeling efforts involving support of excavation projects in Rhode Island

The only way to simulate the last stages of excavation and wall displacement was to use unreasonably low values of strength and stiffness. Possible explanations for this poor agreement include: a) loss of ground during pumping reduced the stability in the excavation and led to larger movements; b) the excavation caused significant disturbance (almost liquefaction) of the soil at the base of the excavation; and c) the soil surrounding the inclinometer tubes behind the wall moved or became disturbed and the measured movements are not representative of the actual wall movements.

Dynamic loading of the soil from pile driving could not be directly modeled within PLAXIS. Therefore, the effects of pile driving were modeled by reducing the strength and stiffness of the underlying silts to simulate disturbance and possible liquefaction. Again, the properties of the soil were reduced until the predicted movements match field observations. Although this ignores the fact that the actual process is at least partially undrained, the approach used in this thesis is a first step in understanding movement of adjacent structures in Rhode Island silts due to pile driving.

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1. INTRODUCTION

Non-plastic Rhode Island Silts are very sensitive to construction activities such as pile driving or excavation processes. Vibrations caused by these construction activities can cause pore pressure generation and lateral deformations. This leads to temporary reductions in effective stress and can ultimately lead to a decrease of soil strength and stiffness (Idriss and Boulanger, 2008). In some cases, this can lead to liquefaction of the soil.

Several instances of movement of adjacent ground and cracks in nearby structures have been recorded on Rhode Island construction sites (Davis, 2004; Bradshaw et al., 2007; Trautman, 2009; Taylor, 2011). Unfortunately, the effects of pile driving on the properties of silts is difficult to estimate apriori and contractors and engineers dealing with Rhode Island Silts collect in-situ data with inclinometers and piezometers during construction to monitor these effects. These measurements are useful for determining when problems are occurring, but it would be beneficial to have a method for estimating these problems beforehand.

Such measurements were taken at one construction site in Providence, Rhode Island for a near surface gate and screening structure for a large combined sewer overflow rehabilitation project. Deflections and piezometric heads were measured during the excavation and the pile driving processes and provided an opportunity to study the behavior of Rhode Island silts during excavation and pile driving.

In this study, a finite element analysis is performed to simulate the observed in-situ behavior. A commercial finite element program called PLAXIS Version 7 is used. The objective is to optimize the soil properties in the finite element model in such a

way that the simulated deflections during the excavation and pile driving process are similar to the measured deflections in the field.

In the first part of the study a literature review is presented. It involves case studies that have used finite element analyses to simulate the behavior of soils due to excavations. Publications that present some basic ideas and problems of finite element simulations for such case studies are also presented.

The second part of this study describes the construction activities and in-situ conditions at the gate and screening structure to be studied. The observed measurements are also presented.

Finally, the finite element analyses and the results are shown in the third part of this study. Analyses are divided into a simulation of the excavation and a pile driving-simulation. The reason is to highlight the differences but also the similarities between those two construction processes. A discussion of the results and conclusions drawn in this study are presented at the end of the thesis.

2. REVIEW OF PREVIOUS WORK

2.1 INTRODUCTION

In the literature, there are numerous studies that deal with finite element simulations of excavation support projects. This chapter presents summaries of selected studies and published papers relevant to this research.

An overview of some methods and problems of finite element analyses is presented in section 2.2. Examples of real-time in-situ monitoring and analyses that can be run to improve or verify finite element simulations will also be described. Section 2.3 describes selected studies that have used finite element simulations to model the behavior of excavation systems.

The following papers and studies summarized herein are listed in Table 2-1:

Table 2-1: Studies summarized in this thesis

Reference	Content of the Paper/Study
Brown and Booker (1985)	Discription of an FE analysis method
Finno et al. (1991)	Parametric studies of FE analyses
Finno et al. (2007)	Effects of geometry on FE analyses
Finno and Hashash (2009)	In-Situ monitorng and back-analysis methods
Whittle et al. (1993)	FE analysis of excavation in Boston, USA
Zornberg et al. (1998)	FE analysis of excavation in Sao Paulo, Brazil
Langousis (2007)	FE analysis of excavation in Seatle, USA
Hsiung (2009)	FE analysis of excavation in Kaohsiung City, Taiwan

The case studies presented in the following were chosen because of the different constitutive soil models presented, FE-codes or programs used, and relevance to the present study.

2.2 MODELING SUPPORT OF EXCAVATION SYSTEMS

2.2.1 FINITE ELEMENT ANALYSIS OF EXCAVATION BY BROWN AND BOOKER (1985)

This paper presented the results of a finite element analysis for an excavation simulation. A method was described that could provide good results when using a linear elastic constitutive model, and additionally for non-linear soil behavior as well as multi-stage excavations.

In general the following steps can describe the simulation of an excavation:

- The objective of the simulation presented was to remove the shaded portion A of Figure 2-1(a).
- Portion A was replaced by tractions (τ_i) as shown in Figure 2-1(b).
- If the tractions are removed because of simulating the excavation process, the behavior of portion B will change. This could be simulated by applying equal and opposite tractions as presented in Figure 2-1(c).
- This process could be repeated for more excavation stages.

Figure 2-1: Simulation of Excavation (Brown and Booker, 1985)

The authors noted that there were various methods to compute the described simulation process. One example was the approach of Mana (1976) in which the excavation boundary forces were determined by:

$$f = \sum_{m=1}^M \int_v B^T \sigma dV \quad (2.1)$$

where M was the number of elements, B was the displacement strain matrix, σ was the stress vector and f was the vector of nodal forces. This method presumed the direct determination of tractions and nodal forces from known values of stresses.

The authors of this paper proposed an approach in which a virtual work methodology was used. In fact, the nodal forces could be found by adequate numerical integration of stresses, body forces and external tractions throughout the soil mass. It was also assumed that total equilibrium was maintained at each stage of excavation. An advantage of this approach is the capability of producing no additional errors while simulating multi-stage excavations. Throughout different tests and simulations it could be shown that this method was also capable of simulating non-linear materials.

2.2.2 ANALYSIS OF BRACED EXCAVATIONS WITH COUPLED FINITE ELEMENT FORMULATION BY FINNO ET AL. (1991)

Finno et al. presented parametric studies of braced excavation behavior from finite element analyses. Parameters that were investigated included soil models, boundary conditions and details of the construction process. For the purpose of validation, the studies in this paper referred to a case study in Chicago (called HDR-4). The aim was to appraise the effects of commonly-made assumptions of the mentioned parameters on the computed deformation behavior of braced excavations.

For the studies a special finite element code was used (Harahap, 1990) that could compute plane strain and axisymmetric stress conditions and could perform drained, undrained loading as well as consolidation and fluid flow analyses. The soil

was simulated by eight-node biquadratic isoparametric elements with pore pressure degrees of freedom in the corner node (for consolidation effects) and without pore pressure degrees of freedom (for cohesionless soil or fully drained analyses). Sheet piles were simulated by beam elements and struts were simulated by bar elements. Additionally the interface between sheet pile and soil was coded by six-noded, quadratic isoparametric slip elements.

Parametric studies were performed with two different methods of loading. In the first method detailed in-situ inclinometer measurements of sheet pile displacements were applied as boundary displacements in the computations. It resulted in a certain soil response on the active side on the wall. It was assumed that uncertainties with the simulated excavation process were minimized by this displacement-controlled analysis. Nevertheless the results only provided a basis of comparison of computed response since only the active side of the wall was simulated.

The second method was the more common stress-controlled loading, which included simulations of excavation of pilot trenches, sheet-pile installation and alternating steps of excavation and strut installation. Comparisons between computed responses and measured responses were made during sheet-pile installation, the deepest excavation stage and after installation and preloading each strut level (4 levels in total).

Three different constitutive soil models were compared: a modified isotropic plasticity (i.e. modified cam clay) model (MCC) (by Roscoe et al., 1968), an anisotropic bounding surface model (ABSM) (by Banerjee et al., 1985) and an isotropic bounding surface model (IBSM) (by Harahap, 1990). All models were based

on effective stress response and were coupled with fluid flow equations to simulate pore pressure changes. Inputted soil parameters were determined from laboratory tests and from the literature. Table 2-2 summarizes the studies performed.

Table 2-2: Summary of parametric studies (Finno et al., 1991)

CASE NO.	LOADING TYPE	SOIL MODEL	BOUNDARY CONDITIONS	SHEET-PILE INSTALLATION	CONSTRUCTION SEQUENCE
1	BD	ABSM	West sheet pile	Included	Observed
2		IBSM		Included	Observed
3		MCC		Included	Observed
4		ABSM		Ignored	Observed (1)
5	SC	ABSM	Full Mesh	Included	Observed
6		IBSM	Full Mesh	Included	Observed
7		MCC	Full Mesh	Included	Observed
8		ABSM	Full Mesh	Ignored	Observed (1)
9		ABSM	⊗ Symmetric Mesh	Included	Observed
10		ABSM	⊗ Symmetric Mesh	Ignored	Observed (1)
11		ABSM	Full Mesh	Included	No Overexcavation

Key: BD = Boundary displacements applied at sheet-pile
SC = Stress controlled cycles of excavation and bracing
ABSM = Anisotropic bounding surface model
IBSM = Isotropic bounding surface model
MCC = Modified Cam Clay model
(1) = Observed after start of excavation

The results of the displacement-controlled simulation showed that there is little effect of either the sheet-pile installation or the soil model on the actual results (changes in stress). The authors concluded that the models only responded to active loadings, that is why their responses (stress path plots) looked quite similar.

More important were the results of the stress-controlled simulations since here an accurate simulation of sheet-pile deflection was attempted. There were many differences investigated, including the soil models, the finite element procedure, sheet-pile installation effects and construction sequences. Because of that, the authors tried to investigate the effects of the relevant parameters in more detail.

It was found that both of the isotropic soil models underpredicted the observed sheet-pile displacements compared with the ABSM model. The different passive responses of the soil were mainly caused by significantly different computed

responses. The isotropic MCC model behavior was essentially elastic for the entire loading process, which resulted in very stiff responses and small sheet-pile displacements. In contrast, irreversible deformations were allowed for the ABSM model that resulted in smallest passive resistance and stiffness and finally caused bigger deformations.

It was concluded that for isotropic models the deformations were greatly affected by the elastic constants (smaller values of shear or bulk moduli lead to higher deformation) used in the analysis. Also, these kinds of models lead to more accurate results if passive loadings dominate the behavior. Nevertheless, finding the right parameters required considerable judgment to obtain reliable predictions of deformation.

The second point of interest was the effect of sheet-pile installation. Different stress conditions at the excavation wall induced by the installation caused different deformations. It was observed that not including the sheet-pile effect in computations lead to 35% less deformations compared with computations that included the installation of sheet pile. This difference was mainly caused by a reduction of available passive resistance because of changing the pore water pressures in the soil throughout pile installation.

The third aspect that was studied was the mesh effect. Three different analyses were run incorporating the ABSM soil model. This includes a full mesh model (both excavation walls were modeled) and a centerline symmetric mesh model (one half of the excavation was modeled). The full mesh analysis includes the sheet-pile effect whereas the centerline symmetric mesh was analyzed with and without sheet-pile

effect, respectively. It was shown that the displacements in the half mesh analysis were overestimated, especially when the sheet-pile effect was incorporated. The other two analyses showed better, but still different results, which highlight the importance of a proper accounting of sheet-pile effects.

Furthermore, the effect of construction procedures was evaluated. Therefore a complete simulation of construction was computed including the activation of modeled strut levels as the excavation reached the proper elevation. Overexcavation was minimized. Also the real construction order and time was maintained to demonstrate the influence of construction sequence. Similar trends compared with former analyses were evaluated, but the final excavation stage predicted a 2.3 times smaller deflection of the piles than observed. The authors emphasized the importance of correctly simulating the real construction sequence and limiting the overexcavation.

Finally the authors concluded that incorporating all facts mentioned above in finite element analyses would significantly increase the quality of the results.

2.2.3 THREE-DIMENSIONAL EFFECTS FOR SUPPORTED EXCAVATIONS IN CLAY BY FINNO ET AL. (2007)

This paper presented results of finite-element simulations to define the effects of excavation geometry. Factors investigated included length, width, and depth of excavation, wall system stiffness and factor of safety against basal heave. All these factors lead to development of a plane strain ratio (PSR), defined as the maximum movement in the center of an excavation wall computed by three-dimensional analyses and normalized by plane strain analyses.

$$PSR = \frac{\sigma_{3D}}{\sigma_{2D}} \quad (2.2)$$

The authors first summarized observations that were made in different finite element studies:

- Smaller movements developed at the corners, compared with the center of the excavation wall.
- 2D calculations mostly overpredicted the movements near the center of the excavation wall for large distances between rigid stratum and excavation bottom and also for smaller ratios of length to height of the wall.
- In contrast, 3D simulations matched more closely with field responses.

Since no systematic evaluations of excavation geometry had been made previously, this paper addresses these influences in a parametric study.

PLAXIS 3D Foundation and PLAXIS 2D version 8.0 were used as the three-dimensional and plane strain geotechnical finite element software, respectively. A hardening soil model (non-linear elastic) was used to describe the soil behavior. Modeled excavations varied from 20 m by 20 m (L x B) to 160 m by 80 m. Excavation depths H_e ranged from 9.8 to 16.3 m. Consequently the ratio L/H_e varied from 0.5 to 12. Also, mesh boundaries were located more than 120 m from the excavation wall, that was more than the 5 times H_e recommended by Roboski (2004). Values of wall system stiffness S from 32 (flexible wall) to 3200 (stiff wall) were used to investigate stiffness effects.

The simulations showed that the computed movements of 3D analysis were less than those computed by plane strain simulations especially for smaller excavations. For larger excavations the observed movements were almost the same. To evaluate the effects of excavation size and depth, PSR values of normalized geometric parameters were compared. These were the ratio of primary wall length to elevation depth L/H_e and the ratio of primary wall length to secondary wall length L/B . It could be shown that a L/H_e ratio greater than 6 resulted in a PSR value of approximately 1. That means 3D and 2D analyses lead to similar wall movements. In contrast, large differences between 3D and 2D simulations were observed for L/H_e ratios smaller than 2 (see Figure 2-2, top). Furthermore a L/B ratio of 2 indicated no significant difference between 3D and 2D computed movements (=PSR of 1). A smaller ratio than 1 implied a movement of the shorter side of the excavation by what the L/H_e ratio became more important to determine the PSR (see Figure 2-2, bottom).

An investigation of the wall stiffness effect lead to the result that L/H ratios less than 2 caused lower PSR values, whereas the PSR of the flexible and medium walls increase faster for higher excavation depths than the stiff wall. This indicated a higher corner restraint for stiff wall systems.

