Numerical Simulation of Pile Installation and Following Setup Considering Soil Consolidation and Thixotropy

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NUMERICAL SIMULATION OF PILE INSTALLATION AND FOLLOWING SETUP CONSIDERING SOIL CONSOLIDATION AND THIXOTROPY

A Dissertation

Submitted to the Graduate Faculty of the Louisiana State University and Agricultural and Mechanical College in partial fulfillment of the requirements for the degree of Doctor of Philosophy in

Department of Civil and Environmental Engineering

by

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May 2016
To my wife Raheleh, for her unwavering love and patience.

and

My father for honoring his endless smile, which is supporting me even without his physical presence.
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ABSTRACT

During pile installation, the stresses and void ratios in the surrounding soils change significantly, creating large displacements, soil disturbance and the development of excess porewater pressures. The disturbed soils, especially fine grain soils, tend to regain their strength over time due to both the consolidation of the excess porewater pressure and thixotropic behavior of soil particles. In this research, the pile installation process and the subsequent consolidation, the thixotropy and load tests for several test piles were modeled using finite element (FE) model. A new elastoplastic constitutive model, which was developed based on the disturbed state concept (DSC) and critical state (CS) theory, was implemented to describe the clayey soil behavior. The developed model is referred as critical state and disturbed state concept (CSDSC). Pile installation was modeled by applying prescribed radial and vertical displacements on the nodes at the soil-pile interface (volumetric cavity explanation), followed by vertical deformation to activate the soil-pile interface friction. The soil thixotropic effect was incorporated in the proposed model by applying a time-dependent reduction parameter, which affects both the interface friction and the soil shear strength parameter. The results obtained from the FE numerical simulation included the development of excess porewater pressure during pile installation and its dissipation with time, the increase in effective normal stress at the pile-soil interface, and the setup attributed to both soil consolidation and thixotropy effects at different times after end of driving. The FE simulation results using the developed model were compared with the measured values obtained from the full-scale instrumented pile load tests to verify the proposed FE model. The results obtained from verification indicated that simulating soil response using the proposed CSDSC elastoplastic constitutive model and incorporating soil thixotropic behavior in the FE model can accurately predict the pile shaft resistance. A parametric study was then conducted by varying the main soil properties, which have significant contribution in setup phenomenon. The obtained data were
analyzed using existing statistical techniques and applying non-linear regression analysis. Several nonlinear regression models were developed under different sets of variables, and finally three sets of regression model were proposed to correlate the soil setup behavior to the contributing soil properties.
1 INTRODUCTION

1.1 Background

Pile setup is defined as increase over time in pile capacity after installation of driven piles. The axial capacity of driven piles in clays has been observed to increase over time after end of driving (EOD). When a displacement pile is driven into the soil, it displaces a soil volume equal to the volume of the pile. Thus, very high normal and shear forces are applied on the surrounding soil layer, causing increases of porewater pressure and changes in the stress state (Basu et al., 2014). Pile setup can reach as much as 12 times the pile capacity at the end of driving (Titi and Wathugala, 1999). Since the pile setup can be significant, a reliable pile design that accounts for pile setup, especially for those installed in fine-grain saturated soils, may reduce project cost while providing required performance criteria (Titi and Wathugala, 1999).

Besides empirical and analytical techniques, numerical simulation has been used to study pile setup (e.g., Wathugala and Desai, 1991; Elias, 2008; Basu et al., 2014). During pile installation, soils adjacent to a pile are significantly disturbed and remolded generating excess porewater pressures in saturated soils. Therefore, it is necessary to consider a valid numerical simulation technique to study pile installation and the following setup phenomenon appropriately. An appropriate constitutive model is required to describe the soil behavior in the vicinity of driven piles. Researchers mostly have used conventional soil models like modified Cam Clay model or Drucker-Prager model for soil (e.g. Sheng et al., 2005; Dijiksta et al., 2008; and Fakharian et al., 2013). Besides the conventional soil models, some researchers attempted to develop advanced model, which capture more realistic soil response during shear (e.g., Shao, 1998; and Basu et al., 2014). Historical studies about numerical modeling of pile installation and setup is presented in next section.
1.1.1 Numerical Simulation of Pile Installation

During pile driving, the stresses and void ratios in the surrounding soil are continuously changed. The development of large deformations and porewater pressure followed by the soil disturbance and remolding are common in the vicinity of pile-soil interface. Modeling pile penetration using the finite element (FE) method usually encounters nonlinearities caused by large deformation, frictional contact and elastoplasticity (sheng et al., 2005). Many research studies, which have been performed on pile installation, attempted to overcome uncertainties in geotechnical engineering that, are related to the variation of stresses and strains during pile installation. The factors that have been considered by researchers (e.g., Desai, 1978; Carter et al., 1979; Baligh, 1985; Wathugala, 1990; Sheng et al., 2005 and 2009; Mabsout et al., 1994, 1995, 2003, and Qiu et al., 2011) are:

- The method of pile movement in FE analysis
- Soil behavior at the vicinity of the pile
- Large differences between pile and soil stiffness
- Problems due to geometry of pile at the corners
- Friction between pile surface and soil
- Mesh distortion problem due to abovementioned items

1.1.2 Numerical Simulation of Pile Setup

The pile setup is generally caused by the increase in the effective stresses at the pile-soil interface after EOD, which is related to dissipation of excess porewater pressure, as well as the thixotropic effects that cause increase in shear strength over time (Budge, 2009). The pile setup
of piles driven in saturated soil has three main components: 1) setup due to consolidation of the induced excess porewater pressure, 2) setup due to soil thixotropic behavior and 3) aging. The excess porewater pressure, which is developed during the pile installation, dissipates with time and causes increase in the effective stresses. In the numerical simulation, the theory of consolidation is used to model dissipation of the excess porewater pressure with time.

Thixotropy is defined as the increase in soil strength over time after remolding under constant water content (Fakharian et al., 2013). Regardless of dissipation of the excess porewater pressure, the remolded soil particles tend to refabricate / rearrange, causing densification and increase in shear strength. Thixotropy is more common in clay soil with flocculated particle structure (Mitchel, 1960). Literature discussing the contribution of thixotropy in pile setup phenomena is rare. Most previous researchers explain thixotropy as a soil aging mechanism after completion of consolidation. However, in reality, thixotropic behavior starts immediately after remolding of soil (i.e., immediately after EOD in a driven pile case), which indicates that part of the soil strength regained during consolidation can result from thixotropy. In addition, thixotropy is common in fine grain soils, while aging is a well-known phenomenon for coarse grain soils. Therefore, investigating thixotropic effects in pile setup is necessary before dissipation of the excess porewater pressure, as well as after that time.

As mentioned earlier, the aging phenomenon has been associated with an increase in pile capacity, which occurs after dissipation of the excess porewater pressure (e.g., Bullock et. al., 2005). This phenomenon is more common in sandy soil due to secondary compression and change in soil fabric under constant effective stress (creep compaction).
1.1.3 Soil Constitutive Model

Constitutive models are used to describe the stress-strain behavior of soils by providing a framework for understanding how soil will behave under different loading conditions. Constitutive models are implemented in FE technique to define material behavior. Wood (1990) suggests that considering the past history and future behavior of the soil and identifying an appropriate level of complexity is necessary in selecting a constitutive model. The overconsolidation ratio (OCR) exhibits the history of a soil. In comparison with the normally consolidated (NC) soils, the overconsolidated (OC) soils have lower void ratio, higher strength, and show the strain softening response (Yao et al., 2007). Wood (1990) suggested that for engineering purposes, a relatively simple model like the Modified Cam Clay (MCC) model can be modified, and levels of complexity added as necessary in order to provide insight into particular problems. Wroth and Houlsby (1985) suggest that, for a constitutive model to be useful in solving engineering problems, it should be simple and reflect the physical behavior of the soil. Duncan (1994) states that for the constitutive models to be practical, they should utilize a low number of model parameters, which can be easily obtained from conventional soil tests.