Figure 2-2: Effect of plan dimensions and excavation depth on PSR (Finno et al., 2007)

Moreover it was observed that the factor of safety FS_{BH} against basal heave could have an influence on the PSR. In particular the PSR of stiff walls decreased for smaller FS, while flexible walls were not much affected by the FS. Nevertheless for L/H_e ratios of 6 or higher the PSR remained 1 regardless of stratigraphy and stability.

Finally an empirical equation was developed from the finite-element parametric study data:

$$PSR = (1 - e^{-kC(\frac{L}{H_e})} + 0.05(\frac{L}{B} - 1)) \quad (2.3)$$

where the factor C depended on the factor of safety against basal heave and could be determined by:

$$C = 1 - \{0.5(1.8 - FS_{BH})\} \quad (2.4)$$

Furthermore the value of k depended on the support system stiffness and could be taken as:

$$k = 1 - 0.0001(S) \quad (2.5)$$

where S could be determined by:

$$S = \frac{EI}{\gamma_{water} h^4} \quad (2.6)$$

Figure 2-3 presents a plot of Equation 2.3 compared with published data and data from this study. The solid line represented a base case (flexible support system and a FS_{BH} of 1.8 or higher), whereas the dashed lines represented upper (approximates plain strain conditions) and lower (extreme conditions like very stiff wall and high FS_{BH} or flexible wall and low FS_{BH}) bounds of Equation 3.

Figure 2-3: Comparison between published data and results of parametric study (Finno et al., 2007)

The authors concluded that despite of different soil models and assumptions used in this study, the trends in the finite-element results could be reasonably represented by computed limits of Equation 2.3 (see Figure 2-3). Also the proposed

equation included all observed effects that could influence the PSR like geometry of excavation, the stiffness of lateral support and the factor of safety against basal heave and could consequently be used to analyze the differences between plain strain analyses and 3D analyses.

2.2.4 INTEGRATED TOOLS FOR PREDICTING, MONITORING AND CONTROLLING GROUND MOVEMENTS DUE TO EXCAVATIONS BY FINNO AND HASHASH (2009)

Finno and Hashash described in their paper the collecting and use of monitoring data to update performance predictions of supported excavations. Also, inverse analysis methods used to improve computed ground movements were presented. The aim was to introduce a monitoring program during construction to record ground movement and use this data to control the process of construction and constantly update predictions of excavation movements.

According to the authors it was necessary to simulate all aspects of the construction process, like installation of supporting walls, hydrodynamic effects or material responses, which could affect the stresses around the excavation. This could increase the accuracy of predicted behavior. Moreover the plane strain ratio, PSR, described in section 2.2.3 had to be taken into consideration if the L/H_e ratio was less than 6.

Inclinometers, laser scanning systems, webcams, automated total surveying station and remote access tiltmeters, produce real time data. For typical elasto-plastic constitutive models inclinometer data based on measurements close to a support wall was the most useful. Horizontal displacements, settlements and pore water pressures

were also recorded. The construction process could be tracked by a three-dimensional laser scanning method called LIDAR (Light Detection and Ranging) and internet accessible webcams. Furthermore optical surveying stations were installed to monitor the displacement of optical prisms placed at different locations around an excavation site. Additional tiltmeters were attached to adjacent buildings to compute angular distortions (related to settlements). All monitoring stations included data transmission communication system, like RS232 serial interfaces or radio transceivers that allows for collecting collecting of data on a remote host computer.

This data was then used in an inverse analysis. In this analysis parameter values and other aspects of the model were adjusted until the computed results of the model matched the observed results of the system. The advantage was the ability to calculate parameter values automatically. In contrast the disadvantages were complexity, non-uniqueness of the solution and numerical instability. Two different types of inverse analysis had to be distinguished; gradient methods and artificial intelligence methods like artificial neural networks or generic algorithms.

Figure 2-4 shows a flow chart of the parameter optimization for a gradient method. The computed results (of a “first” finite-element analysis) were compared with field results by means of a weight least-squared objective function. This function provided a quantitative measure of accuracy of the predictions. By the weight function the parameters used for further analysis was chosen by its reliability (e.g. errors associated to measurements were minimized). Furthermore a sensitivity matrix was produced by a forward difference approximation. As a result optimized parameters like soil properties were obtained that were used in a final finite-element computation.

It was now possible to get good agreement between computed and measured excavation behavior and to update predictions of movements.

Figure 2-4: Flow chart for inverse analysis (Finno and Hashash, 2009)

Another inverse analysis method presented in this paper was the SelfSim self-learning engineering simulation. This analysis extracted relevant constitutive soil information directly from field measurements like deformations or settlements. After a certain learning process described below the resulting soil model, used in the final finite-element analysis, provided deformation results that were consistent with observed field behavior. This updated model could then also be used for predictions of similar excavations (similar soil layers, construction method, supporting structure, etc.).

Figure 2-5 describes the steps of a SelfSim analysis. At first, wall deformations and surface settlements were measured in the field (step 1). The measured deformations and the known excavation stages were then traded as complementary sets, which had to be computed in a numerical model. The key of this method is a neural network based model that had to simulate the soil response. This model was

used to compute the soil response using stress-strain data (initially from laboratory tests or stress history of the soil). In a second step a finite-element analysis using the initial neural network soil model was performed with a numerical model that represented the investigated construction sequence (step 2a).

Figure 2-5: SelfSim learning Training (Finno and Hashash, 2009)

Another finite-element analysis was done with a second numerical model where measured wall deflections and settlements were imposed as additional

displacements boundary conditions (step 2b). The neural network model was also used for this simulation. Subsequently the stress field of step 2a and the strain field of step 2b were extracted to build stress-strain pairs, which were used to train the neural network soil model. It was assumed that these stress-strain pairs represented an approximated constitutive soil response. The analyses of step 2 were repeated until analyses of steps 2a and 2b provided similar results. Finally the resulting and “trained” constitutive soil model could be used for predictions of later construction steps or even for other excavations with similar ground conditions (step 3).

The authors concluded that their integrated tool to predict, monitor and control ground movements provided good results between predictions and observed performance of excavations. In particular, the calibration of numerical models (by means of parameter optimization) throughout inverse analysis could minimize the errors between measured and predicted results.

2.3 CASE STUDIES WITH FINITE ELEMENT SIMULATION

2.3.1 ANALYSIS OF DEEP EXCAVATION IN BOSTON BY WHITTLE ET AL. (1993)

The author described the application of a finite element analysis to model an excavation of a seven story underground parking garage in Boston, Massachusetts. The aim of this analysis was to predict soil deformations and ground water flow that occurred during the excavation. For this analysis coupled flow and deformation analyses of the soil at different construction sequences were incorporated. Furthermore a numerical algorithm for modeling a nonlinear soil and an advanced constitutive model of clay behavior was used.

The construction site occupied an area of 6880 m² and was surrounded by up to 40 stories tall buildings. A cast-in-place, reinforced concrete diaphragm wall with a thickness of 0.9 m and an elevation depth of -21 m was build to resist the lateral loads. The roof and the floor levels, which are used as supporting structure, had been cast in a top-down excavation sequence (i.e. the soil beneath the recently constructed slab was excavated). Additionally sump pumps and deep wells, located inside the excavation were used to dewater the area. Figure 2-6 shows the garage structure, soil types, measured pore pressures and stress history.

Figure 2-6: Garage Structure (Whittle et al., 1993)

A base case analysis was performed first. The purpose of this was to show the capabilities of the finite-element model to evaluate geotechnical variables like magnitudes of wall movements and soil deformations that occurred during the excavation process, settlements due to pore pressure changes in the clay layer and the quantity of penetrated water into the excavation. For this purpose a modified ABAQUS finite-element code was used to run the analysis. Based on the structure presented in Figure 2-6, a two-dimensional, plain-strain geometry model was created. This was done since the computational effort in performing a three-dimensional analysis would have been very large and uncertainties in site conditions and soil properties would have a larger influence on the results.

Two different soil models were used to model the soil behavior. The stress-strain-strength properties of fill, sand, till and argillite were described by an elastoplastic model using a Drucker-Prager failure criterion with a nonassociated flow rule

(EP-DP). Initial parameters were obtained from laboratory data or from the literature (Table 2-3). The MIT-E3 effective stress soil model by Whittle (1990) was used to describe the behavior of the clay layer. The parameters used in this model were validated by laboratory data. Table 2-3 summarizes the input properties used for the finite element analysis.

Table 2-3: Input Properties used in the Finite Element Analysis (Whittle et al., 1993)

Stratum (1)	Soil model (2)	Total unit weight, γ , (kN/m ³) (3)	Initial Conditions		Deformation Properties		Shear Strength ^a		Permeabil- ity k (cm/s) (10)	References (11)
			K_0 (4)	Over-con- solidation ratio (5)	G/σ'^b (at $\gamma = 0.1\%$) (6)	ν' (7)	c' (kPa) (s_u/σ'_{v0}) (8)	ϕ'_{psc} (degree) (9)		
Dry, fill, wet	ED-DP	19.6, 22.0	0.5	—	35	0.3	0	30	1×10^{-4}	—
Boston Blue Clay	MIT-E3 ^c	19.6	1.22, 0.6	6.0, 2.0	1,100 ^d (150)	0.28	(1.6) ^e , (0.62) ^e	42 ^e	5×10^{-8}	Whittle (1990)
Sand	ED-DP	19.6	0.4	—	170	0.3	0	37	1×10^{-5}	—
Till	ED-DP	20.4	1.0	—	110	0.3	0	43	1×10^{-5}	Lutenegger et al. (1983)
Weathered argillite	ED-DP	22.0	1.0	—	1,160	0.3	1,720	32	1×10^{-5}	Einstein et al. (1983) and Young (1990)
Intact argillite	EP-DP	23.6	1.0	—	4,900	0.3	1,720	32	1×10^{-5}	Einstein et al. (1983)

^aAll strength properties refer to plane strain compression conditions.
^b $\sigma' = 1/3(\sigma'_{xx} + \sigma'_{yy} + \sigma'_{zz})$.
^cComplete input properties for MIT-E3 are shown in Table 3.
^dSmall strain stiffness, G_{max}/σ' .
^eUndrained shear strength ratio and large strain (critical state) friction angles are predicted by model.
^fElastic properties of concrete; $E = 2.3 \times 10^4$ MPa; $\nu = 0.15$.

Successive “stages” in the analysis were used to simulate different construction sequences at the site. The repetitive sequence of excavating and building each floor was simulated by three stages in the analysis. First an undrained excavation to the associated elevation (beneath each built floor) was computed. Then a time delay was incorporated to simulate curing of the concrete and partial drainage. Finally a structural prop that corresponded to the installed floor slab was simulated. Any computed deformations were then relative to an initial equilibrium state (stage 5 of Figure 2-7). Additionally, allowing pore pressures to partially dissipate at each stage simulated the dewatering.

Figure 2-7: Construction Sequences of the Finite Element Model (Whittle et al., 1993)

The finite element mesh itself consisted of 611 isoparametric elements with 4,410 nodal degrees of freedom. Each element included eight-displacement nodes and four-corner pore pressure nodes. Solid eight-node elements displayed the diaphragm wall and the fill whereas the floor slabs were modeled by one-dimensional springs.

As shown in Figure 2-8 the boundary conditions along the bottom and the side of the excavation were different. For the base case analysis the lower boundary was assumed to be non-flow boundary, whereas the side was an open boundary with maintained initial pore pressure.

Figure 2-8: Finite Element Mesh (Whittle et al., 1993)

The predictions of the base case analysis were compared with measured field data. This data was constantly recorded by inclinometers (lateral movements of the diaphragm wall and lateral soil displacement), optical surveys (surface settlement and movement of the surrounding buildings), extensometers (relative, vertical displacement of the clay, till and rock) and piezometers (ground-water and piezometric levels). All these measured variables were also computed by the finite-element analysis.

The comparison between prediction and field data showed that there was a significant difference in computed and real behavior. In particular, the predicted settlements caused by piezometric elevations were overestimated. Also, the lateral wall deflections were not accurate because of shrinkage and expansion effects of the floors that were not incorporated in the analysis before.

The first analysis was then modified significantly. Floor slab shrinkage was incorporated and the lower boundary was changed into a constant pore pressure boundary condition. After these changes, the computed deflections matched much

better with the field data. Figure 2-9 presents the base case analysis, the modified analysis and measured data:

Figure 2-9: Comparison between Predicted and Measured Wall Deflections (left) and Piezometric Elevations (right) (Whittle et al., 1993)

This case study showed that it is possible to make a reasonable prediction of deformations due to a top-down construction project in soft clay. Nevertheless there are some remarks of the author that described some solutions for occurred difficulties:

1. Not only lateral wall deflections should be recorded and compared with predicted values. Additional information provided by measurements of soil deformation show the effects of excavation procedures much better and are essential to validate the model predictions.
2. Uncertainties of soil properties need to be minimized as model complexity increases.
3. Concrete shrinkage, important for cast-in-place floor systems, should be taken into consideration to compute more trustable wall deflections.
4. Defined boundary conditions can affect significantly the change in piezometric elevations.
5. An improved characterization of small strain nonlinear behavior of soils can improve the predictive capabilities of a finite element model.

2.3.2 NUMERICAL PREDICTION OF THE BEHAVIOR OF AN EXCAVATION IN RESIDUAL SPOILS BY ZORNBERG ET AL. (1998)

Zornberg et al. (1998) performed a finite element analysis to evaluate the behavior of a deep, braced excavation in residual soil. The excavation is part of a tunnel-access of a subway in São Paulo, Brazil. For this analysis a nonassociated elasto-plastic model by Lade (1977, 1979) was calibrated from results of laboratory tests. The aim was to predict the displacement of the excavation, including the stress

fields induced in the residual soil mass during different excavation stages. This model would then be used to predict the performance of adjacent structures.

The analyzed excavation was 31 m deep and was located next to an existing 17-story building with two underground levels. A soldier pile and lagging system, supported by three strut levels, were used to resist lateral forces. 18 m of “Residual Red Clay” underlain by a “Residual Variegated Soil” layer was found on the construction site. Furthermore the water level that was initially located at the base of the red clay layer was lowered to 35 m before the excavation process. Figure 2-10 shows a cross-section of the excavation:

Figure 2-10: Cross-section of the excavation (Zornberg et al., 1998)

The author mentioned that there is little experience about the application of an elasto-plastic model to represent the behavior of “undisturbed samples of unsaturated soils.” Therefore an extensive laboratory-testing program was performed, which

included tests at different shear stress paths representative of excavation. A summary of the elasto-plastic parameters used for the residual soil are shown in Table 2-4).

Table 2-4: Summary of elasto-plastic parameters for the residual soils (Zornberg et al., 1998)

			Red Clay	Variegated Soil	
Elastic Parameters			K_{ur}	153.97	1051.46
			n	0.56	0.11
			v	0.25	0.29
Plastic Parameters	Collapsible		p	0.019	—
	Expansive	Failure	η	133.63	561.12
			m	0.88	1.20
			S	0.40	—
		Plastic Potential	t	3.67	—
			R	-17.46	—
			P	0.41	—
		Work Hardening	l	0.44	—
			α	1.65	—
			β	0.00	—

The results of the laboratory tests were compared with the predictions of the nonassociated elasto-plastic model at the element scale. A good agreement between measured and predicted behavior was achieved that supported the applicability of the model to the overall analysis.

The computer code ANLOG (Zornberg and Azevedo, 1990) was used to perform the finite element simulation of the excavation. This code also incorporates Lade's elasto-plastic model, which includes two yield surfaces; a conical shaped plastic expansive surface (characterized by a nonassociated flow rule) and a cap-type plastic collapsive surface (governed by a associated flow rule). Consequently, elastic, plastic expansive and plastic collapsive components were used to describe the total strain increments. Eight-node isoparametric elements were used to model the soil and three-node elements were used to describe struts and anchors. The finite element mesh finally consisted of 481 nodal points and 147 elements.

Two sets of analyses were run to simulate the excavation process. First, the state stresses in the soil prior to the excavation were analyzed. This involved four steps as shown in Figure 2-11 and summarized as.

- Step 1: Characterization of initial geostatic state, defined by the soil unit weight and the earth pressure coefficient K_0 .
- Step 2: Simulation of two underground level excavation of the adjacent building.
- Step3: Application of a distributed loading to simulate the effect of the building foundations.
- Step 4: Lowering of the water table.

The final state of stress level describes the initial stress level for the second set of analyses.

Figure 2-11: Analyses performed to define the stress state before the excavation (Zornberg et al., 1998)

Four construction phases were simulated for the excavation as shown in Figure 2-12. The first excavation step did not include placement of struts whereas the following phases incorporated struts that were simulated by activating the corresponding bar elements.

Figure 2-12: Four simulated excavation phases (Zornberg et al., 1998)

The aim of the analysis was to obtain stress and displacement fields in the soil elements for each stage. Additionally the loads in the structural elements were estimated.

One result showed that the settlements of the adjacent building were negligible. Moreover, a maximal lateral displacement of 37 mm was predicted at the bottom strut level (Figure 2-13), 4th stage. This is consistent with a comparatively high force in this level. Finally, the analysis predicted no development of a failure mechanism in the surrounding soil, although there was some fully mobilization of shear strength detected at the bottom of the excavation.

Figure 2-13: Predicted lateral wall displacement during the four construction stages (Zornberg et al., 1998)

Unfortunately, the author did not address the issue of the reliability of the results. Since the main reason for the simulation was to predict the performance of adjacent building structures, no extensive in-situ measurements were incorporated. Only the predicted settlements of the 17-story building were confirmed by several measurements.

2.3.3 AUTOMATED MONITORING AND INVERSE ANALYSIS OF A DEEP EXCAVATION IN SEATTLE BY LANGOUSIS (2007)

The aim of this case study was to test the performance of an automated survey system that was invented to monitor the behavior of a deep excavation. Finite element analyses were performed to predict the wall movements and to compare it with results gained by monitoring data. Several sets of finite element analyses were performed to evaluate the effects of soil stresses, tieback placements and 3-D corner restraint on the predicted displacements.

The construction site was located in Seattle, WA and consisted of a 21.6 m deep excavation for 5 parking levels. Soldier pile walls with wooden lagging and 4 to 5

rows of tiebacks were installed on the North, South and East sides of the excavation. For the West shoring wall a special design by GeoEngineers, consisting of a soldier wall with 9 to 10 rows of soil nails and 4 to 5 rows of tieback, was used to keep the deformation smaller than 1 inch. These West wall piles were part of further investigations (Figure 2-14). The soil consisted of fill, silty sand, clayey silt and very dense sand.

Figure 2-14: Design sections for a case study of an excavation support system in Seattle, WA (Langousis, 2007)

Different systems were used to measure lateral movements, especially along the west wall of the excavation. This included three inclinometers (situated at the back of pile 6, 13 and 18), optical surveying by the help of a “Leica TPS 1101 Total Surveying Station”, strain gages on the soil nails and load cells on the tiebacks (to monitor the load transfer from the ground to the structural components of the wall). The data obtained from these measuring systems was used to provide an early detection of deflections that could potentially damage the nearby structures and to validate a numerical model.

The design company GeoEngineers first numerically simulated the excavation with the finite element program PLAXIS V8 (2002) in 2-D. A hardening-soil model was used, which includes soil dilatancy and a volumetric yield surface that isotropically expands due to plastic straining. It also incorporates irreversible plastic strain due to primary deviatoric loading. The finite element mesh (15-node triangular elements) used by GeoEngineers simulated the excavation (East boundary is not symmetry axis) and adjacent buildings (Qwest Building) (Figure 2-15). The mesh was further refined near the wall, the soil nails and the tiebacks.

Figure 2-15: Finite element mesh designed to represent the Olive 8 excavation (Langousis, 2007)

The recorded deflection from the inclinometers was compared with the deflections predicted by GeoEngineers for the three design sections (Figure 2-16). It could be shown that the deflections were over predicted, especially for the Upper Sand and Clay layers. However, it was concluded that the shoring wall and the soil

responded more stiffly than expected since the patterns of observed and predicted deflection profiles are quite similar.

Figure 2-16: Observed and predicted deflections (Langousis, 2007)

Since there was a large difference between predicted and observed movement a new numerical model was developed after the excavation was completed.

The new model includes a staged excavation for the Qwest building to simulate the stress history induced by the construction of this building. Therefore, the excavation was computed and the soldier wall was replaced with a rigid wall. The struts were replaced by floor slabs. These changes in the numerical model lead to different K_0 values compared with the design parameters of GeoEngineers. Mainly affected were the upper and the lower sand (K_0 values for the upper sand went from 0.38 to 0.66; lower sand values went from 0.36 to 0.6). Revised soil parameters can be

seen in Table 2-5. The constitutive soil models used in the analyses were either the Hardening–Soil (S/H), or the Mohr-Coulomb M/C) models. All the structural elements and the concrete were assumed to behave as elastic materials. Moreover, drained and undrained analyses were run to compute an upper and lower limit of response in the clay layer.

Table 2-5: Soil parameter used for the design in the revised simulations (Langousis, 2007)

Soil Parameters					
Parameter	Fill	Silty Sand	Clayey Silt	Dense Sand	Jet-Grout
Constitutive Law	S/H	S/H	S/H	S/H	M/C
Soil Unit Weight, γ (pcf)	125	130	125	130	145
Friction Angle ϕ ($^{\circ}$)	32	38	34	40	0
Cohesion, c (pcf)	100	200	200	0	7000
Lateral Stress Coefficient, K_0	0.47	0.6	0.7	0.6	-
Poisson's Ratio, ν	0.3	0.3	0.2	0.3	-
Dilation Angle, ψ ($^{\circ}$)	0	0	0	0	-
Soil Stiffness E_{50}^{ref} (ksf)	600	1000	500	1500	-
Unload/Reload Stiffness E_{ur}^{ref} (ksf)	2600	3000	1600	4500	-
Oedometer Stiffness, E_{oed}^{ref} (ksf)	500	1000	700	1500	-
Interface Reduction Factor, R_{inter}	0.67	0.67	0.67	1	-
Reduced Interface Factor, R_{inter}	-	0.2	0.2	-	-
Reference Pressure p_{ref} (atm)	1	1	1	1	1

The finite element mesh (Figure 2-17) consisted of 15-node triangular elements. Both excavations (Qwest and Olive 8) were simulated in their full dimension. Also, frictional interface elements were placed between wall and soil as well as between anchors/nails and soil. The left and right boundaries were designed 5 times the excavation depth away from the excavation to ensure no influence of these boundaries on the simulation.

Figure 2-17: Finite element mesh for new numerical analysis (Langousis, 2007)

The construction process was simulated by successive design stages, which are: the sheet pile installation, followed by alternating stages of excavating and soil nail installation (between 9 to 10 rows of nails), and then alternating stages of excavation and tieback installation (between 4 to 5 rows of tieback). All these steps were simulated after the Qwest excavation was initially computed.

Figure 2-18: Comparison between drained/ undrained analyses, GeoEngineers analysis and observed deflections (Langousis, 2007)

Finally revised results showed better agreement with observed field data (Figure 2-18). Nevertheless, the computed deflections in the Silty Sand and the Clayey Silt layers were still larger than observed. Furthermore the undrained analyses reduced the deflection in the clay layer.

One reason for that difference could be 3-D effects in which stiffening effects of the corners of the excavation are not considered in plane strain analysis. It was investigated that especially the Clayey Silt layer 50 to 75 feet below the surface could not be simulated in plane-strain analyses in a sophisticated way (critical PSR value). The results obtained from two-dimensional analyses likely over-predicted the movements in this layer.

Another factor that was discussed in the case study was the numerical representation of the tiebacks in the finite element analysis. Tiebacks have a three-dimensional geometry but modeling them as two-dimensional elements includes significant approximations. The two-dimensional analysis might cause the stresses transmitted to the soil overlap, reducing the tieback load-bearing capacity and producing excessive displacements. To evaluate this influence tiebacks were modeled as equivalent struts (springs that transmit axial forces) with an equivalent strut length and stiffness. This study had shown that modeling the tiebacks as equivalent struts has little effect on the calculated deflections.

Moreover, two sets of inverse analysis (drained and undrained) were performed to find the soil parameters that provided the best fit to the observed lateral deflections. Parameters that were optimized are: reference value for primary deviator loading E_{50}^{ref} , the value for elastic unloading and reloading E_{ur}^{ref} and the odometer stiffness E_{oed}^{ref} since

these parameters had the most influence on the excavation behavior. Through the inverse analysis better agreement to the observed lateral deflection was achieved.

Table 2-6: Results of drained (left) and undrained inverse analysis (right) (Langousis, 2007)

Initial Stiffness Input Values versus the Values after the Three and Two Parameter Optimizations						Comparison of 2 and 3 Parameter Optimization for Undrained Clayey Silt Analysis				
		3-par Optimization		2-par Optimization					3-par Opt.	2-par Opt.
Layer	Parameter (kips/ft ²)	Initial Value	Final Value	Initial Value	Final Value	Layer	Parameter (kips/ft ²)	Initial Value	Final Value	Final Value
Silty Sand	E_{50}^{ref}	1000	1385	1385	1150	Silty Sand	E_{50}^{ref}	1000	601	1713
	E_{oed}^{ref}	700	970	970	805		E_{oed}^{ref}	700	421	1199
	E_{ur}^{ref}	3000	4156	4156	3450		E_{ur}^{ref}	3000	1803	5139
Clayey Silt	E_{50}^{ref}	500	1306	1306	12437	Clayey Silt	E_{50}^{ref}	500	277	6280
	E_{oed}^{ref}	350	914	914	8706		E_{oed}^{ref}	350	194	4396
	E_{ur}^{ref}	1600	28808	3919	37311		E_{ur}^{ref}	1600	5794	18840

Finally, the author drew the following conclusions:

1. Including the excavation history of the Qwest building in the finite element analysis lead to a displacement profile that was closer to the movements observed in the field.
2. Undrained instead of drained analysis for the Clayey Silt layer resulted in a more accurate deflection profile.
3. The computed deflections in the Clayey Silt layer are higher than observed since the 3-D effect increased the stiffness at the corner of a deep excavation.
4. Since the inclinometers were attached at the soldier piles, they were influenced by the stiffness of the soldier pile. The inclinometer should be located behind the wall so that localized effects to not influence the results.
5. Computing the tiebacks as equivalent struts did not influence the displacement profile.

6. Inverse analyses of certain input parameters lead to more accurate results of lateral movements.

2.3.4 A CASE STUDY ON THE BEHAVIOR OF A DEEP EXCAVATION IN SAND BY HSIUNG (2009)

This case study presented an analysis of an excavation in sand combined with numerical analyses of soil elasticity, creep and soil-wall interface. Additional back-analyses were performed to determine soil parameters that are important to predict excavation movements in sand more accurately.

The excavation for a subway station was located in Kaohsiung City, Taiwan and is part of the orange line in the Kaoshiung rapid transportation system. The length of the excavation was 194 m and the width was 20.70 m. A 1 m thick and 36 m deep reinforced concrete diaphragm wall was used to retain a 19.60 m deep excavation. Additional supporting structures were 5 levels of horizontal struts (w-shaped). The soil consisted of silty sand and the groundwater level was at 3.5 m below the surface. Figure 2-19 shows a cross section and layout of the excavation.

from the modeled excavation minimized the effects of themselves. It was also assumed that the installation of walls and struts had no effect on the surrounding soil. The elastic-plastic “Mohr-Coulomb” model was used and the soil parameters were estimated from laboratory tests, standard penetration tests (SPT) and measurements of shear wave velocity on site (Table 2-7).

Figure 2-20: Finite element mesh (Hsiung, 2009)

Table 2-7: Soil parameters (Hsiung, 2009)

Depth (m, below ground level)	SPT-N value	c' (kPa)	ϕ' ($^{\circ}$)	Undrained shear strength (S_u , kPa)
0.0–7.5	5–14	0	32	–
7.5–10.0	4	0	30	28
10.0–22.5	6–22	0	32	–
22.5–25.0	12–16	0	33	98
25.0–29.5	19–29	0	33	–
29.5–32.0	13–19	0	32	112
32.0–60.0	28–42	0	33	–

The analyses showed that the determination of the soil stiffness from shear wave velocity (denoted “WV”) compared to estimates from the standard penetration test (denoted “SPT” and “SPTR”) lead to different predictions of wall movement. Moreover settlements were underestimated and did not match with observations.

A different computer program (PLAXIS) was used for the purpose of comparison. This program incorporated an interface element between the wall and the

soil, which was not available in the FLAC analyses. Seepage analyses were also included in the revised analyses. The stiffness parameters followed the same assumptions like before (shear wave velocity: denoted “ E_s ” and SPT: denoted “ E ”).

Figure 2-21: Observed and predicted lateral wall movement in 3.4 m excavation depth (a) and final excavation depth (b): FLAC (left), PLAXIS (right) (Hsiung, 2009)

The PLAXIS simulation predicted smaller movements than those predicted using FLAC (Figure 2-21). Since there is no significant difference in the predicted vertical displacement for FLAC and PLAXIS the author concluded that seepage did not have a significant effect on the movements (PLAXIS allowed seepage, whereas FLAC did not). The influence of the interface element, however, was identified as the main reason for the different predictions of lateral wall movements.