1.2 Objectives and Scope of the Study

1.2.1 Objectives of the Study

The main objective of this study is to investigate pile setup in saturated cohesive soils using numerical simulation techniques. This objective includes the following main parts:

1. Identification of an appropriate technique to penetrate pile into the saturated subsurface soil, which covers the real pile installation effects, such as excess porewater pressure generation, displacement in soil adjacent to the pile, and variation in the stress state due to pile installation.
2. Studying the pile setup in saturated cohesive soils, to assess increase in pile capacity over time after end of driving (EOD), using an appropriate numerical simulation technique.

3. Developing an elastoplastic constitutive model for saturated cohesive soils, which can capture the soil behavior adjacent to driven piles during pile loading.

4. Simulating pile setup case studies of Bayou Lacassine Bridge site, Sabin river case study, Bayou Zouri site, Bayou Bouef site and Baton Rouge Cajun site for verification of the model.

5. Performing a parametric study to identify and evaluate the most effective parameters in pile setup.

6. Developing an analytic regression model to evaluate pile setup in clayey soils.

1.2.2 Scope of the Study

To accomplish the objectives of the study, the following analysis was conducted for each part:

1. The first objective was achieved through FE analysis by first applying series of prescribed displacement in the soil’s axisymmetric boundary to create a displaced volume in the soil equal to the size of the pile for pile placement (volumetric cavity expansion). The pile was then placed inside the cavity, and the interaction between pile and soil surfaces was activated along with applying vertical penetration until the steady state condition is reached.

2. The second objective was achieved through numerical study of the setup for the following components:

   a. Consolidation setup: based on the consolidation theory, numerical study of radial consolidation around the pile shaft was performed and the increase in the effective stresses
was evaluated. The increase in effective stresses will result in an increase in pile capacity, especially at the pile shaft.

b. Thixotropic setup: in this study, an exponential evolution function was introduced to define the soil strength regained with time after remolding the soil. This function was implemented during the numerical study and the amount of setup caused by thixotropic behavior was obtained.

3. The third objective was achieved through application of the disturbed state concept (DSC) on the soil behavior during shear loading. The critical state modified Cam Clay (MCC) model was implemented into the DSC as a reference state. Based on this combination, a new elastoplastic constitutive model was developed, which is able to define both NC and OC clay behavior.

4. The parametric study was performed by running several models with different soil properties to evaluate these properties as well as other parameters contributing to soil setup behavior.

5. In order to develop analytical model to evaluate pile setup in the clayey soils, the nonlinear multivariable regression analysis was conducted on data obtained from the parametric study.
2 LITERATURE REVIEW

2.1 Numerical Simulation of Pile Installation

During pile driving, the stresses and void ratios in the surrounding soil are continuously changed. The development of large deformations, large strains and porewater pressure followed by soil disturbance and remolding are common, especially at the vicinity of pile-soil interface. Modeling pile penetration using the FE method usually encounters with nonlinearities caused by large deformations, frictional contact and elastoplasticity (Sheng et al., 2005). Therefore, the numerical modelling of a pile installation process using the FE method is not an easy task. Many research studies of pile installation have attempted to overcome uncertainties in geotechnical engineering that are related to the stress and strain variations during pile installation.

2.1.1 Methods of Installation

Most previous studies assumed that the model pile is located in a pre-bored hole to final depth, and an additional small penetration is then applied to simulate the static load test performed in the field (e.g., Trochanis et al., 1991; Mabsout and Tassoulas, 1994; Mabsout and Sadek, 2003; and Dijkstra et al., 2008). However, pre-bored modeling cannot capture the generated excess porewater pressure in the soil body during pile installation. Therefore, other approaches, such as the strain path method or cavity expansion theory, coupled with FE analysis have been adopted by some researchers (e.g., Wathugala, 1990; Shao, 1998; Titi and Wathugala, 1999) to estimate the generated excess porewater pressure during pile driving.

There are some studies reported in the literature that attempted to simulate pile driving from ground surface to the desired depth using the FE method (e.g., Sheng et al., 2005, Hugel et al., 2008, Sheng et al., 2009; Dijkstra et al., 2010, Dijkstra et al., 2011; and Sheng et al., 2013). The majority of these studies have not used the coupled pore pressure analysis in simulating pile
installation. However, pile driving in saturated cohesive soil is usually associated with large displacements and development of excess porewater pressure followed by the dissipation of induced excess porewater pressure, rearrangement of soil particles (thixotropic behavior), and longtime aging that results in an increase in pile resistance. An appropriate numerical simulation should be able to capture all these components. There are few studies in the literature that focus on the numerical simulation of the entire pile driving process, including changes in the soil shear strength and the porewater pressure distribution after EOD (e.g., Elais, 2008; Fakharian et al., 2013). The available information in the literature regarding pile installation and the following setup are addressed in the next section.

2.1.1.1 Cavity Expansion Method

The cavity expansion theory is based on the theoretical analysis of the stresses, strains and porewater pressure produced by a cylindrical or spherical cavity in soils. During pile installation, soil displacement is a combination of the expansion of a spherical and cylindrical cavity, plus a small further vertical displacement in soil occurs when the pile tip pass that level (Randolph and Wroth, 1979). In cavity expansion, pile driving is modeled based on the cylindrical cavity of the pile shaft under undrained condition. In cylindrical cavity expansion, which is shown in Figure 2.1 the radial displacement $u_r$ is the only component for the displacement vector ($u = u_r$). In this figure, the strain vector can be defined as:

$$\begin{align*}
\{\varepsilon_r\} &= \left\{ \frac{du}{dr}, \frac{u}{r}, 0 \right\} \\
\varepsilon_\theta &= \frac{\partial u}{\partial r} \\
\varepsilon_z &= 0
\end{align*}$$

(2.1)

where $\varepsilon_r$, $\varepsilon_\theta$ and $\varepsilon_z$ are the radial, circumferences and vertical strain components, respectively.
where, $r$ is the current radial coordinate, which is changed from initial radius, $r_i$ to the final radius after expansion, $r_f$. For elastic material under cylindrical cavity expansion, we can write the following relation for active strain components:

$$
\varepsilon_r = \frac{d}{dr} (r \varepsilon_\theta) 
$$

(2.2)

$$
\varepsilon_r = \frac{1-v^2}{E} \left( \sigma_r - \frac{v}{1-v} \sigma_\theta \right) 
$$

(2.3)

$$
\varepsilon_\theta = \frac{1-v^2}{E} \left( -\frac{v}{1-v} \sigma_r + \sigma_\theta \right) 
$$

(2.4)

where $\sigma_r$ and $\sigma_\theta$ are radial and circumferences stress components, respectively. The equilibrium equation for a cylindrical cavity problem can be expressed in terms of $\sigma_r$ and $\sigma_\theta$ as follows:

$$
\frac{d\sigma_r}{dr} + \frac{(\sigma_r-\sigma_\theta)}{r} = 0 
$$

(2.5)
Solving the simple differential equation (2.5) and applying appropriate boundary conditions yield the following solution for stress components:

\[
\sigma_r = \frac{1}{r^3(r_f^2 - r_i^2)} \left[ p_0 r_f^3 (r^3 - r_i^3) + pr_i^3 (r_f^3 - r^3) \right] \quad (2.6)
\]

\[
\sigma_\theta = \frac{1}{2r^3(r_f^2 - r_i^2)} \left[ p_0 r_f^3 (2r^3 + r_i^3) - pr_i^3 (r_f^3 + 2r^3) \right] \quad (2.7)
\]

where \( p_0 \) and \( p \) are the applied pressure at \( r = r_f \) and \( r = r_i \), respectively.

Randolph (1979) proposed the following relation to predict porewater pressure change along shaft as a function of the initial and final mean effective stresses, the soil elastic shear modulus, and the un-drained shear strength:

\[
\begin{align*}
\n u^{\text{excess}} &= (p'_i - p'_f) + 2c_u \ln \left( \frac{R}{r} \right)
\end{align*}
\quad (2.8)
\]

The parameter \( R \) represents the radius of plastic and elastic boundary, and its value is:

\[
R = r_0 \sqrt{\frac{G}{c_u}} \quad (2.9)
\]

Where, \( r_0 \) is the pile radius, and \( r \) is distance from pile center along with shaft. The cavity expansion theory has been widely applied in soil mechanics problems such as in-situ soil testing, deep foundations, tunnels and underground excavation in soil and rock (Yu, 2000).