Additional back-analyses were performed to investigate the influence of soil creep and soil-wall interface. During the construction process there was a late installation of a strut level. It was assumed that this could have an impact on lateral wall movements. Therefore a simple creep model (visco-elastic) was incorporated in FLAC simulations. Different values of dynamic viscosity “ D_v ” were used in further analyses, whereas other input values (like small strain parameters) were kept consistent. The back analyses showed that different dynamic viscosities lead to different lateral wall movements (Figure 2-22). Good agreements was achieved by D_v values of $1.5 \cdot 10^{15} - 2.0 \cdot 10^{15}$ Pa. Also, better results for the prediction of settlements were obtained.

Figure 2-22: Observed and predicted lateral movement (left) and surface settlement (right) on different D_v : (a) 3.4m of excavation depth, (b) final excavation depth (Hsiung, 2009)

Nevertheless, the predictions of surface settlement were under-estimated due to missing assumptions regarding the behavior of the soil-wall interface. Further analysis incorporated changing interface parameters (friction angle Φ_{sw} , normal and shear stiffness K_n and K_s). The final results showed that analyses not using a soil-wall interface under-estimate surface settlements.

Finally, the author made the following conclusions:

1. Predictions based on constant soil elasticity over-estimated the lateral wall movements below the excavation level. Furthermore, the predictions of surface settlement did not match at locations close or far from the excavation.
2. An elastic-perfect plastic model provides more consistent predictions for the use of small strain parameters.
3. The effect of an excavation-induced seepage had a limited effect on vertical displacement.
4. Creeping (time-dependent behavior of soils) caused by late installation of struts affected the vertical movement and could be addressed by using a special dynamic viscosity “ D_v ” parameter.
5. Limits of the applied constitutive model caused inconsistencies between predictions and observations in surface settlements.
6. The use of a soil-wall interface lead to more reliable predictions of surface settlement. A certain factor of normal and shear stiffness K_n and K_s was used to address this issue.

2.3.5 SUMMARY AND FINDINGS OF CASE STUDIES

It has been demonstrated that stress-controlled loading (simulating excavation and strut installation) leads to more reliable results than the displacement-controlled loading (in-situ measurements applied as boundary conditions) since the latter only simulates the active side of the wall (Finno et al., 1991). Also, different constitutive soil models have been used with varying degrees of success. Quite simple linear soil models (e.g. constant soil elasticity) mainly overestimate lateral movements (Finno et al., 1991; Hsiung, 2009) whereas more complex models like the Hardening-Soil model (Langousis, 2007) or an effective stress soil model (Whittle et al., 1993) lead to better results.

The influence of sheet pile wall installation was addressed although there seems to be no satisfactory method to incorporate this into the simulation (Finno et al., 1991). Therefore all finite element studies presented above assumed that the wall was “wished in place” and have no influence on the computed results. An interface element between soil and wall could provide something like a proper simulation (Hsiung, 2009) of pile driving.

An influencing fact was the boundary conditions. Simulations containing water flows (like settlement and dewatering problems) had to deal with this in particular. For example Whittle et al. (1993) changed the lower boundary conditions from non-flow to static water head boundary condition and the results changed favorably. All other studies created boundary conditions that are at least 5 times the excavation depth away from the excavation wall to minimize any influence.

Furthermore, simulation phases/ sequences similar to the real construction activities on site were used in all studies to compute different movements in different construction phases. This process was mostly used to simulate the change in stress during certain construction activities or even ground water lowering.

The mesh effect addressed by Finno et al. (1991) was not considered in every study (e.g. Whittle et al., 1993 and Hsiung, 2009) although the influence of opposite walls connected with struts or concrete slabs was recognized by some authors (e.g. Langousis, 2007).

Another issue addressed in some studies is the stiffness effect at the corners of an excavation. All studies presented above used two-dimensional simulations instead of three-dimensional simulations. For this reason it could be that the simulated deflections near the corner of an excavation were much higher than measured in the field. Finno et al. (2007) developed an equation that incorporates excavation dimensions and wall stiffness to calculate a PSR-ratio (defined as deflections calculated with three-dimensional analysis normalized by deflections calculated with two-dimensional analysis). It was found that an excavation-length/excavation depth ratio higher than six will probably lead to comparable results in 3-D and 2-D simulations. In contrast L/H_e values smaller than 2 will definitely lead to different results.

Back-analyses were used to improve the simulation results, and both artificial intelligence methods (Finno and Hashash, 2009) or inverse-analyses were used (Langousis, 2007).

3. CASE HISTORY - THE NARRAGANSETT BAY COMMISSION COMBINED SEWER OVERFLOW PROJECT (CSO)

3.1 BACKGROUND OF CSO PROJECT AND SITE DESCRIPTION

The City of Providence in Rhode Island has a combined sewer system that collects storm water runoff and wastewater at the same time. During large rainfalls the existing system, which is already operating at capacity due to an increasing population, is incapable of handling these large flows. For this reason the existing system had to be upgraded. The solution was to construct a 4.8 km long and 7.9 m inside diameter storage tunnel that could temporary handle sewer overflow. Also part of the project was a 35.7 m long pump station and a series of near surface divisions as well as conduits that connect the new tunnel to the existing Providence sewer system by a series of drop shafts. Preliminary study and design phases for the Narragansett Bay Commission (NBC) – CSO project were begun in the early to mid 1990's and the final design was completed by May 2001. Construction started in October 2001 and was completed in 2007.

The focus of this chapter is to provide information about a site containing a gate and screening structure for one of the vertical drop shafts. The structure is called C-8. This site is located in the southern end of Providence, where Route 1A (Allen's Ave) and Interstate Route I-95 run parallel to the site (see Figure 3-1).

Figure 3-1: Site location of gate and screening structure C-8 (Google Maps)

The gate and screening structure at site C-8 has a length of 10.4 m, a width of 5.4 m and a depth of about 7.6 m. It is constructed of concrete and is supported on deep foundations. For construction activities a temporary support of excavation (SOE) by means of CZ-128 sheet piles with three levels of internal bracing was installed. The bottom of the excavation was at elevation -5.1 m. Levels of bracing (wales and struts) was located at elevation 2.4 m, elevation -0.9 m and at elevation -3.0 m, respectively. The dimensions of the sheet pile are 45 m long, 9.75 m wide and a tip elevation of -

11.9 m. A plan view of the gate and screening structure and the sheet pile wall location can be seen in Figure 3-2.

The site is relatively flat (~elevation +4 m above sea level) close to the excavation and east of the excavation (towards Allens's Ave) compared to a slope of 2H:1V (up to elevation 11.6 m) on the west side of the excavation (towards I-95) (see cross section Figure 3-3). A Geotechnical Baseline Report (Haley and Aldrich, Jacobs Civil, 2002) and other regulations provided in the NBC contract documents provided the basis for the design of the excavation support and deep foundations.

Figure 3-2: Site plan showing extents of excavation. The sewer runs from the north into the gate and screening structure and then continues to the drop shaft. The extent of the excavation held by the sheetpile walls is shown as a dashed line.

Figure 3-3: Cross section A-A from Figure 3-2

3.2 SUBSURFACE CONDITIONS AND CONSTRUCTION ACTIVITIES

The subsurface conditions were determined by means of two geotechnical borings (Haley and Aldrich, Jacobs Civil, 2002) close to the excavation. Borehole BS98-7 was located near the north end of the excavation, and borehole BD98-12 was located near the south end of the excavation (see Figure 3-2). The borings indicated that the soil directly below the ground surface consists of approximately 2.1 m to 3.1 m of loose to dense fill. At the north end of the excavation the fill is underlain by 0.9 m alluvial deposits (brown fine sand, sandy silt with silt) followed by glaciolacustrine deposits (gray to brown coarse to fine sand with various amounts of gravel, silt or fine sandy silt). At the southern end of the excavation the fill is underlain by 4.9 m estuarine deposits (gray medium to fine sand with organic silts) followed by 0.6 m of glaciofluvial deposits (gray to brown coarse to fine sand with gravel or silt). Underneath the estuarine deposits are approximately 24.1 m of glaciolacustrine

deposits (brown to gray laminated to varved silt with varying amounts of clay or fine sand).

Figure 3-4: Cross section showing subsurface conditions

The ground water varies from elevation 1.8 m (before excavation) to 0.6 m (after excavation) due to dewatering inside the excavation.

On October 1, 2003, preparations for the construction at the gate and screening structure began. The sheet pile walls were installed first from 10/08/2003 to 10/29/2003. This was followed by alternating stages of excavation and installation of supporting struts (11/04/2003 to 01/08/2004). The excavation process was finished when the final grade was reached (01/02/2004 to 01/08/2004). Afterwards a concrete mud mat was placed (01/13/2004) at the bottom of the excavation, before the piles were driven (02/02/2004 to 02/18/2004) to depths ranging from 20 to 24 m below the bottom of the excavation. A vibratory hammer was used to install the piles along the perimeter of the structure. Finally, the capacity of the piles were checked and verified

using an impact hammer (02/24/2004 and 02/25/2004). A summary of the construction activities is shown in Figure 3-5. Figure 3-6 shows the supporting structure (sheet pile walls, struts and wales) and the dropshaft at the C-8 site.

Figure 3-5: Construction sequence for the C-8 site

Figure 3-6: Photo of the excavation and Support of Excavation (SOE) system taken at the southern end of the C-8 site

3.3 IN-SITU INSTRUMENTATION

Because of the unpredictability of Rhode Island Silts it was decided to install 3 inclinometers and 2 multi-level piezometers prior to the excavation to observe lateral movements and pore pressures, respectively. A monitoring well to detect changes in the water level was also installed.

The inclinometers were located along the west side of the excavation, in particular at the north (INC-4) and south side (INC-5) of the excavation as well as in midspan (INC-10). Piezometers were installed below the bottom of the excavation inside the screening structure (PZ-1) and outside the structure (PZ-2). The monitoring well (OW-4) was located at the east side of the excavation. Locations of the instrumentation are shown in Figure 3-7.

Figure 3-7: Locations of geotechnical instrumentation

Inclinometers measure lateral movement in two orthogonal directions, and these were measured perpendicular (A-axis) and parallel (B-axis) to the direction of

the excavation. In two of the inclinometers (INC-4 and INC-5), the magnitude of the movements parallel to the excavation were comparable (up to 12 cm) to the perpendicular movements. However, parallel movements were not simulated or studied as part of this thesis.

During construction perpendicular measurements in all inclinometers showed a bulging deflection with the highest movement at the bottom of the excavation. This behavior is characteristic of braced cuts where passive soil resistance decreases because of the removal soil inside the sheet pile wall (Ergun, 2008; Bradshaw et al., 2007). Movements inside the excavation occur before the struts can be installed. Different stages of excavation depth and strut installation caused different amounts of displacement. For example, excavating the first bracing level caused only small movements between 0.8 and 2.1 cm at the top of the sheet pile wall. In contrast, significant movements occurred as the excavation reached the glaciolacustrine deposits and the third level of bracing. Maximum deflections of 9.5 cm were measured at INC-4, 10.8 cm at INC-5 and 20.5 cm at INC-10 (see Figure 3-8 to Figure 3-10). The different amount of movements, especially for the midspan measurement, is probably caused by the stiffness effect described in section 2.2.3 and by a slightly higher excavation depth in the middle of the excavation.

During pile driving, additional movements occurred mostly below elevation - 10 m both perpendicular and parallel to the excavation. Ultimately the movements became so large that INC-5 and INC-10 became unreadable shortly after pile installation. Only INC-4 remained intact to take measurements (Figure 3-8).

Figure 3-8: Inclinometer data A-axis (INC-4)

Figure 3-9: Inclinometer data A-axis (INC-5)

Figure 3-10: Inclinometer data A-axis (INC-10)

It was also noticed that data from piezometers (PZ-1 and PZ-2) showed a response that was likely related to the pile driving activity. The data suggests that the installation of the piles caused pore pressures to increase. According to the pile-driving journal there was a trend where pore pressures increased most when the pile-driving activity was close to the piezometer, whereas there was less, as the pile installation was farther away. This trend was confirmed by Bradshaw et al. (2007), where the pore pressure ratio r_u was calculated from the ratio of the excess pore pressure to the initial vertical effective stress. Since r_u did not reach a ratio of unity ($=1$), there was no indication of liquefaction, although pore pressure ratios of 60 % were calculated. Nonetheless, excess pore pressures dissipated fairly quickly, mostly within a few hours.

Figure 3-11 shows the time history for piezometric head recorded at PZ-1 during pile driving. It can be seen that pore pressures increased rapidly during installation of pile. Also, the trend of increasing excess pore pressures with decreasing distance from pile driving is visible.

Figure 3-11: Time history of piezometric head recorded at PZ-1 (at elevation -21 m)

4. FINITE ELEMENT ANALYSIS

In this chapter a 2-dimensional numerical model is presented that simulates the excavation process and the influence of the pile driving on deformation of the sheet pile walls. The finite element software PLAXIS Version 7 (Plaxis, 1998) was used for the simulations. Measured sheet pile wall deflections (presented in Chapter 3) were used to calibrate the soil parameters that provide best fit between measured and computed deflections. Since the only soil data for this particular site were provided by SPT data of two boring logs, soil strength and stiffness parameters were chosen based on engineering judgment and a rather limited set of geotechnical data.

4.1 FINITE ELEMENT MODEL

4.1.1 FINITE ELEMENT MESH

The dimensions of the finite element mesh were 90 m x 41.6 m (including the embankment). The length of the mesh was chosen to minimize the influence of the left and right boundaries. The boundary effects can be considered to be negligible for a distance greater than $5 \times H$ (H is the excavation depth) (see also Roboski, 2004). The lower boundary on the finite element mesh was set to where the silt layer was approximately underlain by glacial till. For the lower boundary horizontal and vertical fixity was defined. At the left and right boundaries horizontal movement was prevented by fixity, whereas vertical movement could occur.

15-node triangular elements were used to represent the soil. Structural elements like the sheet pile wall or the struts were modeled as elastic materials. The mesh was

refined inside and around the excavation to get more accurate estimates of the deflections.

Since three inclinometer locations (4, 10, 5) existed, three analysis sections were modeled. These are named sections A, B and C. Each section varied slightly in terms of soil layer thickness and excavation steps.

Table 4-1: Design sections

Inclinometer	4	10	5
Design Section	A	B	C

Figure 4-1: Finite element mesh of a typical design section

To generate the finite element mesh described above the following assumptions were made beforehand and verified by several test simulations:

- 1) Using 15-node triangular elements provided more accurate results than the Plaxis default 6-node elements although the calculation time was increased by a factor of 2.5 (the calculation time for each design section was enlarged from 4 minutes to approximately 10 minutes). The 6-node element provides a second

order interpolation for displacements including three Gauss stress points compared to an order of four interpolation and twelve Gauss stress points of the 15 node-element.