The undrained cylindrical cavity expansion theory was developed by Carter et al. (1979). Randolph et al. (1979) used this theory to model installation of displacement piles in clayey soil, using a prebored model pile, and then variation of radial effective stress and porewater pressure during and after pile installation were modeled using the cavity expansion theory. When a solid pile is driven into the soil, it must initially displace a volume of soil equal to the pile size. At
shallow penetrations, there will be some heave in the ground surface, but in deeper penetrations, there is only outward displacement in radial direction (Randolph et al., 1979). Definition of stress-strain relation in cavity expansion needs an appropriate constitutive model. Therefore, cavity expansion theory has been developed based on an elastoplastic constitutive model. For small deformation problems, cavity expansion theory is able to obtain a close form solution for the pressure and displacement relationship, but for large deformation and in the case of saturated two-phase materials as if the soils it is required the use of numerical techniques (Carter et al., 1979). For large deformations, the cavity pressure approaches a limiting value that can be determined explicitly and independently from the numerical solution (Carter et al., 1986). However, pile installation in a soil with perfectly plastic behavior can lead a closed form solution for excess porewater pressure in soil, but this can only capture porewater pressure due to cavity expansion, not the porewater pressure value, which was produced by shear at the pile-soil interface during pile installation (Randolph et al., 1979). The conventional consolidation theory is used to model porewater pressure dissipation; therefore, variation of the excess porewater pressure, stresses and strains might be determined during cavity expansion as well as during the subsequent consolidation phase.

Desai (1978) evaluated the effects of piles driven into saturated soil by using numerical simulation, and the effect of changes in stresses and porewater pressure during the consolidation with the FE procedure. The cavity expansion theory was used to obtain the stresses and porewater pressure values during installation, and these values were considered as the initial conditions in the FE analysis. His results were included the stresses and the excess porewater pressure variations during cavity expansion, and dissipation of excess porewater pressure over time based on one dimensional theory of consolidation.
Randolph et al. (1979) studied pile installation effects on the surrounding soil by using the cylindrical cavity expansion and conducted the parametric study by assuming the pile installation problem as a plain strain case. This parametric study was done for stress states obtained in the pile installation phase. The soil consolidation phase also was modeled, so stresses and strength changes during the consolidation stages were evaluated. These studies demonstrated that predictions for displacement in the radial direction based on the cavity expansion method has a very good agreement with field measurements, as shown in Figure 2.2.

![Figure 2.2: Displacement in soil body due to pile installation (from Randolph et al, 1979).](image)

The modified cam clay model was used to capture the soil response, and the induced porewater pressure related to shearing was expressed as a function of soil deviatoric stresses. In general, the following conclusions were obtained from their research:

- The undrained shear behavior at the pile-soil interface due to pile installation “effectively erased the memory of the soil” at the interface; therefore, as one can see in Figure 2.3 the generated normalized excess porewater pressure at interface does not change significantly with
increasing OCR values (Randolph et al, 1979). It should be noted that the effect of the OCR parameter has already been considered in the undrained shear strength values $C_u$. For the overconsolidated soils, there is an area in 6 to 11 pile radius distance along with pile shaft that it can cause crack, and subsequently increase in the soil permeability.

![Figure 2.3: Change in the excess porewater pressure at the pile surface with OCR (from Randolph et al, 1979).](image)

- For soil with higher elastic shear modulus, higher excess porewater pressure is developed; however, the final stress changes around the pile are relatively independent of the value of shear modulus. Typical stress and porewater pressure changes in soil along with pile shaft immediately after pile installation has been shown in Figure 2.4.

- They proposed the following formula to predict the change in porewater pressure along the pile shaft as a function of the initial and final mean effective stresses, the soil elastic shear modulus, and the undrained shear strength:

\[ u^{excess} = (p' - p'_f) + 2c_u \ln(\frac{R}{r}) \]
where, $p_i'$ and $p_f'$ are the initial and final effective stress values, respectively; and $r$ is the radial distance from pile center along the pile shaft. The parameter $R$ represents radius of the plastic to elastic boundaries, and its value is defined as follows:

$$ R = r_0 \sqrt{\frac{G}{c_u}} $$

Where, $r_0$ is the pile radius, $c_u$ is the soil undrained shear strength, and $G$ is soil secant shear modulus.

- Based on measured data obtained from field tests, the porewater pressure dissipation is mostly radial over the pile shaft, and soil particles are displaced mostly in the radial direction during consolidation.
- The results showed that the soil at close distance to the pile shaft (less than $2r_0$) undergoes plastic deformation; therefore using an appropriate elastoplastic constitutive model for soil during the consolidating phase is necessary. On the other hand, during consolidation, the soil close to the pile will yield and experience hardening behavior, and the consolidation
coefficient and compressibility factor are controlled by gradient of virgin compression line in $e - ln p'$ curve. However, the soil at further distance will be unloading in shear, so the compressibility and consolidation coefficient is controlled by the gradient of the swelling line.

- The values of OCR have insignificant effect in the consolidation process, but selecting different value for shear modulus has significant effect on the values of the generated excess porewater pressure and the time required for dissipation.

- As shown in Figure 2.5, the increase in the undrained shear strength from its initial value to the time after the completion of the consolidation and dissipation of the excess porewater pressure is almost constant for soils with different histories.

Figure 2-5: Normalized long-term shear strength of soil at different OCR values (from Randolph et al, 1979).
• The main shortcoming of their research is assuming plane strain problem for pile driving and neglecting existence of shear in the out of plane surface, which has an important role in balancing the stresses during pile driving; therefore, solving problem with an appropriate axisymmetric model may can help to analyze in a more accurate way.

• The soil close to the pile will be remolded during pile installation, and the shear strength in this condition is lower than the shear strength of the undisturbed sample at peak point. This kind of drop in strength is related to the sensitivity of the soils (Randolph et al., 1979).

• There is extra porewater pressure in the pile-soil interface due to large drop in the effective stress during remolding of the sensitive soil structure (Randolph et al, 1979).

• They used an analytical method to evaluate the increase in shear strength of soil over time after EOD. They normalized the predicted pile capacity at each time with respect to the capacity after consolidation. Comparison between the model prediction and the measured values from field tests is shown in Figure 2.6.

![Figure 2.6: Comparison between model prediction and field measurement for shear strength (from Randolph et al, 1979).](image)
2.1.1.2 Strain Path Method

The strain path method (SPM) was developed based on the fact that changes in the geometry of the soil body under penetration by a rigid material is unique regardless of the soil type, and is dependent only on the shape of penetrating rigid material (Wathugala, 1990). Therefore, pile installation is a strain control problem, and is independent of soil behavior. This method is based on an analogy between soil flow and fluid flow around a solid body and simulates pile installation, cone penetration, and undisturbed soil sampling by introducing strain paths based on deformation field. The SPM was first introduced by Baligh, (1985) and used later by other researchers (e.g., Wathugala, 1990; Shao, 1998), provides an analytical solution to calculate strain variation in the un-drained penetration in saturated clays. This method assumes deep penetration in saturated clays to be a fully-constrained process (no volume change); the deformations and strains developed during the penetration of a foreign object are considered to be independent of the shear resistance offered by the soil. Figure 2.7 shows a deformation field in the soil body due to pile installation. Any vertical line in Figure 2.7 represents a streamline for fluid flows around the pile.

Based on this figure, Baligh (1985) derived the following relation for any vertical streamline:

\[
\left( \frac{r_z}{R} \right)^2 = \left( \frac{r_0}{R} \right)^2 + \frac{1}{2} \left( 1 + \cos \varphi \right)
\]  

(2.10)

And

\[
\varphi = \arctan \left( \frac{r_z}{z} \right)
\]

(2.11)

where \( r_z \) is the \( r \) coordinate of the streamline and \( r_0 \) is the \( r_z \) value when \( z \to -\infty \).
Baligh (1985) derived following strain field for any soil point located at \((r,\phi)\) coordinates:

\[
\varepsilon_{rr} = \left(\frac{R}{2r}\right)^2 \left[1 + \cos\phi(1 + \sin^2\phi)\right] \tag{2.12}
\]

\[
\varepsilon_{zz} = \left(\frac{R}{2r}\right)^2 \left[-\cos\phi\sin^2\phi\right] \tag{2.13}
\]

\[
\varepsilon_{\theta\theta} = \left(\frac{R}{2r}\right)^2 \left[-(1 + \cos\phi)\right] \tag{2.14}
\]

\[
\varepsilon_{rz} = \left(\frac{R}{2r}\right)^2 \left[-\sin^3\phi\right] \tag{2.15}
\]

Figure 2.7: Deformation field in saturated clay during pile installation (from Wathugala, 1990).

Figure 2.8 shows graphical form for Equations (2.12) to (2.15) at a typical radial distance \(r\).

These strain equations are used to define the corresponding stress field using an appropriate constitutive model.
Whittle and Setabutr (1998) used the SPM to derive EOD stresses and porewater pressure around a displaced pile, which was installed in saturated clay. A one-dimensional FE model was then used to investigate the radial dissipation of generated excess porewater pressure. Variation of effective stresses and the soil properties were described by the elastoplastic MIT-E3 constitutive model. In the SPM, the strain path for each point in the soil body can be obtained by having the deformation field. The effective stresses then, which are related to the obtained strain path, are calculated using an appropriate constitutive model. The porewater pressure and the total stress are also calculated by applying equilibrium equation in the soil body. Accuracy of the prediction of parameters depends on the degree to which the defined displacement field is identical with actual soil displacement in real pile installations (Wathugala, 1990).

![Strain distribution after pile driving at distance r=2R](image)

**Figure 2-8:** Strain distribution after pile driving at distance r=2R (from Wathugala, 1990).

Wathugala (1990) and Shao (1998) simulated pile load test for 1.72 inch and 3 inch steel pile segments in Sabin clay using the FE method, assuming a pre-bored installation to the its final position. The SPM was used to simulate the pile installation phase, and a four-noded linear element was used for soil body. This type of element was selected because of shear-locking problem experienced during shearing when nonlinear elements were used in saturated soil (Wathugala,
1990 and Shao, 1998). Figure 2.9 presents the FE model for the 3-inch pile segment, which had been penetrated to 16 m depth by the Earth Technology Corporation in 1986. Pile installation, excess porewater pressure dissipation, and the load tests at different time after probe penetration were simulated in separate stages, as shown in Figure 2.10, and the results for each stage were used as initial conditions of the next stage. The hierarchical single surface elastoplastic model (HISS model) was used to describe the soil behavior. The values of stresses and porewater pressure obtained from SPM are unbalanced in the vertical direction because the SPM is solved for variables only in horizontal direction. Wathugala, 1990, concluded that the excess porewater pressure obtained from SPM was underestimated by a factor of 0.30 in comparison with field measurements. The normalized excess porewater pressure after pile driving was calculated and compared with the field result. Figure 2.11 compares the normalized excess porewater pressure with the field measurements for the 3-inch pile segment. (Later in this dissertation, this case study is simulated using the proposed techniques.)

Figure 2.9: Finite element mesh for 3-inch pile segment (from Wathugala, 1990).
Figure 2.10: Different stages for numerical simulation of probe installation, consolidation, and load tests (From Wathugala, 1990).

Figure 2.11: Change in excess porewater pressure over time after EOD (From Wathugala, 1990)

Load test was simulated at different times after pile installation by applying additional vertical displacement. Figures 2.12 and 2.13 show results for typical shear transfer and porewater pressure generated during the pile load test.
2.1.1.3 Finite Element Method

The finite element (FE) techniques have been used directly or indirectly to simulate pile installation. There are two methods in the FE models that can be used to simulate pile movement from ground surface to desired soil depth: the prescribed load or the prescribed displacement.
method. In the first method, pile is penetrated using the applied vertical load and in the second
method, pile is penetrated by applying the prescribed displacement to the pile / soil elements to
push it into the soil body as is shown in Figure 2.14.

Aristonous et al. (1991) used the commercial FE software Abaqus in order to simulate pile
installation with a three dimensional FE model for combined latterly and vertically loaded single
and double piles. Soil domain was 12 pile width along the pile radius and 1.7 pile length in depth
for single pile. The horizontal dimension considered twice in double pile case. They used both
linear elastic and Drucker-Prager model for soil behavior, and the Mohr-Coulomb friction law was
used for the soil-pile interface at contact condition. The pile was assumed pre-bored in the desired
depth, and the prescribed load was applied to the pile in order to simulate the load tests.

![Figure 2-14: Pile penetration with prescribed displacement (from PLAXIS software manual).](image)

Mabsout and Tassoulas (1994) simulated pile installation using a prebored pile in normally
consolidated clay with an axisymmetric mesh, as shown in Figure 2.15. They used rounded pile
tip to facilitate the numerical solution. The soil domain was selected 10 pile diameter in horizontal
direction and 1.5 pile length in vertical direction. The traditional contact Coulomb model was used
for pile-soil interface, and the bounding surface constitutive model proposed by Dafalias and
Hermann (1982) was used to describe the soil behavior. A small hole below the pile tip was
provided to ease penetration of pile in the FE model. The aspect ratio for the soil element (i.e.,
ratio of vertical to horizontal dimensions) at interface was selected to be 3 to 1. They used dynamic
analysis under impact load to simulate hammer force, and special absorbing viscous-type
boundaries were defined at the far domain to transmit the waves produced in soil body during pile
driving. An overburden pressure equivalent to 1-meter soil height over the ground level was
considered for convergence purposes.

Figure 2-15: Finite element discretization of pile driving problem (from Mabsout and Tassoulas,
1994).
Mabsout et al. (1995) repeated their prior study, using saturated clay with different shear strength. They evaluated the effect of driving at various initial levels of preboring, by calculating the soil resistance at each level. The shaft resistance, tip resistance and change in the porewater pressure were calculated. The pile geometry, finite element model and the applied constitutive models were the same as those used by Mabsout and Tassoulas (1994). They concluded that the prebored model could not accurately simulate soil disturbance at the soil-pile interface; therefore, they concluded that an appropriate model for simulating the effect of pile driving under these conditions should be developed (Mabsout et al., 1995). They related the soil resistance during pile installation $\tau_s$ and the coefficient of friction at interface $\mu$ with the undrained shear strength of the soil $c_u$ using the following equation:

$$\tau_s = \alpha \ c_u = \mu \ \sigma_r^t$$  \hspace{1cm} (2.16)

where, $\sigma_r^t$ is total radial stress, and the factor $\alpha$ depends on clayey soil type (soft or stiff clay), and the method of installation of the pile. $\alpha$ varies from 1 and higher for soft clay to 0.50 and lower for stiff clay (Mabsout et al., 1995). For more disturbance at interface, values of $\alpha$ and $\mu$ decrease, so they considered a minimal disturbance at the soil-pile interface; the value of the factor $\alpha$ was chosen to be 0.65; and the value of $\mu$ was assumed to be 0.10 and 0.07 for stiff and soft clay, respectively.

Following their previous studies, Mabsout et al (2003) used the same procedure to study the effect of pile installation in saturated clay by comparing two different finite element models: first, a 17 m prebored pile plus 1 meter extra penetration to reach desired depth was simulated, but the second pile was pebored exactly to 18 m depth. The first case was named “driven” pile and the
latter one was called “prebored” pile. Figure 2.16 shows deformed mesh for the first and second models in left and right sides, respectively.

Figure 2-16: Deformed mesh for driven pile and prebored pile (From Mabsout et al, 2003).

The induced porewater pressure during pile installation in both cases was related to the volume change $d\varepsilon_{mm}$ through the mix bulk modulus of soil-water $\Gamma'$ with the following equation:

$$ du = \Gamma' d\varepsilon_{mm} $$  \hspace{1cm} (2.17)

Comparison between the results of prebored and driven piles showed a considerable difference in pile tip resistance; however, difference in the pile shaft resistance was negligible.