- 2) A global coarseness setting influenced the element size. It was also governed by the outer geometry. The standard coarseness setting in Plaxis varied from very coarse (~ 50 elements) to very fine (~ 1000 Elements). For the designs in this study a very fine mesh with local refinements close to the excavation was used. The number of elements varied from 2329 (section A), 2333 (section B), to 2323 (section C).
- 3) Simulating only half of the excavation as a symmetrical problem (here called half-mesh simulation) lead to differing results compared to a full excavation simulation (called full-mesh simulation). The half-mesh simulation would assume a symmetrical problem – in this case an identical embankment at the east side of the excavation, which was not true. Therefore only a full-mesh simulation was appropriate. The deflections calculated at the west and east sides of the excavations were different. However, there were no inclinometer data along the east side of the excavation, therefore the behavior of the east side was not studied further.
- 4) Preliminary simulations showed that accurate modeling of the embankment geometry is significant for calculating realistic wall deflections. It was not appropriate to use a comparable distributed load or even to neglect the presence of the embankment. Additionally, the embankment toe close to the excavation was reduced by a cut of 1 m to simulate a walkway for the construction workers at the site (see Figure 3-6).

- 5) The sheet pile wall was “wished in placed” consequently the installation of the sheet pile was assumed to have no influence on the surrounding soil. This neglects any soil disturbance that may have occurred during installation of the sheet piles.
- 6) Interface elements were used to model the interaction between the sheet pile wall and the soil. In principal the interface element relates wall friction and adhesion to the soil strength by using a strength reduction factor. The PLAXIS manual suggests a factor of 0.67 for steel – sand interaction and a factor of 0.5 for steel - clay. (Plaxis, 1998). The interface reduction factors for all soils were chose to be 0.67 for the simulations in this study.
- 7) The mud mat that was constructed after excavating to final grade was not incorporated in the finite element model. The reason for that was that the influence (gravity load, stiffness element) of the mat was assumed to be small for the in-situ soil behavior, whereas the influence in the finite element model would be very high. In preliminary simulations the soil inside the excavation was “pushed” upwards, but the mud mat – which added additional gravity load – would push the soil back in an excessive manner and affected the results greatly.

4.1.2 CONSTITUTIVE MODEL

Modeling an excavation problem in non-plastic silts was considered to be a drained analysis. That means no pore pressures, caused by rapid loading and low permeable soils, will be generated. At the same time the finite element model accounts for volume changes that are triggered by compression of the voids in the soil. Drained

conditions are especially valid for soils with high permeability. Simulating soil disturbance due to pile driving – which is in fact a fast loading – may need an undrained analysis. Because of a constant soil volume assumed in an undrained simulation, pore pressures can be generated and affect the strength of the soil. However, drained conditions were assumed for the actual excavation process (section 4.2) and the pile driving simulation (section 4.3) (see section 4.1.5 for more details about drained and undrained assumption to simulate pile driving).

The constitutive models used in this study were either the elastic-plastic Mohr-Coulomb model (MC) or the Hardening-Soil model (HS). The first model was used for the fill and the sand layer, whereas the second model was used for the silt layer.

Basic input parameters of the Mohr-Coulomb model were Young's modulus E , Poisson's ratio ν , effective stress friction angle ϕ' , cohesion c' and the dilatancy angle ψ . The stress-strain behavior of the MC-model consists of an elastic part and a plastic part. The elastic part is represented by Hooke's law (linear elastic), whereas the plastic part is defined by a fixed yield surface.

Figure 4-2: Elastic-Perfectly Plastic stress-strain behavior of the Mohr-Coulomb model

The MC-model is a quite simple model, since the elastic-plastic behavior does not represent most soils very accurately (also this model uses an estimated constant average stiffness for each soil layer). Therefore, it was suggested to use this model for quick estimates or when little soil parameters are known (Plaxis, 1998). Because little is known about the fill and sand layers it was decided to use the MC-model for those two layers. This decision was also made since the fill and sand layer did not have an important role for the purpose of this study.

The HS-model uses a more advanced approach to simulate soil behavior. The Hardening-Soil Model is a non-linear hyperbolic model similar to the well-known Duncan-Chang model (Schanz et al., 1999). Basic input parameters are stiffness for primary loading E_{50}^{ref} , stiffness for primary compression E_{oed}^{ref} , stiffness for un-/reloading E_{ur}^{ref} , stress dependent stiffness according to a power law m and the basic parameters c' , ϕ' , Ψ . In contrast to the MC-model, the yield surface is not fixed in principal stress space, but can expand due to plastic straining (Plaxis, 1998). This is called “hardening” and consists of two main types: shear hardening due to primary deviatoric loading and compression hardening to primary compression in oedometer loading and isotropic loading.

Figure 4-3: Hyperbolic stress-strain relation in primary loading (Plaxis, 1998)

The stiffness moduli used for the HS-model are stress dependent and can be calculated with:

$$E_{50} = E_{50}^{ref} \left(\frac{c \cot \varphi - \sigma_3'}{c \cot \varphi + p^{ref}} \right)^m \quad (4.1)$$

and

$$E_{ur} = E_{ur}^{ref} \left(\frac{c \cot \varphi - \sigma_3'}{c \cot \varphi + p^{ref}} \right)^m \quad (4.2)$$

Actual values of modulus consequently depend on the minor principal stress σ_3' which can be determined by a triaxial test and also depends on a reference confining pressure p^{ref} which is usually 100 kPa. The power m shows the amount of stress dependency. PLAXIS suggests using m around 0.5 for normal soils and increase m to 1.0 for soft soils. For most calculations in this study it was assumed $E_{oed}^{ref} = E_{50}^{ref}$ and $E_{ur}^{ref} = E_{50}^{ref}$.

4.1.3 INPUT PARAMETERS

Finding the right input parameters was hindered by the limited set of soil data. A geotechnical boring was performed at the north (BS98-7) and south (BD98-12) end of the excavation to investigate soil properties. The distribution of uncorrected blow counts (SPT-values) is presented in Figure 4-4.

Figure 4-4: Blowcounts measured at boring log BS98-7 (left) and BD98-12 (right)

The blowcounts, N , presented in Figure 4-4 were used to estimate initial input parameters γ , φ and K_o :

Table 4-2: Initial estimate of input parameters

BS 98-7				
	Fill	Sand	Silt	Source
Depth in m	0 - 3.2	3.2 - 4.3	4.3 - 13.7	Boring log 01/22/99
N	15.5	7	17.8	Boring log 01/22/99
γ assumed (kN/m ³)	19	18.4	19.5	Holtz, Kovacs, Sheahan 2010
Overburden pressure σ' (kPa)	47.1	56.3	145.6	including watertable
CN	1.2	1.1	0.85	(Hannigan et al.1998) F 4.4
N' (CN*N)	18.6	7.7	15.1	
Wet soil unit weight (kN/m³)	18.6	16.6	18.1	(Hannigan et al.1998) Tb 4.5
Dry soil weight (kN/m³)	16.5	15	16	Holtz, Kovacs, Sheahan 2010
Friction angle (°)	32.15	29.4	31.3	(Hannigan et al.1998) Tb 4.5
Ko	0.47	0.51	0.48	Ko = 1-sin ϕ

BD98-12				
	Fill	Sand	Silt	Source
Depth in m	0 - 2.1	2.1 - 7.2	7.2 - 16.8	Boring log 01/22/99
N	4	5.7	7.0	Boring log 12/15/98
γ assumed (kN/m ³)	17	18	18.5	Holtz, Kovacs, Sheahan 2010
Overburden pressure σ' (kPa)	32.7	73.5	154.7	including watertable
CN	1.3	1.05	0.8	(Hannigan et al.1998) F 4.4
N' (CN*N)	5.2	6	6.3	
Wet soil unit weight (kN/m³)	14.9	15.4	15.6	(Hannigan et al.1998) Tb 4.5
Dry soil weight (kN/m³)	14	13	15	Holtz, Kovacs, Sheahan 2010
Friction angle (°)	28	28.6	29	(Hannigan et al.1998) Tb 4.5
Ko	0.53	0.52	0.51	Ko = 1-sin ϕ

The estimated parameters were then compared to soil parameters that were suggested in the “Geotechnical Base Line Report” (Haley and Aldrich, Jacobs Civil, 2002). The report assumed three soil layers for the entire construction side without any variances between the north and the south side of the excavation.

Table 4-3: Soil Parameter for initial design (from Haley and Aldrich, Jacobs Civil, 2002)

Soil Parameter (Geotechnical Base Line Report)						
Soil	γ (kN/m³)	γ' (kN/m³)	ϕ (°)	passive ko	active ko	kp
Fill	18.9	9.1	32	0.47	0.31	3.26
Alluvial and Estuarine Deposits	18.1	8.3	24	0.59	0.42	2.37
Glaciolacustrine Deposits	19.6	9.9	32	0.47	0.31	3.26

Because the initial estimates differed so much, it was decided to use a certain set of parameters for each design section including modified values from the initial estimates and Geotechnical Base Line Report soil parameters. These values are shown in Table 4-4.

Table 4-4: Initial input parameters for all design sections

Design Section A			
	Fill	Sand	Silt
Wet soil unit weight (kN/m ³)	18.9	18.1	19.6
Dry soil weight (kN/m ³)	16.5	15.5	16.5
Friction angle (°)	32	30	32
Dilatancy Angle (°)	0	0	0
Ko	0.47	0.5	0.47
Permeability kx / ky (m/day)	1 / 1	0.5 / 0.5	0.5 / 0.5
Poisson's Ratio	0.3	0.3	0.3
Interface Reduction Factor R _{inter}	0.67	0.67	0.67

Design Section B			
	Fill	Sand	Silt
Wet soil unit weight (kN/m ³)	18.9	18.1	19.6
Dry soil weight (kN/m ³)	16.5	15.5	16.5
Friction angle (°)	31	29	31
Dilatancy Angle (°)	0	0	0
Ko	0.47	0.53	0.47
Permeability kx / ky (m/day)	1 / 1	0.5 / 0.5	0.5 / 0.5
Poisson's Ratio	0.3	0.3	0.3
Interface Reduction Factor R _{inter}	0.67	0.67	0.67

Design Section C			
	Fill	Sand	Silt
Wet soil unit weight (kN/m ³)	18.9	18.1	19.6
Dry soil weight (kN/m ³)	16.5	15.5	16.5
Friction angle (°)	30	28	30
Dilatancy Angle (°)	0	0	0
Ko	0.5	0.53	0.5
Permeability kx / ky (m/day)	1 / 1	0.5 / 0.5	0.5 / 0.5
Poisson's Ratio	0.3	0.3	0.3
Interface Reduction Factor R _{inter}	0.67	0.67	0.67

Parameters for horizontal and vertical permeability and Poisson's ratio were taken from values found in the literature (e.g. Holtz, Kovacs and Sheahan, 2010). Identical unit weights for all design sections were chosen to minimize the gravity effect (see also section 4.1.4) and to provide reproducibility. Nevertheless, to account for the different soil densities investigated in the two boring logs, the effective stress friction angles and subsequently the K_o values were varied.

Furthermore, stiffness parameters (E , E_{50}^{ref} , E_{oed}^{ref}) had to be estimated. There are some methods in the literature to determine E using SPT-values. Most methods use corrected SPT-values called N_{60} , which can be calculated as

$$N_{60} = \frac{E_m * C_B * C_S * C_R * N}{0.60} \quad (4.3)$$

with:

- E_m – hammer efficiency
- C_B – borehole diameter correction
- C_S – sampler correction
- C_R – rod length correction

For this study those values were:

Table 4-5: Correction factors for blowcounts

Factor	BS98-7	BD98-12	Comment
E_m	0.6	0.6	Safety Hammer
C_B	1	1	Borehole diameter 10 cm
C_S	1.2	1.2	Generally used for sampler
C_R	1	1	Rod length

Table 4-6: Corrected blowcounts

Blowcounts	BS98-7			BD98-12		
	Fill	Sand	Silt	Fill	Sand	Silt
N	15.5	7.0	17.8	4.0	5.7	7.0
N_{60}	18.6	8.4	21.4	4.8	6.8	8.4

Typical values and calculated values for Young's moduli:

Table 4-7: Typical and calculated values for Young's moduli

Method / Source	Used SPT Value	BS98-7			BD98-12		
		E (Fill) (kN/m ²)	E (Sand) (kN/m ²)	E (Silt) (kN/m ²)	E (Fill) (kN/m ²)	E (Sand) (kN/m ²)	E (Silt) (kN/m ²)
API 1110-1-1904 Appendix D	-	23940 - 95760	9576 - 23940	23940 - 191520	23940 - 95760	9576 - 23940	23940 - 191520
Bowles, 1997: Table 5-6	N	15250	11000	7140	9500	10350	3900
Bowles, 1997: Table 2-8	-	50000 - 81000	10000 - 25000	2000 - 20000	50000 - 81000	10000 - 25000	2000 - 20000
Florida Pier Manual, 1996	N ₆₀	17825	8050	10182	4600	6516	8050
Poulos, 1994: Table 2	N	38750	17500	44500	10000	14250	17500
Braja, 2007: Table 5-8	-	17250 - 27600	10500 - 25000	10350 - 17250	17250 - 27600	10500 - 25000	10350 - 17250
Braja, 2007: Equation 5.43	N ₆₀	18600	8400	10700	4800	6800	4200

Typical stiffness parameter for E_{50}^{ref} , E_{oed}^{ref} or correlations between those parameters and SPT-values are harder to find in the literature. Tjie-Liong (2011) recommended the following correlation for silty and clayey soils:

$$E_{oed} = 303 * N_{60} \quad (4.4)$$

$$E_{50}^{ref} = 292 * N_{60} \quad (4.5)$$

Using Equations 4.4 and 4.5 stiffness parameters for the north side of the excavation were calculated as 6484 kN/m² (E_{oed}^{ref}) and 6248 kN/m² (E_{50}^{ref}) compared to values for the south side with 2545 kN/m² (E_{oed}^{ref}) and 2453 kN/m² (E_{50}^{ref}). However, it is part of this study to find parameters that represent the existing site. Consequently, estimates from the literature were only used for basic computations and were changed in a “trial and error” method until fitting.

Besides appropriate soil elements, structural elements were included in the model to simulate sheet pile wall and struts. The stiffness of those structural elements was calculated by means of drawings where dimensions and material were described. The sheet pile wall consists of CZ-128 sheet piles and was simulated in PLAXIS as a

beam element defined by a bending stiffness EI and a normal stiffness EA . Struts were simulated as node-to-node anchor (elastoplastic spring elements with two fixed ends on either side of the excavation wall) and a defined normal stiffness EA . Different dimensions of struts were used for the three strut levels, therefore the normal stiffness had to be adjusted for each level (see Table 4-8). The horizontal strut spacing was 8.1 m whereas the vertical spacing was 3.3 m (1. level to 2. level) and 2.1 m (2. level to 3. level).