Wehnert and Vermeer (2004) used interface element in FE software Plaxis to model pile load tests for the prebored piles. They simulated a concrete pile, which was prebored in the soil. The soil was modeled using three constitutive models available in Plaxis: Mohr-Coulomb (MC) model, the soft soil (SS) model and the hardening soil (HS) model. An axisymmetric FE mesh was adopted for soil domain, which was large enough to reduce the boundary effects. The FE analysis was
performed with and without thin layer interface elements located at the soil-pile interaction zone. During the first steps, pile was placed at the desired location; the load test was then simulated by applying an additional prescribed displacement at the pile head. Three models with the interface elements and three other models without interface were simulated. They also evaluated mesh size effects by varying mesh size for each model. Figures 2.17 and 2.18 represent the effect of mesh size on the results. These figures indicate that using the thin layer interface element allows use of a coarser mesh. However, if fine mesh size was used an appropriate result can be achieved even without using the interface elements. The obtained result indicates that using interface elements has advantages because numerical simulation results obtained using very coarse elements and those obtained from very fine elements are approximately the same. There is a small difference only for the base resistance because of the in-consistency between the element size and the pile base dimension for coarse mesh (Wehnert and Vermeer, 2004). Figure 2.19 presents the load-settlement results obtained from adopting different constitutive models to simulate soil behavior. This figure indicates that using HS, SS or MC models to simulate soil behavior yields close load-settlement relation for the pile tip resistance; however, the prediction of MC model for pile shaft resistance differs from other models. Figure 2.20 compares the results obtained from numerical modeling with the measured values calculated from the load test data. Figure 2.20 shows that both HS and SS models are able to predict pile shaft resistance, but the numerical simulation prediction of the MC model significantly underestimates the data. In these figures $R_b$, $R_s$ and $R$ represent pile base resistance, shaft resistance, and total resistance respectively. In general, results show that use of three significantly different models it do not significantly affect prediction of base resistance; however results for predicting shaft resistance vary significantly depending on the choice of the constitutive model.
Figure 2.17: Mesh size effect for Mohr-Coulomb model, with interface (left) and without interface (right) [from Wehnert and Vermeer, 2004].

Figure 2.18: Mesh size effect for Hardening Soil (HS) model, with interface (left) and without interface (right) [from Wehnert and Vermeer, 2004].
Figure 2.19: Results for Mohr-Coulomb, Soft-Soil, and Hardening-Soil Models

Figure 2.20: Comparison results for pile load test and numerical simulation for shaft resistance (from Wehnert and Vermeer, 2004).
Sheng et al. (2009) solved penetration problems in dry sandy soil by using the FE method and applying the arbitrary Lagrangian Eulerian (ALE) adaptive meshing technique to solve the governing equations. At the end of an updated Lagrangian solution, the mesh may be distorted since it moves with the interface material. The mesh and material then release each other so that the deformed mesh can move independently to make a new and more appropriate mesh. This step can be formulated by an additional Eulerian technique. Therefore, by a combination of these two main stages of solution, the ALE method could be used to solve the mesh distortion problem during pile installation. The authors solved two sample case studies to evaluate the effectiveness of the ALE method. The first case was an axisymmetric model for cylindrical pile with 0.40 m diameter and 3.0 m length, and with the cone angle 60 degree, which was penetrated into the 2.4×4.8 meter soil domain. The linear triangular elements are used for both pile and soil domain. The pile penetration was applied using prescribed displacement to the pile. The soil mesh size at the interface area was selected at 1, 0.5, 0.25 and 0.125 pile radius. Figure 2.21 presents different FE models.

The authors applied the updated Lagrangian method to the two coarser meshes (meshes A&B), while the ALE method was applied to the three finer meshes (meshes B, C and D). The pile was modeled as an elastic material with modulus ratio equal to 20000 with respect to the modulus of the soil. The FE model results for variation of pile resistance over soil depth are shown in Figure 2.22. By evaluating the results, the oscillation of the diagram was observed in the coarser mesh, dictating that very fine element must be used for more accurate results (Sheng et al., 2009).
In addition, they used Mohr-Coulomb (MC) and modified Cam Clay (MCC) models to simulate different cases. Less dilatant models showed less convergence problem. Figures 2.23 and 2.24 show a model consisting of very fine mesh under MCC model for soil and the comparative results for different types of analyses. Figure 2.24 indicates that the UL analysis may not simulate
full penetration of the pile for such a fine mesh, and oscillation behavior was observed due to the large amount of interfacial friction coefficient (Sheng et al., 2009). It should be noted that the largest value that was possible to be specified for the soil-pile interface friction coefficient was 0.10, which is less than the actual friction coefficient.

Using Abaqus, Hugel et al (2008) simulated pile installation in soil by using the explicit FE model and under vibratory loading. They used both two-dimensional axisymmetric and three-dimensional models to cover homogeneous and non-homogeneous subsoil, respectively. A rigid tube of 0.10 mm radius, with no friction between this tube and surrounding soil, was used as a model along the axis of penetration to ease the penetration process. During penetration, the pile pushed the tube away, and the interaction between pile and the surrounding soil was activated. Mesh distortion problem occurred for interface friction angles greater than $\phi/3$. The mesh problem was solved in the axisymmetric explicit model by adopting ALE technique and refining mesh in the three-dimensional model (Hugel, 2008). Figure 2.25 presents the FE model for simulation of pile driving in subsoil.

Figure 2-23: Very fine mesh for different analysis method with MCC model (from Sheng et al., 2009).
Figure 2-24: Load-displacement curves using MCC model for soil (from Sheng et al., 2009).

Figure 2-25: Finite element simulation of pile installation (from Hugel et al., 2008).
Dijkstra et al. (2008) presented two FE methods for modeling the stress and strain behavior in the soil due to pile installation. In the first method, the small-strain FE method was used to simulate pile installation. An axisymmetric model in Plaxis software was used, and the pile was placed 30 pile diameters away from boundaries to reduce boundary effects. They indicated that “re-meshing technique can partially help to overcome mesh distortion problem, but frequent remapping of the solution variables from distorted mesh into the new mesh is an additional source of numerical error”. The pile was already prebored to the final depth at the beginning of analysis. Interface elements were placed between pile shaft and the soil, and these elements were inactive during the pile expansion phase. The pile installation process was simulated directly after the initial step by expanding the soil by prescribed displacements at the soil-pile interface. The interface elements were activated after this phase. In the second method, large deformation numerical analysis was used for simulation of the installation phase. In the Eulerian scheme the mesh and the material flows are decoupled, resulting in material flows through the mesh. The FE analysis was done with an axisymmetric mesh with triangular elements for the soil and pile (Dijkstra et al., 2008). Figure 2.26 shows mesh and geometry of boundary conditions for the second case. The pile was initially embedded for the first eight meters. The installation phase was then executed using the prescribed vertical displacements at the penetration velocity of 3.5 cm/sec, which corresponds to the scaled pile penetration velocity during the centrifuge test (Dijkstra et al., 2008). A total extra penetration equal to 7 meters was applied. The Euler backward time stepping technique was used to achieve numerical stability. Figure 2.27 shows the results for base resistance obtained with the Eulerian method compared with the values measured from the centrifuge test. As shown in Figure 2.27, there is a good correlation between the results at the beginning and end of penetration; however, there is no correlation between the results during pile installation.
Following their previous research, Dijkstra et al. (2010 and 2011) introduced two Eulerian schemes to simulate pile installation in sandy soil. In their first approach, they fixed the pile and let the soil flow around it, while in the second approach, they kept the soil fixed and the pile moved.
into the soil body. The Mohr-Coulomb constitutive model was selected for soil behavior. An axisymmetric FE model was produced with soil domain, which was 20 pile diameter in the horizontal direction and 2 pile length in vertical direction. Figure 2.28 shows the results for pile resistance in the two approaches, and that the prediction of the first and second methods exceeded experimental results by 55% and 20% respectively.

In order to evaluate the installation effect on surrounding soil, Pham (2010), applied prescribed displacement to a prebored pile, which was already located inside the soil. The pile was considered prebored because it was impossible to penetrate the pile from the ground surface to the desired depth in PLAXIS software (Pham, 2009).