Table 4-8: Stiffness of structural elements

Structural Element	Bending Stiffness - EI (kNm^2/m)	Normal Stiffness - EA (kN/m)	Element Type
Sheet Pile Wall CZ-128	6.46×10^4	3.25×10^6	Beam
1. Strut Level (W14X90)	-	3.42×10^6	Anchor
2. + 3. Strut Level (W14X120)	-	4.56×10^6	Anchor

4.1.4 SIMULATION PROCESS FOR EXACATION

The simulation process in PLAXIS should represent the in-situ excavation process as presented in Figure 3-5. In general, the simulation started with installing the sheet pile walls by activating the beam elements in the model and was followed by alternating steps of excavation and strut installation. Deactivating the soil cluster in the finite element model simulated an excavation process. Activating the node-to-node anchor simulated struts installation. The detailed construction/simulation activity for each designs section will be shown in section 4.2.

In PLAXIS each of the steps described above was simulated by a plastic calculation (no time effect included). The loading for calculating deformations in the finite element model occurred because of using a staged construction procedure. That

means, changing geometry configurations lead to a changed ultimate state (equilibrium) that had to be calculated by the finite element program.

Figure 4-5: Typical simulation steps in PLAXIS

The embankment at the west side of the excavation encountered some problems since it created an asymmetrical problem. Because of the higher load caused by the embankment the entire soil profile was shifting to the east and caused large movements even before the excavation was started. Therefore it was decided to include an initial simulation step where only gravity loading was calculated. For this step, no structural elements were activated or soil was deactivated, just the embankment was allowed to “settle” (this process was not a real settlement calculation since no pore pressure change was allowed). The goal was to create a certain stress history for the soil. After doing this, all displacements (not the stresses) were reset to

zero and the next simulation step was applied by activating the sheet pile walls. By doing this procedure the effect of the asymmetrical problem could be reduced.

Groundwater conditions were also simulated in the analysis. The initial water table was situated at elevation 1.8 m, but it decreased because of pumping water out inside the excavation. A “phreatic line” defined the water table level at the beginning of the simulation and was used to calculate initial water pressure. During excavation a prescribed groundwater head was used as left and right boundary conditions (here 1.8 m). Because of the staged construction method and defined impermeable sheet pile walls no water was assumed to be in the excavated areas inside the excavation. Finally, using a groundwater flow calculation could simulate a change in groundwater table and changed water pressure (Figure 4-6).

Figure 4-6: Pore water pressure due to groundwater flow calculations

4.1.5 SIMULATION PROCESS FOR PILE DRIVING

The overall scope of this study was to simulate the soil behavior of Rhode Island Silts due to pile driving. However, simulating the process of pile driving was not trivial. The problem was, driving piles into the ground cannot be simulated with

PLAXIS directly. Even if it was possible to model piles by means of structural elements, it was not possible to simulate a dynamic motion. Therefore, a special approach was used to solve this problem.

In principle, this approach did not attempt to simulate the pile driving itself, but rather the immediate effect of pile driving on the surrounding soil properties. It is well known in the literature that pile driving installation can lead to degradation of soil strength and stiffness. This effect is called liquefaction. There are numerous definitions of the phenomena, but most of them describe it as a reduction in effective stress due to pore pressure generation leading to loss of strength and stiffness (Taylor, 2011; Wu et al., 2004).

Two different kinds of liquefaction can be distinguished. They depend on the state of the granular soil (contractive or dilative). When the static shear stress of a soil is greater than the shear strength of that soil in a liquefied state then this is called *flow liquefaction*. Usually this can happen in cohesionless soils (sands and non-plastic silts). Applying cyclic loads (like pile driving) bring the soil to an unstable state, which causes a dramatic reduction in strength. This happens suddenly and causes large deformations (Kramer, 1996; Idriss and Boulanger, 2008). The second possibility of liquefaction is called *cyclic mobility*. This occurs when the static shear stress is less than the shear strength of the liquefied soil. Each cycle of load produces a gradually increase of strain and it is driven by cyclic and average shear stresses. Deformations increase proportionally and signalize failure when the strains are unacceptable large. This kind of liquefaction is common in cohesive soils (clays and plastic silts) (Kramer, 1996; Idriss and Boulanger, 2008).

For the analysis of dynamic degradation the effective strength of the soil element governs the behavior and degradation of the medium. In a standard cyclic triaxial test pore pressures develop due to a lack of drainage within the specimen and as the effective stress decreases the stiffness decreases, which can be seen in increasing strain. Figure 4-7 and Figure 4-8 present typical cyclic triaxial test results, including an increasing pore pressure ratio until reaching $r_u = \Delta u / \sigma' = 1$ as failure criterion.

Figure 4-7: Typical deviator stress vs. axial strain hysteresis loop from a stress controlled cyclic triaxial test (Taylor, 2011)

Figure 4-8: Typical pore pressure increase with resulting increase in pore pressure ratio for stress controlled cyclic triaxial tests (Taylor, 2011)

As shown in Figure 3-11 excess pore pressures were measured during pile driving and therefore supports the assumption of decreasing effective stress with increasing excess pore pressures. The question arises how to relate the loss of strength

to the build-up of excess pore pressures? In general the liquefaction potential is evaluated by relating the cyclic shear stress induced by the source (earthquake, pile driving activity) to the cyclic resistance (Idriss and Boulanger, 2008).

For this study it was not necessary to calculate any cyclic shear stresses or cyclic resistance, but to assess the strength loss due to pile driving itself. Parameters that could be reduced in PLAXIS to simulate strength loss were the effective stress friction angle ϕ' and the stiffness E , E_{oed}^{ref} , E_{50}^{ref} and E_{ur}^{ref} . Also, there are many attempts in the literature to relate cyclic loading to soil properties, mainly to bulk moduli, K , and shear moduli, G , that can be correlated to Young's modulus and the oedometer modulus (Wood, 1990; Plaxis, 1998). Nevertheless, the approach of this study is to estimate appropriate values of moduli and effective stress friction angle first using a trial and error method and then verifying the optimized parameters later.

In this study, calculations in PLAXIS were executed as effective stress analyses; consequently the input parameters were effective stress parameters. The soil was assumed to behave as a drained material. In contrast, to simulate excess pore water pressure caused by pile driving an undrained soil behavior should have been selected. But since the excess pore pressures dissipated relatively quickly after pile driving the time dependency was important. Unfortunately, simulating this time dependency was not possible with a staged construction, which had to be used to change the soil parameters (consolidation simulation could have been chosen instead, but this did not account for plastic deformations). Additionally, defining a value of excess pore pressures for certain soil clusters was not possible in PLAXIS. Consequently, the only

possibility to simulate the effect of excess pore water pressure due to pile driving was to execute a drained analysis with reduced effective stress parameters.

4.1.6 3-D EFFECT

As described in section 2.2.3 the 3-D effect could have a huge influence on the simulation results, when using a 2-D simulation. Finno et al. (2007) explained the 3-D effect as a function of excavation depth and dimensions of excavations. As a result, a plane strain ratio, PSR, could be calculated – defined as deflection computed in the midspan area of the excavation by a 3-D simulation, divided by deflection computed with plane strain simulation (see Equation 2.2). Since the excavation profile in this study is not rectangular two cases of L/B ratio (see Figure 4-9 and Figure 4-10) were investigated.

Figure 4-9: Small L/B ratio (case A)

Figure 4-10: Big L/B ratio (case B)

The L/B ratio for case A was $\frac{15.5}{9.75} = 1.6$ and for case B $\frac{45}{9.75} = 4.6$. Figure 4-11 shows the effect of plan dimensions L/B on the PSR. Case A represented a PSR of 0.7 – 1.0 what means that 2-D simulations would over predict deflections about 42 %. In contrast, the PSR of case B is around 1, therefore 2-D simulations would result in similar results like 3-D simulations.

Figure 4-11: Effect of plan dimensions on PSR (after Finno et al., 2007)

Since the PSR of case A is smaller than 4, a length to excavation depth ratio L/H_e had to be taken into consideration to determine a more accurate PSR. Each excavation step (simulation stage) therefore had a certain H_e . Figure 4-12 displays the PSR for different excavation steps of design section B (since this was the midspan location). An L/H_e ratio greater than 6 resulted in a PSR of around 1. In contrast, very small PSR were reached for L/H_e ratios smaller than 2, which indicated large differences between plane strain and 3-D simulations.

Figure 4-12: Effect of excavation depth on PSR (after Finno et al., 2007)

Consequently, when assuming a smaller L/B ratio like in case A the plane strain simulation would over predict the deflections especially when reaching the 3rd simulation stage. Assuming a bigger L/B ratio like case B, more reliable plane strain calculated deflections could be determined.

In principle, the conical shaped excavations towards the south and north end of the excavations cannot be treated as perpendicular sheet pile walls (like assumed in case A) related to the simulated west sheet pile wall. This supports the assumption

that case B was more reliable and more accurate. Consequently the 2-D simulation done in this study was assumed to lead to similar results like a 3-D simulation.

4.2 RESULTS OF EXCAVATION SIMULATION

The following section presents the results of the excavation simulation performed with PLAXIS. For each design section the simulation steps including computed deflections and the deformed finite element mesh (appendix A) are shown. Also the optimized soil parameters necessary to match simulated with measured movements are summarized.

The soil parameters presented in Table 4-4 were used as default values for the simulation. The only parameters that were adjusted during the simulations were the moduli and the effective stress friction angles of the soils. The moduli of the fill and sand layer were, after an initial adjustment, kept constant, and the modulus and effective stress friction angle of the silt layer was decreased with progressing excavation. This was done to account for some amount of soil disturbance that may have occurred in the silt during excavation (Russell, 2011). Therefore, an area up to 2 m away from the sheet pile walls (east and west) was characterized as disturbed area. Note that the magnitude of disturbance might differ in this area. For example, the soil inside the excavation could be more disturbed (because of heavy equipment) than the outside area. Further effects of soil disturbance will be discussed in section 4.4.

The design sections used in this study are presented in the following figure.

Figure 4-13: Design sections

In the field, the locations of the inclinometers were approximately 1 m away from the west sheet pile wall, respectively. That means the deflections shown in Figure 3-8, Figure 3-9 and Figure 3-10 might not exactly represent the deflections of the sheet pile wall itself. Taking this into consideration, simulations were run to estimate both deflections at the inclinometer location and at the sheet pile wall. Figure 4-14 shows how simulating the displacements at the inclinometer location with optimized soil parameters lead to slightly different results compared to simulated displacements at the sheet pile wall when using the same set of parameters.

Figure 4-14: Example of computed deflections, optimized only for inclinometer location

4.2.1 DESIGN SECTION A

The excavation stages simulated at design section A are shown in Figure 4-15. It shows that the 3rd and final excavation stages were simulated in one step. This was done based on field reports that indicated that the contractor excavated the last two stages in one step.

Figure 4-15: Excavation simulation of design section A

Also, it was not clear whether the struts were pre-stressed at the construction site or not. At some sites this is a common method to reduce wall movements. The deflections measured at the inclinometers indicated that some kind of recovery of the sheet pile wall took place. Therefore, in the simulation the first strut level and the second strut level were subjected to a pre-stress load of 80 kN and 20 kN, respectively. After doing this the deflection curves provided a much better fitting with measured curves compared to simulations without pre-stressing the struts.

Plots of the staged calculation and the respective deformed mesh are shown in Appendix A. Here in this section only the final results will be presented. The optimized deflection curves for design section A can be plotted as following:

Figure 4-16: Optimized deflection curves of section A

The optimized soil parameters for this design section were found to be:

Table 4-9: Optimized parameter of design section A

	Parameter	Inclinometer optimized			Beam optimized		
		Fill	Sand	Silt	Fill	Sand	Silt
Initial + 1. Stage	E_{ref}	25000	20000	-	25000	20000	-
	E_{50}^{ref}	-	-	35000	-	-	35000
	E_{oed}^{ref}	-	-	35000	-	-	35000
	E_{ur}^{ref}	-	-	105000	-	-	105000
	ϕ	32	30	32	32	30	32
	m	-	-	0.5	-	-	0.5
2. Stage	E_{50}^{ref}	-	-	14000	-	-	16000
	E_{oed}^{ref}	-	-	14000	-	-	16000
	E_{ur}^{ref}	-	-	42000	-	-	48000
	ϕ	32	30	26	32	30	26
	m	-	-	0.5	-	-	0.5
3. Stage + Final Stage	E_{50}^{ref}	-	-	500	-	-	700
	E_{oed}^{ref}	-	-	575	-	-	806
	E_{ur}^{ref}	-	-	1000	-	-	1400
	ϕ	32	30	14	32	30	14
	m	-	-	1.0	-	-	1.0

stiffness moduli in kN/m_2 friction angle in $^\circ$

Table 4-9 shows that the moduli of the silt had to be decreased by a significant amount. In detail, the 2nd stage silt moduli were reduced to 40 % (inc. optimized) and 46 % (beam optimized) the initial value. Additionally the friction angle was decreased to 80 %. To simulate deflections measured for the 3rd stage and final stage the moduli had to be set to 1.4 % (inc. optimized) and 2 % (beam optimized) for some clusters. The friction angle was decreased to 44 %. Also, the power value m was increased to 1.0 to simulate a very soft soil (as suggested in Plaxis, 1998).

It has to be noted that different soil areas depending on their location relative to the “working area” could be subjected to different amounts of soil disturbance. For example, the silt around the sheet pile toe remained undisturbed (see Figure 4-15), whereas the silt right underneath the excavated area and outside the sheet pile wall was assumed to be heavily disturbed. In general, the closer the silt was to the “working

area”, the more disturbance was assumed. The exact location of disturbed areas can be seen in the plots presented in appendix A.

4.2.2 DESIGN SECTION B

The excavation stages simulated at design section B were:

Figure 4-17: Excavation simulation of design section B

For this design section four stages were simulated. The construction report mentioned those separate stages, therefore it was implemented in the simulation. It should be noted that the soil profile had changed compared to section A.

Also, the strut pre-stress was included in this simulation as described in the section before.

Then optimized curves are:

Figure 4-18: Optimized deflection curves of section B

As shown in Figure 4-18, the Stage 1 and 2 were simulated well. In contrast, Stages 3 and 4 produced some problems to simulate them correctly. It was solved by adding additional area of disturbance below the already assumed area of disturbance. Because section B was the midspan location more deflection was expected. Also, the excavation depth was 0.5 m deeper. Therefore, the deflections are double the amount measured at sections A and C. However, as shown in the following table the soil disturbance was assumed to be very high and could not be increased more without creating problems in the finite element calculation.