![Figure 2-28: Results for two Eulerian approach (from Dijkista et al., 2010).](image)

A transitional zone was created at the middle of the pile, so the pile was divided into two parts to simulate different behaviors of soil at the different depths. The pile installation at the upper part was simulated by movement of the soil particles at the pile interface, which was achieved by applying only vertical prescribed displacement, while at the lower part they considered soil particles moving in both horizontal and downward vertical directions. The conical shape for pile
tip created by applying different values of prescribed displacements to the nodes at the pile tip. Baskrap sandy soil was used and results were compared with results obtained from centrifuge tests. Based on the numerical simulation results, the void ratio increased along the pile shaft for a distance of 1.0 to 1.5 times the pile diameter and shear band was created in this region, but the width of the produced shear band was larger than that obtained by other researchers (Pham, 2009).

Qiu et al. (2011) used the coupled Eulerian-Lagrangian method (CEL) to overcome the contact problem and the distortion of the FE mesh in the soil body. They simulated jacking of a circular pile in dry granular soil using a three-dimensional model to make use of the CEL method possible. In the numerical simulation using the CEL method, the Eulerian material is tracked as it flows through the mesh by computing its Eulerian volume fraction (EVF). Each Eulerian element is assigned a percentage, which represents the portion of that element filled with a material. If an Eulerian element is completely filled with a material, its EVF is 1; if there is no material in the element, its EVF is 0. Figure 2.29 depicts the geometry and the Eulerian mesh for the model. In this figure, the first two meters of the soil body are modeled to be material free at the initial step to allow the soil to flow into this region during installation. They simulated 5 m extra pile penetration and compared the results with the classical finite element method. These comparative results for variation of void ratio and radial stress are shown in Figures 2.30 and 2.31.

The authors used 8-noded linear brick elements with reduced integration for soil body. The Abaqus/Explicit software, which adopts the explicit time integration scheme in FE analysis, was used to implement the CEL method. The master-slave contact model using the Coulomb friction rule was applied at the pile-soil interface. The strength reduction factor for interface friction coefficient is assumed to be equal to 1/3. The hypoplastic constitutive model was used to model the granular soil behavior. The pile was cylindrical concrete with a 30 cm diameter that was
modeled as a rigid elastic material. They simulated 5 m pile penetration and compared the obtained results with those produced by the classical finite element method, which models pile as initially prebored for a few centimeters and uses zipper type technique to simulate pile penetration (Qiu et al., 2011). They concluded that the influence zone in the soil body due to the pile installation is about 5 to 10 times the pile diameter.

Figure 2-29: Eulerian mesh for modeling pile installation (from Qiu et al., 2011).
The results obtained from numerical simulation revealed strong agreement between the classical FE method and the CEL method. In both models, dilatation behavior was observed in the soil, along with increases in the soil void ratio at the pile-soil interface zone, and soil contraction was observed at further distances from the pile surface. The radial stress increased in the pile surface due to expansion behavior of soil during pile installation. Figures 2.30 and 2.31 show that the influence zone due to the pile installation is about 5 to 10 times pile diameter. The main
advantage of the CEL method is its ability to capture the loosening behavior of the soil at the ground surface, while the classical FE is unable to capture this behavior because it usually utilizes the prebored technique to simulate pile penetration.

Fakharian et al. (2013) simulated pile installation with Abaqus software by applying the prescribed nodal displacements at the soil-pile interface to create a cylindrical cavity in the soil body. They used the coupled porewater pressure elements to capture porewater pressure variation during the cylindrical cavity expansion. The Mohr-Coulomb elastic perfectly plastic constitutive model was used to define soil behavior. They studied the effects of pile installation in the soil body by analyzing changes in the stresses and porewater pressure during pile installation.

2.1.1.4 Similarities between Pile Installation and Cone Penetration

There is similarity in numerical simulation between pile installation and cone penetration. Abu-Farsakh et al. (1998) presented an axisymmetric FE method for numerical modelling of piezocone penetration test in soft soils. In their method, cone penetration was simulated in two phases: (1) applying a prescribed volumetric cavity expansion equal to the cone size, and (2) applying prescribed shear displacement to cone that cause continuous penetration of piezocone into the soil. Penetration of the cone modeled by prescribed vertical displacements, and they used coulomb contact criteria for the cone-soil interface. An axisymmetric domain with changeable boundary conditions was used for the FE simulation. They assumed the cone is prebored initially at a certain small depth to overcome the penetration problems. They considered two different situations for cone and soil interaction:

- The cone penetration without interface friction
- The cone penetration with interface friction
Cone penetration was simulated by prescribed displacement with a constant rate of penetration (0.02 m/s), and they indicated that it is not possible to use separate interface elements for soil at the cone interface because of infinite stretch at the interface elements during penetration. The MCC model was used to define the soil for cases under strain hardening behavior; however, to overcome the numerical complexity it was assumed that the soil behaves as a perfectly plastic material in the strain-softening zone. Abu-Farsakh et al. (2003) presented an axisymmetric FE method for numerical modelling of piezocone penetration tests in soft soils. This method simulated cone penetration in two continuous steps: first, applying prescribed cylindrical cavity expansion equal to pile radius and second, applying prescribed shear displacement to the cone, causing continuous penetration of piezocone into the soil body. This model was able to capture large deformation, the effect of over consolidation ratio, the excess porewater pressure generated by cone penetration, and cone tip resistance. They observed that the cone tip resistance increased linearly with increasing initial lateral earth pressure. By increasing the overconsolidation ratio (OCR), the differences in the excess porewater pressure values between the cone base and tip increase by a linear trend. They also concluded that the excess porewater pressure dissipation is dominated by the horizontal permeability rather than the vertical component. On the other hand, consolidation mainly occurs in the horizontal direction in cone penetration tests.

Using the remeshing technique and FE modeling, Markauskas et al. (2005) developed a numerical method to capture cone penetration in the saturated porous media. Cone resistance and pore pressure fields under the influence of different soil permeability values were evaluated. They transferred state variables from the old mesh to the new mesh by using two different methods: moving least square method was used to transfer the stress field and the hardening variables, and interpolation method by a polynomial function was used to transfer the porewater pressure fields
at nodes. They implemented this technique into the Abaqus software to simulate a smooth cone penetration problem. The cone was placed at a pre-bored depth in an axisymmetric model with four-noded bilinear elements. The obtained results showed that the excess porewater pressure at the cone tip decreases, and cone resistance increases with increasing permeability. It was concluded that there is no excess porewater pressure when cone penetration is done in soil with permeability greater than $10^{-5}$ m/s.

Susila and Hryciw (2003) incorporated the auto-adaptive remeshing technique into the Abaqus software to overcome the mesh distortion problem during cone penetration in clean sandy soils. The frictional contact model was used for the cone-soil interface to consider the effect of interface on cone resistance. An explicit solution was adopted to solve the governing differential equations because it is more efficient than the implicit solution when the domain is very large and the mesh becomes very fine (Susila and Hryciw, 2003). The general master-slave contact model was used for interface, and the coulomb friction model was adopted to analyze shear behavior at the contact zone. They assumed that the coefficient of interface friction is a function of the internal friction angle of soil.

Walker and Yu (2006) used the explicit dynamic FE model and Abaqus software to simulate cone penetration from the ground level to the desired depth in clays under undrained condition. The ALE remeshing technique was applied to overcome mesh distortion problems. The undrained clay behavior was modeled by perfectly plastic Von-Mises failure criteria. In the explicit dynamic procedure, large numbers of time increments are run and the problem is solved in only one iteration (Walker and Yu, 2006). They encountered two difficulties during installation process in the dynamic model: the mesh distortion problem due to the hour glassing effect and difficulty in applying an initial condition for stress states in the soil domain. The first problem occurs because
of the reduced integration scheme used in the dynamic explicit analysis, and it can be solved by manually controlling artificial stiffness that reaches minimum artificial work done under the specified artificial stiffness. The second problem was solved by defining an initial stress state through applying horizontal and vertical loads as an initial boundary condition.

2.2 **Pile Setup**

Pile setup phenomenon, or an increase in the pile capacity over time after EOD, is related mainly to the following mechanisms:

1) Dissipation of the excess porewater pressure,

2) Thixotropic behavior of the soil, which is dominant mostly in the sensitive clay soil, and

3) Aging, which is a long-term increase in the soil stiffness, common in coarse grain soils.