Table 4-10: Optimized parameter of design section B

	Parameter	Inclinometer optimized			Beam optimized		
		Fill	Sand	Silt	Fill	Sand	Silt
Initial + 1. Stage	E_{ref}	20000	18000	-	20000	18000	-
	E_{50}^{ref}	-	-	15000	-	-	15000
	E_{oed}^{ref}	-	-	15000	-	-	15000
	E_{ur}^{ref}	-	-	45000	-	-	45000
	ϕ	31	29	31	31	29	31
	m	-	-	0.5	-	-	0.5
2. Stage	E_{50}^{ref}	-	-	7000	-	-	7000
	E_{oed}^{ref}	-	-	7000	-	-	7000
	E_{ur}^{ref}	-	-	21000	-	-	21000
	ϕ	31	29	27	31	29	27
	m	-	-	0.5	-	-	0.5
3. Stage + Final Stage	E_{50}^{ref}	-	-	100	-	-	100
	E_{oed}^{ref}	-	-	115	-	-	115
	E_{ur}^{ref}	-	-	200	-	-	200
	ϕ	31	29	14	31	29	14
	m	-	-	1.0	-	-	1.0

stiffness moduli in kN/m_2 friction angle in $^\circ$

Table 4-10 shows that the moduli of the silt had to be decreased by a significant amount. Note that the inclinometer and the beam optimization resulted in the same parameters. To account for a decreasing soil stiffness from the north end to the south end of the excavation the initial soil stiffness for all three layers is smaller than the parameters used in design section A. The 2nd Stage silt moduli were reduced to 47 % the initial value. Additionally, the friction angle was decreased to 87 %. To simulate deflections measured for the 3rd Stage and Final Stage the moduli had to be set to 0.7 %. The friction angle was decreased to 45 %. Also, the power value m was increased to 1.0 to simulate a very soft soil (as suggested in Plaxis, 1998). Furthermore, an additional area of disturbance with reduced modulus of 47 % and reduced friction angle to 87 % had to be included below the already existing area (see Figure 4-17).

4.2.3 DESIGN SECTION C

The excavation stages simulated at design section C were:

Figure 4-19: Excavation simulation of design section C

Note that the 3rd Stage and the Final Stage were excavated in one step. Thus this had to be simulated as well. Also, the soil profile changed again. Strut pre-stress was used as described in the sections before.

Figure 4-20: Optimized deflection curves of section C

Table 4-11: Optimized parameter of design section C

		Inclinometer optimized			Beam optimized		
Parameter		Fill	Sand	Silt	Fill	Sand	Silt
Initial + 1. Stage	E_{ref}	15000	12000	-	15000	12000	-
	E_{50}^{ref}	-	-	30000	-	-	30000
	E_{oed}^{ref}	-	-	30000	-	-	30000
	E_{ur}^{ref}	-	-	90000	-	-	90000
	ϕ	30	28	30	30	28	30
	m	-	-	0.5	-	-	0.5
2. Stage	E_{50}^{ref}	-	-	15000	-	-	14000
	E_{oed}^{ref}	-	-	15000	-	-	14000
	E_{ur}^{ref}	-	-	45000	-	-	42000
	ϕ	30	28	26	30	28	26
	m	-	-	0.5	-	-	0.5
3. Stage + Final Stage	E_{50}^{ref}	-	-	500	-	-	3000
	E_{oed}^{ref}	-	-	575	-	-	3000
	E_{ur}^{ref}	-	-	1000	-	-	9000
	ϕ	30	28	14	30	28	15
	m	-	-	1.0	-	-	0.5

stiffness moduli in kN/m_2

friction angle in $^\circ$

Table 4-11 shows that the moduli of the silt had to be decreased by a significant amount. To account for a decreasing soil stiffness from the north end to the south end

of the excavation the initial soil stiffness for the fill and sand layer is smaller than the parameters used in design section A and B. The 2nd Stage moduli were reduced to 50 % (inc. optimized) and 47 % (beam optimized) the initial value. Additionally the friction angle was decreased to 87 %. To simulate deflections measured for the 3rd Stage and Final Stage the moduli had to be set to 1.7 % (inc. optimized) and 10 % (beam optimized). The friction angle was decreased to 47 % (inc. optimized) and 50 % (beam optimized). Also, the power value m was increased to 1.0 to simulate a very soft soil (as suggested in Plaxis, 1998). This was only done for the inclinometer optimization. The beam optimization did not need a power reduction to simulate soft soil.

4.2.4 SUMMARY OF EXCAVATION SIMULATION

Moduli and effective stress friction angle optimizations had been made for the “real” inclinometer location (1 m from the sheet pile wall) and the sheet pile wall itself. Appropriate soil parameters could be found to simulate the deflection of the first and second excavation stage, whereas the moduli and effective stress friction angle for the third and final stage of construction are not reasonable.

Because of soil disturbance the stiffness moduli and the friction angles of the silt layer had to be reduced for each excavation stage to match measured curves. However, decreasing the soil stiffness up to 99 % is very unrealistic. This fact and other explanations for this issue will be discussed in section 4.4. The following table summarizes the parameters found in this study:

Table 4-12: Optimized parameter summary for silt

	Inclinometer Optimized								
	A - E_{50}^{ref}	A - E_{ur}^{ref}	A - ϕ	B - E_{50}^{ref}	B - E_{ur}^{ref}	B - ϕ	C - E_{50}^{ref}	C - E_{ur}^{ref}	C - ϕ
1. Stage	35000	105000	32	15000	45000	31	30000	90000	30
2. Stage	14000	42000	26	7000	21000	27	15000	45000	26
3. Stage + Final Stage	500	1000	14	100	200	14	500	1000	14

	Beam Optimized								
	A - E_{50}^{ref}	A - E_{ur}^{ref}	A - ϕ	B - E_{50}^{ref}	B - E_{ur}^{ref}	B - ϕ	C - E_{50}^{ref}	C - E_{ur}^{ref}	C - ϕ
1. Stage	35000	105000	32	15000	45000	31	30000	90000	30
2. Stage	16000	48000	26	7000	21000	27	14000	42000	26
3. Stage + Final Stage	700	1400	14	100	200	14	3000	9000	15

The difference between the optimized parameters in each stage is illustrated in Figure 4-21 for the soil parameters and Figure 4-22 for the beam parameters.

Figure 4-21: Plot of soil parameters at each stage of excavation optimized to match measured inclinometer data (inclinometer location)

Figure 4-22: Plot of soil parameters at each stage of excavation optimized to match measured inclinometer data (beam location)

4.3 RESULTS OF PILE DRIVING SIMULATION

The overall scope of this study was to simulate soil movements due to pile driving. This section presents the results of a parameter optimization to fit finite element simulated curves with measured deflection curves. Calculations were made based on the assumptions presented in section 4.1.5.

It was decided to use a more detailed silt layer system below the already existing area of disturbance due to excavation (see Figure 4-23, Figure 4-28 and Figure 4-31). Consequently each “sub”-layer could be subjected to a different amount of soil disturbance. During the simulation process adjustments were made for each layer and the modulus was decreased in step sizes of 5 %. Below an absolute value of 5 % the step size was decreased to 1 %.

As discussed in section 4.1.1 the mud mat was not included in the simulation. Doing this would probably result in lesser silt strength and stiffness parameter than presented below, because the mat would push the soil downwards (gravity) and therefore reduced the sheet pile wall moving. Consequently, to obtain the measured deflection curves, even larger reductions in soil properties would be required.

4.3.1 DESIGN SECTION A

As described above, decreasing the soil strength and stiffness simulated pile driving. The area subjected to disturbance is shown in Figure 4-23. It expands below the tip elevation of the sheet piles and had the same distance from the sheet pile walls (2 m) like the areas disturbed by the excavation.

Figure 4-23: Pile driving simulation of design section A

As for the excavation model it was necessary to simulate and optimize parameters for the true inclinometer location and the wall location, respectively.

The amount of disturbance (shown as percentage of the initial silt stiffness) that was necessary to match the observed deflections is shown in Figure 4-24. A comparison of the simulated displacements and measured data is shown in Figure 4-25.

Figure 4-24: Reduced soil strength after pile driving (beam)

Figure 4-25: Deflection curve for optimized soil parameter (beam)

To simulate the measured deflections caused by pile driving a more detailed soil setup compared to the excavation setup was necessary. The soil setup shown in Figure 4-24 finally lead to the deflections presented in Figure 4-25. It was not possible to optimize the deflection curves for the sheet pile wall in a satisfactory manner. In particular the peak movements below elevation -10 m could not be reproduced well with the simulation. Nevertheless, it could be shown that reducing the soil strength and stiffness in the areas of pile driving activity increased the deflections of the sheet pile wall.

As presented in Table 4-13 the stiffness and strength parameter of the silt were not reduced as much as deeper elevations. Since the sheet pile wall ended at elevation -11.9 m it was necessary to provide a certain amount of resistance against moving. Using lower values than presented above, would have caused the lower part of the sheet pile wall to move excessively.

Table 4-13: Optimized parameter for pile driving of design section A (beam)

Elevation (m)	E_{50}^{ref}	E_{oed}^{ref}	E_{ur}^{ref}	ϕ	m
-4.6 to -6.5	700	806	1400	14	1
-6.5 to -9.0	700	806	1400	14	1
-9.0 to -11.0	1000	889	2000	18	1
-11.0 to -13.0	16000	16000	48000	26	0.5
-13.0 to -15.0	16000	16000	48000	26	0.5

stiffness moduli in kN/m_2

friction angle in $^\circ$

In contrast to the beam-optimized simulation, much better results could be determined for the inclinometer location using the soil profile shown in Figure 4-26.

Figure 4-26: Reduced soil strength after pile driving (inclinometer location)

The main difference between the soil setup presented for the inclinometer location and the sheet pile wall was the reduced soil strength at elevation -13 m to -15 m. A summary of the reduced soil parameters is shown in Table 4-14, and a comparison between the simulated displacements and the measured data is shown in Figure 4-27.

Table 4-14: Optimized parameter for pile driving of design section A (inclinometer location)

Elevation (m)	E_{50}^{ref}	E_{oed}^{ref}	E_{ur}^{ref}	ϕ	m
-4.6 to -6.5	700	806	1400	14	1
-6.5 to -9.0	1750	2016	3500	14	1
-9.0 to -11.0	700	806	1400	14	1
-11.0 to -13.0	16000	16000	48000	26	0.5
-13.0 to -15.0	3500	3500	10500	26	0.5

stiffness moduli in kN/m_2
friction angle in $^\circ$

Using these parameters the following deflection curve was obtained:

Figure 4-27: Deflection curve for optimized soil parameter (inclinometer location)

Shown in Figure 4-27 is a much better fitting deflection curve for the inclinometer location. Even the movements at elevation -10 m could be simulated quite well. As before, the strength reduction right at the toe of the sheet pile wall could not exceed a certain amount (here 70 % reduction). When using smaller values excessive movement would have occurred.

4.3.2 DESIGN SECTION B

According to construction field reports, inclinometer 10 became unreadable shortly after the beginning of pile driving. Consequently no measured deflections exist to use for optimizing parameters. However, in this study the parameters determined for design section A were applied for this design sections to investigate possible deflection caused by pile driving. Since only the inclinometer location optimized parameters

resulted in acceptable deflection curves (Figure 4-26), only this percentage of strength reduction was incorporated in design section B as initial parameters.

Figure 4-28: Pile driving simulation of design section B

Unfortunately, it was not possible to use the same percentage of reduction as design section A, since the soil strength would have been too low to provide any resistance. Deflections of more the 0.5 m were obtained by those parameters. Therefore the parameters had to be changed until at least some usable deflection curves were obtained. A summary of the reduced soil parameters is shown in Table 4-

15, and a comparison between the simulated displacements and the measured data is shown in Figure 4-30.

Figure 4-29: Reduced soil strength after pile driving

Table 4-15: Optimized parameter for pile driving of design section B

Elevation (m)	E_{50}^{ref}	E_{oed}^{ref}	E_{ur}^{ref}	ϕ	m
-4.6 to -6.5	100	115	200	14	1
-6.5 to -9.0	100	115	200	14	1
-9.0 to -11.0	5250	5250	15750	26	0.5
-11.0 to -13.0	7000	7000	21000	26	0.5
-13.0 to -15.0	5250	5250	15750	26	0.5

stiffness moduli in kN/m_2

friction angle in $^\circ$

Figure 4-30: Deflection curve for optimized soil parameter

The shape of the curve presented in Figure 4-30 shows reasonable agreement with measured results of design section A. Specifically, the peak at elevation -10 m could be simulated well. However, since the deflections at design section B are almost twice that high as design section A, the only goal here was to find a curve that has the right shape not necessarily the right amount of deflection.

4.3.3 DESIGN SECTION C

As with design section B, inclinometer 5 became unreadable shortly after the beginning of pile driving. No measured deflections existed to use for optimizing parameters at design section C. Consequently, optimized parameters (inclinometer location) from design section A were applied to design section C.

Figure 4-31: Pile driving simulation of design section C

A summary of the reduced soil parameters is shown in Table 4-16, and a comparison between the simulated displacements and the measured data is shown in Figure 4-32.

Figure 4-32: Deflection curve using same parameters as for design section A

Although the shape of the deflection looks reasonable (especially the peak at elevation -10 m), the amount of movement does not. The movement was doubled from 10 cm to over 22 cm, which is not comparable to the real deflections measured at design section A. Therefore, it was decided to use different parameters for design section C. The principle remained the same (dividing the underlying silt into layers), but the percentage of strength and stiffness reduction decreased. This assumption could also be verified by the fact that design section C had a far greater distance from the pile driving activity than the other design sections and was consequently subjected to fewer disturbances. Good results were achieved by using the following setup:

Figure 4-33: Reduced soil strength after pile driving

Table 4-16: Optimized parameter for pile driving of design section C

Elevation (m)	E_{50}^{ref}	E_{oed}^{ref}	E_{ur}^{ref}	ϕ	m
-4.6 to -6.5	500	576	1000	14	1
-6.5 to -9.0	500	576	1000	14	1
-9.0 to -11.0	3000	3000	9000	26	0.5
-11.0 to -13.0	15000	15000	45000	26	0.5
-13.0 to -15.0	9000	9000	27000	26	0.5

stiffness moduli in kN/m_2

friction angle in $^\circ$

Figure 4-34: Deflection curve for optimized soil parameter

It can be seen in Figure 4-34 that the optimized soil setup led to a comparable deflection as measured in design section A. Even the magnitude of deflection was more or less equal.

4.3.4 SUMMARY OF PILE DRIVING SIMULATION

The former sections presented an attempt to simulate the effect of pile driving in Rhode Island silts on the movement of the sheet pile walls. The assumptions made in section 4.1.5 regarding reductions in strength and stiffness lead to reasonable agreement between simulated and measured wall movements due to the pile driving. Actual wall movements were only measured at design section A because of the failure of the inclinometers at sections B and C shortly after driving commenced.