2.2.1 **Consolidation Setup**

The porewater pressure state in the soil body is changed during pile installation. The generated excess porewater pressure has two components: the first part is generated by expansion in the soil body as the pile penetrates into the soil depth. The second part is the induced excess porewater pressure due to the shear loads, which can cause positive or negative excess porewater pressure in the soil body. When the excess porewater pressures are generated, the soil starts to consolidate. The excess porewater pressure dissipates over time, causing an increase in the effective stress in the soil adjacent to the pile. The increase in effective stress values causes an increase in pile capacity over time (Titi and Wathugala, 1999). Komurka et al. (2003) indicated that the excess porewater pressure consolidation occurs through three phases: phase (1) represents logarithmically nonlinear dissipation; phase (2) is related to logarithmically linear dissipation, which is separated
by the initial time ($t_0$) with the first phase, and phase (3), or aging, which usually correlates with small amount of setup. Figure 2.32 shows all three phases for consolidation.

![Figure 2.32: Schematic representation of the setup phases.](image)

In order to study consolidation in the soil body, researchers (e.g., Abu-Farsakh, 1997; Titi and Wathugala, 1999; Elias, 2008; Damluji and Anbaki, 2010; Basu et al., 2013) have used the coupled analysis for porous media. The coupled analysis considers the medium as a multiphase material and adopts an effective stress principle to describe its behavior. When the medium is saturated with water, the soil particles and water located in the pores are two phases of the coupled analysis. FE programs such as Abaqus can perform the coupled analysis of nonlinear porous media, which have been used by researchers to study pile setup due to consolidation and following static load tests.

### 2.2.2 Thixotropic Setup

Thixotropy has been defined as the “process of softening caused by remolding, followed by a time-dependent return to the original harder state” (Mitchel, 1960). It is a reversible process, which
can occur under constant composition and volume (Mesri, 1993). In geotechnical engineering, thixotropy is associated with an increase in compressive strength (Ltifi et al., 2014). Thixotropy was first developed to define strength-regaining behavior of colloid materials such as ink (e.g., Barnes, 1997). As Mitchel (1960) indicated “when a thixotropic soil is remolded or compacted a structure is induced which is compatible with the externally applied shearing stresses. When shearing stops the soil is left with an excess of internal energy which is dissipated by means of small particle movements and water redistribution until a structure in equilibrium with the at rest forces is created.”

Studies, which investigate the contribution of thixotropy in the pile setup phenomenon, is rare. Most previous researchers explain thixotropy as a mechanism for soil aging after completion of consolidation. However, firstly, thixotropic behavior starts immediately after remolding the soil (i.e., after pile driving), which indicates that part of the soil strength regaining during consolidation can be attributed to thixotropy. Secondly, thixotropy is common mostly in fine-grain soils, while aging is a well-known phenomenon for coarse-grain soils. Therefore, investigating thixotropic effects in pile setup both before and after dissipation of excess porewater pressure is necessary.

Fakharian et al. (2013) studied the effect of the reduction in the soil-pile interface friction angle due to the soil remolding during installation using a reduction parameter $\beta'$. They ran a model with reduced internal friction coefficient and expressed the obtained results as the consolidation setup. Then they increased $\beta'$ with time to match the numerically obtained results with the actual field measurements. Figure 2.33 shows their results from FE numerical simulation, and indicates that the effect of the soil remolding and strength restoration is greater after EOD, rather than at a later time after primary consolidation. Therefore, this kind of strength regaining at
an early stage after EOD is different from the long-term aging and, its contribution should be considered in setup phenomenon.

Figure 2-33: Increase in pile shaft resistance with time from EOD (from Fakharian et al., 2014).

2.2.3 Aging

The dissipation time for the induced excess porewater pressure varies depending on the soil permeability. For clayey soils, this time varies from one week to six months (e.g., Karlsrud and Haugen, 1985; Fellenius, 2008; and Haque et al., 2014). Some studies show that even after dissipation of excess porewater pressure, pile capacity increases (Bullock, 1999 and Augustesen, 2006). Increase in pile capacity after dissipation of excess porewater pressure is called aging. Schmertman (1991) stated that the mechanism of aging could be attributed to creep or secondary compression, particle interference and thixotropic behavior resulting in increasing soil internal friction and shear strength at constant effective stress. The aging effect in cohesive soils is very small and may not make a significant contribution in the ultimate pile capacity (Steward, 2011). Fellenius (2008) indicated: “It would seem that the capacity increase due to aging (as opposed to
dissipation of excess porewater pressure) is rather small and requires very long time to reach appreciable values.”

2.2.4 Empirical Methods to Evaluate Setup

Several empirical methods, which are used to calculate pile setup after pile driving are described in the literature. Some of the common models are introduced in Table 2-1:

Table 2-1: Summary of the main available empirical model for pile setup estimation.

<table>
<thead>
<tr>
<th>Reference</th>
<th>Empirical Equation for Setup</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pei and Wang (1986)</td>
<td>$\frac{R_t}{R_{EOD}} = 0.236[\log(t) + 1]\left(\frac{R_{max}}{R_{EOD}} - 1\right) + 1$</td>
</tr>
<tr>
<td>Skov and Denver (1988)</td>
<td>$\frac{R_t}{R_0} = A \log\left(\frac{t}{t_0}\right) + 1$</td>
</tr>
<tr>
<td>Svinkin and Skov (2000)</td>
<td>$\frac{R_t}{R_{EOD}} = B [\log(t) + 1] + 1$</td>
</tr>
<tr>
<td>Karlsrud et al. (2005)</td>
<td>$\frac{R_t}{R_{100}} = A \log\left(\frac{t}{t_{100}}\right) + 1$</td>
</tr>
<tr>
<td></td>
<td>$A = 0.1 + 0.4(1 - \frac{PI}{50})OCR^{-0.80}$</td>
</tr>
<tr>
<td>Ng et al. (2011)</td>
<td>$\frac{R_t}{R_{EOD}} = A \log\left(\frac{t}{t_{EOD}}\right) + 1$</td>
</tr>
</tbody>
</table>

In this table, parameters are defined as:

$R_t = \text{Pile resistance at time (t)}$

$R_{EOD} = \text{Pile resistance at EOD}$

$R_{max} = \text{Maximum pile resistance}$
\( R_{100} \) = Pile resistance at 100 days after EOD

\( A \) and \( B \) = Setup factor

\( f_c \) = Consolidation factor

\( C_h \) = horizontal coefficient of consolidation determined from CPTu test

\( N_a \) = Weighted average SPT N-value according to the soil depth

\( r_p \) = Pile radius

\( f_r \) = Remolding recovery factor

\( t_{EOD} \) = Time at the end of driving

\( t \) = time elapsed after EOD

In Table 2-1, the setup factor for the first three models is a constant, and these models cannot incorporate the soil and pile properties in the pile setup. However, the last two models relate the pile setup to soil properties, such as the plasticity index (PI), the overconsolidation ratio (OCR), the pile size (\( r_p \)), the hydraulic conductivity (\( C_h \)), and the shear strength (\( N_a \)).