For design section A, the difference between the inclinometer location and the beam-optimized curve suggests that the inclinometer measurements in the field represented mainly the soil behind the sheet pile wall and did not indicate the real deflection of the sheet pile wall itself. Therefore it was decided to optimize the soil parameters of design sections B and C only for this inclinometer location. Since design section B caused some problems in the excavation simulation (extreme soil strength reduction etc.) the goal there was to get a reasonable qualitative curve without necessarily simulating the right amount of deflection due to pile driving. Consequently, the results there are somewhat questionable. In contrast, for design section C a reasonable deflection curve (both in shape and magnitude) was obtained.

4.4 DISCUSSION ABOUT FINITE ELEMENT SIMULATION

Although the wall deformation patterns could be simulated by reducing the strength and stiffness of the soils during excavation, the magnitude of the reductions in some cases are not reasonable. The following discussion is divided into three parts, a

discussion about the finite element software itself, the excavation simulation and the pile driving simulation, respectively.

4.4.1 FINITE ELEMENT SOFTWARE PLAXIS

There are a few shortcomings when using the finite element software PLAXIS. The first problem was that PLAXIS did not provide the possibility to do a displacement-controlled loading like presented in section 2.2.2. Since measured displacement curves were available, the displacement-controlled simulation would probably have resulted in more realistic soil parameters. However, since this was not possible a trial and error method was used to determine the optimized soil parameters, and some combinations of parameters are not realistic.

A second problem was the choice of constitutive soil model. Although the Hardening-Soil model is a more sophisticated soil model compared to the basic Mohr-Coulomb model, it was not sure if it described the soil behavior in the right way. There was the chance, that a different model like the Soft-Soil-Creep model provided better results. This advanced model can also simulate effects like relaxation (creep) and secondary compression. Because this model is quite sophisticated, more soil input parameters are needed for reliable simulations. Nonetheless, additional parameters like a modified swelling index, modified compression index or modified creep index were not available. For this reason it was decided to use a simple MC-model for the fill and sand layer and a HS-model for the silt layer to keep the complexity (and the number of input parameters) low.

The third problem was that PLAXIS did not allow for allocating excess pore pressures for certain areas in the finite element model. The immediate effect of pile driving - generating excess pore pressures due to vibrations - could not be simulated in this way. In PLAXIS the Pore pressures σ_w are calculated with

$$\sigma_w = p_{steady} + p_{excess} \quad (4.6)$$

where the steady state pore pressures are considered to be input data, generated by groundwater flow calculations. Excess pore pressures are calculated during plastic calculations and are not input data. The effect of disturbance due to pile driving could only be simulated by decreasing the effective stresses (e.g. reduce the soil strength and stiffness). Consequently, simulated deflections in this study did not represent the real soil behavior in the field, but an approximation based on the assumptions made in section 4.1.5.

4.4.2 EXCAVATION SIMULATION

As shown in section 4.2 the first excavation step for each design section could be simulated quite well. No strength and stiffness reduction for the silt was needed to get a similar deflection curve like measured at the site. Only the fill layer was excavated into at this time, therefore the influence on the silt layer had to be negligible small. In contrast to this, there was a large strength and stiffness reduction necessary for the simulation of silt layer in excavation stage 2 and especially for the 3rd and final stage.

The question arose if the measured deflections were unusually high compared to common excavation sites. In general, deflections of excavation walls are influenced

by soil and groundwater conditions, changes in groundwater level, depth and shape of excavations, type and stiffness of the wall and its supports, methods of construction of the wall and adjacent facilities, surcharge loads (Ergun, 2008). Long (2001) and Clough et al. (1990) developed a database for instrumented walls and categorized mainly according to type of soil and type of supporting system. To compare the results they normalized the maximum lateral wall movements by the total excavation height.

Table 4-17: Common lateral wall movements due to excavations

Type of soil	Long (2001)		Clough and O'Rourke (1990)	
	Maximum lateral wall movements normalized by excavation height $\delta_{\max}/H$ (%)		Maximum lateral wall movements normalized by excavation height $\delta_{\max}/H$ (%)	
	Strut support	Anchor support	not incorporated	
Stiff soils, high factor of safety of base heave	0.13	0.14	0.2	0.15
Soft soils, high factor of safety of base heave, stiff soil at dredge level	0.21	0.21	<0.5	n.a.
Soft soils, high factor of safety of base heave, soft soil at dredge level	0.84	0.91		
Soft soils, low factor of safety of base heave	> 3.2		>2.0	n.a.

The normalized lateral deflections σ/H measured at construction site C8 were calculated and are presented in Table 4-18 for every excavation step (1st stage to final stage).

Table 4-18: Normalized lateral movements

	Parameter	Design Section A	Design Section B	Design Section C
	Total Excavation Depth H (m)	8.60	9.10	8.60
1. Stage	Excavation Depth h (m)	2.50	2.50	2.50
	Max. Displacement σ (cm)	0.66	2.36	1.42
	σ/H (%)	0.08	0.27	0.17
2. Stage	Excavation Depth h (m)	6.20	6.20	6.20
	Max. Displacement σ (cm)	2.44	4.37	3.66
	σ/H (%)	0.28	0.51	0.43
3. Stage	Excavation Depth h (m)	8.60	7.70	8.60
	Max. Displacement σ (cm)	8.01	18.90	10.74
	σ/H (%)	0.93	2.20	1.25
Final Stage	Excavation Depth h (m)		9.10	
	Max. Displacement σ (cm)	/	20.23	/
	σ/H (%)		2.36	

The normalized deflections of the first excavation stage indicate stiff soils whereas with further progress of excavating the soil “classification” changes to soft soils. This indicates that the soil at the existing construction site is softening due to excavation. It has to be assumed that the excavation process itself caused significant soil disturbance (almost liquefaction).

In comparison, normalized deflections for design section A and C are almost the same for all excavation stages, while those for design section B increased unproportional in the 3rd stage and final stage. This can also be visualized in Figure 4-35.

Figure 4-35: Normalized lateral displacement (measured) vs. excavation depth

According to Bradshaw et al. (2007) there is one factor that could have played a role in the excessive wall movements observed at the site. The sump pumps that were used to dewater the excavation eroded the silt from beneath the slab (concrete mud) about 30 cm. Russell (2011) confirmed that unusual amounts of silt sediment were found in the tanks used to collect sediment from the pump effluent. This might have caused a reduced vertical overburden stress on the underlying soils. It is the same effect like overexcavation. Additionally, this gap could have provided a space for the surrounding soil to deform into. Both effects would cause less passive resistance and higher displacements. Evidence of additional cracks observed at the top of the west side embankment also supports the idea that the larger wall deformations actually occurred.

Those observations could describe the unusual high deflections after the 3. stage (section A and C) and final excavation stage (section B). Simulating

overexcavation in PLAXIS (additional excavation depth of 0.30 m) without the final decrease in silt strength and stiffness (using the reduction of the 2nd stage for the final excavation simulation) would have increased the final deflections about 0.5 cm only. Therefore, including overexcavation in the finite element model did not lead to satisfactory results. In contrast, reducing the silt strength and stiffness in the finite element simulation led to similar deflections curves like the measured ones and has to be treated as the solution of the problem in this study. Nevertheless, in the authors opinion the problem of overexcavation is not negligible and presents an issue that has to be dealt with in future research.

Another explanation for the unusual high measured deflections is the soil surrounding the inclinometer tubes behind the wall moved or became disturbed and the measured movements are not representative of the actual wall movements. The fact that inclinometers 5 and 10 became unreadable during the later pile driving shows the sensibility of those measuring devices.

4.4.3 PILE DRIVING SIMULATION

Reasonable wall deflection curves were generated to simulate the effect of pile driving by reducing the strength and stiffness of the underlying silts. This suggests that the assumptions made in section 4.1.5 may be acceptable. Pile-driving activities caused pore pressure generations by cyclical loading of the surrounding soil. This can lead to a temporary reduction in effective stress and consequently to a decreased soil strength and stiffness. Similar behavior under cyclic loading of non-plastic silts was also reported by Boulanger and Idriss (2004) and Baxter et al. (2008).

For the simulation process it was assumed that the strength reduction remains constant during the whole simulation step. Since there was no time dependency in the “staged construction” simulation, pore pressure dissipation was not included in the model (this ignores the fact that the actual process is at least partially undrained). However, it was assumed that deflections calculated with this method are the same like a model that would include pore pressure dissipation.

It is still not clear, however, whether the magnitude of strength reduction necessary to match observed deflections are reasonable or exaggerated.

Kraft et al. (1981) proposed a way to include the effect of soil disturbance into the concept of pile load transfer curves (t-z curves). The idea is to calculate an average shear modulus at the pile surface that is smaller than the shear modulus of the undisturbed soil. Based on the assumption that the shear modulus is proportional to the undrained shear strength, the modulus is considered to increase linearly with radial distance from the pile until the undisturbed modulus is reached (Figure 4-36).

Figure 4-36: Idealized radial distribution of soil modulus ratio (Kraft et al., 1981)

It can be seen that the shear modulus is reduced to 0.20 % of the initial value close to the pile. Based on this, soil strength reductions to 10% in the finite element simulation therefore can be regarded as acceptable results. However, Kraft et al. (1981) intention was to describe the soil-pile interaction for pile bearing capacity analyses. The surrounding soil was not a real issue of their paper, but it is a good first explanation of the problem encountered in this study.

Taylor (2011) developed a method to assess the liquefaction potential and hazard due to pile driving. He found out that the main governing parameters for liquefaction potential were in-situ silt density and shear-wave velocity. Furthermore, the overconsolidation ratio (OCR) of the silt was important, with the hazard decreasing with increasing OCR. Also the sequence of pile driving played a significant role in liquefaction potential. Unfortunately, the model used in this study did not incorporate the parameters that Taylor (2011) found out to be important. Therefore, it is suggested for future research to develop finite element models that also include parameters mentioned above and not only soil strength and stiffness.

5. SUMMARY AND CONCLUSIONS

The objective of this study was to perform a finite element analysis of a case study involving significant sheetpile wall movements from an excavation and pile driving activities in Rhode Island silts. The case study was the installation of a pile-supported gate and screening structure as part of a combined sewer overflow project for the Narragansett Bay Commission in Providence. As part of the installation, sheetpiles were driven around the site and excavation occurred in four stages prior to pile driving. Inclinoimeters were installed at three locations, and three design sections A, B and C were modeled. A commercial finite element package, PLAXIS (2-D, version 7.0) was used for the analyses.

First, a literature review of possible results and shortcomings of finite element simulations was presented. The main findings of this review were:

- More complex constitutive soil models like the Hardening-soil model of PLAXIS lead to more reliable finite element results compared to simple models like the linear Mohr-Coulomb model.
- Soil disturbance due to sheet pile wall installation cannot be incorporated well in FE-simulations.
- Boundary conditions are especially important for flow calculations.
- Incorporating building sequences as single simulation phases can enhance the finite element results.
- Stiffness effects of corners of excavations can occur and can lead to different results between 2-D (plane strain) and 3-D simulations.

In Chapter 3, the case study was described in detail, including the geotechnical site conditions, construction sequence, geotechnical instrumentation, and measured wall deflections. It was shown that the deformation patterns of the sheetpile walls were consistent with engineering practice, but the magnitude of the deflection was considered to be unusually high at the later excavation stages. Additional horizontal movements were measured during pile driving activities, accompanied by increased pore pressures in the underlying silts.

Chapter 4 presents a description of the finite element model used to simulate soil behavior during excavation and pile driving activities. The model incorporated three soil layers representing fill, sand and silt layer, respectively. The fill and sand layer were simulated by means of a Mohr-Coulomb soil constitutive model, whereas the advanced Hardening-Soil model (a non-linear hyperbolic model) was used for the silt layer.

In-situ deflection measurements were used to optimize soil parameters of the finite element model. Parameters that were changed to adjust the deflection curves were stiffness parameters E , E_{oed}^{ref} , E_{50}^{ref} and E_{ur}^{ref} and the strength parameter ϕ' . Since it was not sure if the in-situ inclinometer location represented the true deflection of the sheet pile wall or the surrounding soils, two sets of optimizations – for the wall and for the real inclinometer location (1m away from the sheet pile wall), respectively – were executed.

In summary, the first two stages of excavation and wall displacement were modeled well with reasonable values of strength and stiffness. These parameters would

be a good place to start in future modeling efforts involving the Rhode Island silts. This is probably the most important conclusion in going forward with future work.

The only way to simulate the last stages of excavation and displacement was to use unreasonably low values of strength (e.g. $\phi'=14$ degrees) and stiffness. Possible explanations for this poor agreement include:

- The loss of ground during pumping reduced the stability in the excavation and led to larger movements.
- The excavation process itself caused significant disturbance (almost liquefaction) to the soil at the base of the excavation.
- The soil surrounding the inclinometer tubes behind the wall moved or became disturbed and the measured movements are not representative of the actual wall movements.

The effect of pile driving on the wall movements was simulated by reducing the drained strength and stiffness significantly. Although this ignores the fact that the actual process is at least partially undrained, the approach used in this thesis is a first step in understanding movement of adjacent structures in Rhode Island silts due to pile driving.

APPENDIX

Excavation Simulation: Design Section A

Figure A-1: 1. Stage - Soil layer (top) and deformed mesh (bottom) (inclinometer optimized)

Figure A-2: 2. Stage - Soil layer (top) and deformed mesh (bottom) (inclinometer optimized)

Figure A-3: 3. + Final stage - Soil layer (top) and deformed mesh (bottom) (inclinometer optimized)

Figure A-4: 1. Stage - Soil layer (top) and deformed mesh (bottom) (beam optimized)

Figure A-5: 2. Stage - Soil layer (top) and deformed mesh (bottom) (beam optimized)

Figure A-6: 3. + Final stage - Soil layer (top) and deformed mesh (bottom) (beam optimized)

Excavation Simulation: Design Section B

Figure A-7: 1. Stage - Soil layer (top) and deformed mesh (bottom) (inclinometer and beam optimized)

Figure A-8: 2. Stage - Soil layer (top) and deformed mesh (bottom) (inclinometer and beam optimized)

Figure A-9: 3. Stage - Soil layer (top) and deformed mesh (bottom) (inclinometer and beam optimized)

Figure A-10: Final stage - Soil layer (top) and deformed mesh (bottom) (inclinometer and beam optimized)

Excavation Simulation: Design Section C

Figure A-11: 1. Stage - Soil layer (top) and deformed mesh (bottom) (inclinometer optimized)

Figure A-12: 2. Stage - Soil layer (top) and deformed mesh (bottom) (inclinometer optimized)

Figure A-13: 3. + Final stage - Soil layer (top) and deformed mesh (bottom) (inclinometer optimized)

Figure A-14: 1. Stage - Soil layer (top) and deformed mesh (bottom) (beam optimized)

Figure A-15: 2. Stage - Soil layer (top) and deformed mesh (bottom) (beam optimized)

Figure A-16: 3. + Final stage - Soil layer (top) and deformed mesh (bottom) (beam optimized)

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