### 2.3 Soil Constitutive Models

During natural deposition, soils experience a variety of load histories, which causes the soil to be overconsolidated (OC) or normally consolidated (NC). In geotechnical engineering, OC and NC soils display different behavior under applied external loads. OC soils exhibit more complicated behavior than NC soils, and they usually have a lower void ratio and higher shear strength (Yao et al. 2007). For engineering applications like deep foundations and piles, the soil type changes with depth due to different history of deposition and previous loading, making it
necessary to use of an appropriate constitutive model that captures behavior of both NC and OC soils. Several elastoplastic constitutive models described in the literature attempt to model soil responses. Most models for clays are based on the critical state soil mechanics (CSSM) concept (Pestana and Whittle, 1999), which have been formulated for laboratory tests in axisymmetric condition. The well-known MCC critical state model proposed by Roscoe and Burland (1968) is successful in describing isotropic NC clay behavior, but it cannot capture soil anisotropy. Since the MCC model assumes elastic response inside the yield surface, its predictability for OC clay is poor (Likitlersuang, 2003). To overcome this deficiency, Dafalias and Hermann (1986) developed the bounding surface models, and then Whittle and Kavvadas (1994) used it to develop the MIT-E3 model. The bounding surface plasticity has been developed to provide smooth transition from elastic to fully plastic state for general loading. In the bounding surface model, the hardening parameter is related to the distance from the current stress state to the stress state at the bounding surface. Yao et al. (2007) and (2012) introduced a unified hardening model using the Hvorslev envelope to define overconsolidated clay behavior. The linear and parabolic form of the Hvorslev envelope were used to adjust the conventional MCC model for heavily overconsolidated clay, which is located on the dry side of the yield surface. Using CSSM and bounding surface theory, Chakraborty et al. (2013) developed a two-surface elastoplastic constitutive to capture strain rate dependency for clay. Chakraborty et al. (2013) and Basu et al. (2014) used the two-surface plasticity constitutive model for clay and implemented it to calculate shaft resistance in piles. Although their models were able to describe both NC and OC clay behaviors, they required estimates of plenty material parameters, which performing several of laboratory tests is required to obtain these parameters. Likitlersuang (2003) introduced a rate-dependent version of the hyperelasticity model, and verified the proposed model by simulating triaxial test results in
Bangkok clay. Zhang et al. (2014) introduced a mathematical model to explain soil shear behavior at the pile interface by using hyperbolic and bi-linear relations between the pile-skin friction, and the relative displacement between the pile and soil. They also used different hyperbolic relations to define soil-softening behavior at the pile interface zone. It is necessary to consider actual soil behavior in a constitutive model because of the disturbance in the soil structure during loading. The disturbed state concept (DSC) model developed by Desai and Ma (1992) is a powerful technique, which was formulated directly based on soil disturbance. The DSC model includes two boundary state responses: relative intact (i) state, and fully adjusted (FA) [or critical (c)] state. Real soil response or averaged (a) behavior is obtained by linear combination of the intact and FA states. In soils, the critical state concept is the most common approach used to describe the FA state response. In addition, an appropriate elastoplastic constitutive model is necessary to describe the soil behavior at intact state. Desai and his coworkers used the elastoplastic hierarchical single surface (HISS) model to define intact state response (Desai et al., 1986; Wathugala, 1990; Shao, 1998; Pal and Wathugala, 1999; Katti and Desai, 1995; Desai et al., 2005 and Desai, 2007; Desai et al. 2011). Hu and Pu (2003) used the conventional hyperbolic constitutive model to capture sandy soil response at the soil structure interface.
3 RESEARCH METHODOLOGIES

3.1 Numerical Modeling of Pile Installation

In this study, pile installation was simulated by FE numerical analysis using the Abaqus software. The simulated piles were assumed to be cylindrical, and an axisymmetric FE domain was used to model the soil and pile. In all of the FE analysis steps, the linear quadrilateral coupled porewater element was used for the whole soil domain to avoid shear locking and to provide more accurate results than nonlinear elements (Shao, 1998; Walker and Yu, 2006).

3.1.1 Model Phases

Pile installation was modeled by volumetric cavity expansion phase followed by a vertical shear displacement (penetration) phase in an axisymmetric FE model. Pile installation was modeled by first applying a series of prescribed displacements in the soil’s axisymmetric boundary in order to create a displaced volume in the soil equal to the size of the pile (volumetric cavity expansion). The pile was then placed inside the cavity; interaction between pile and soil surfaces was activated along with applying vertical penetration until the steady state condition was reached. Adopting the second phase for simulating pile installation allows accurate mobilization of the shear-induced porewater pressure and pile-tip resistance. In the next step, the excess porewater pressure developed during installation was allowed to dissipate for different elapsed times after installation. The static load test was then simulated by applying an additional penetration and hence additional vertical shear displacement at the pile-soil interface until failure. These steps are described in Figure 3.1, which depict porewater pressure change during cavity expansion, initial vertical penetration, pile placement, consolidation, and final vertical penetration in (a) through (e), respectively. Another illustration of the stabilizing porewater pressure at the pile base is depicted
in Figure 3.2. This figure shows that application of the initial shear step is necessary to stabilize
the induced excess porewater pressure at the pile tip.

Figure 3-1: Pore pressure changes during various steps of pile installation simulation: (a) cavity
expansion, (b) pile placement, (c) initial vertical penetration, (d) consolidation, and (e) final
vertical penetration.

3.1.2 Model Geometry

Soil domain was selected around 15-pile diameter in width and 1.50-pile length in depth. The
vertical soil boundary was modeled as roller, and the soil bottom was modeled with fixed
boundary. The geometry of soil and pile and the applied boundary conditions for a typical pile are
shown in Figure 3.3,a; while the finite element mesh is presented in Figure 3.3,b. Finer mesh was
used for the soil elements adjacent to the pile surface. The smallest element size was selected to
be 1/8 pile radius, as suggested by Sheng et al. (2009). Curved shape was adopted for the pile tip
to minimize the sharp corner problem during penetration. The soil top surface was set to be drained
surface for porewater pressure, and the axisymmetric edge’s boundary condition was changed at different phases of the pile installation and setup.

Figure 3.2: Porewater pressure changes during numerical simulation.
3.1.3 Material Constitutive Models

The pile was modeled using a linear elastic material with the unit weight, Young’s modulus and Poison ratio values of $2 \text{ t/m}^3$, $20 \text{ GPa}$, and 0.20, respectively. In this study, two sets of constitutive models were used to describe soil behavior. First, extensive pile installation and the following setup analysis for different test piles was performed using the Abaqus built in models. In this section, the modified Cam Clay (MCC) model was used to describe the behavior of the saturated cohesive soil, and the Drucker-Prager model was used to describe the sandy soil behavior. Second, a new elastoplastic constitutive model was developed based on combination of the critical state theory and the disturbed state concept, and the new model was then verified and applied to test piles.
3.2 **Numerical Modeling of Pile Setup**

Pile setup in driven piles is related to dissipation of the excess porewater pressure (consolidation), soil strength restoration over time at a constant stress state (thixotropic behavior), and changes in soil fabrics and creep effects over time after end of consolidation (aging). In this study, both consolidation setup and thixotropic setup are simulated numerically, but the aging effect is not evaluated because it is less significant in clayey soils.

### 3.2.1 Consolidation Setup

Research studies indicate that dissipation of excess porewater pressure generated during pile installation is dominant in the radial direction from the pile surface (Randolph, 1979 and Basu, 2013). In order to simulate this phenomenon in the FE model, consolidation theory is applied at different times after end of pile installation, using the times corresponding to the static or dynamic load tests after EOD, which were performed on the full-scale piles in the field. At the end of each consolidation step, a vertical shear was applied at the soil-pile interface, using the prescribed displacement to simulate the pile load test. The pile shaft resistance related to consolidation of excess porewater pressure was obtained for a specific consolidation time.

### 3.2.2 Thixotropic Setup

During pile installation, the soil adjacent to the pile within the influence zone will be disturbed and remolded, resulting in reduction in the soil shear strength. After pile installation is completed, the surrounding soil will regain its strength with time. This process is called as thixotropy, which is defined as strength regaining of the remolded soil due to rearrangement of soil particles at a constant water content. Clay structure and mineralogy, water content and concentration of the dissolved ions in the porewater, contribute to thixotropic behavior of remolded soil (Shen et al. 2005). Any activity that causes collapse in the soil’s natural structure and/or breaks the bonds
between soil particles will be followed by regaining in strength with time. This is because of
tendency in the disturbed soils to return natural condition. Therefore, remolded soil that has less
strength than the original soil will recover its strength with time regardless whether or not the soil
is subjected to consolidation. Based on thixotropic investigation of inks, Heymann et al. (1997)
proposed the following relation to describe the thixotropic behavior of inks (Barnes, 1997):

\[
\sigma_y(t) = \sigma_y^0 + [\sigma_y^\infty - \sigma_y^0](1 - e^{-\left(\frac{t}{\tau}\right)})
\]

(3.1)

where \(\sigma_y^0\) represents the yield strength immediately after remolding and \(\sigma_y^\infty\) represents the
yield strength long after remolding. The parameter \(\tau\) is defined as a time constant related to the
material property, and \(t\) is time after remolding.

In order to simulate the thixotropy of disturbed soil due to pile installation, Fakharian et al.
(2013) proposed a time-dependent reduction factor \(\beta\), which applied to the friction angle between
soil and pile, \(\mu\), increases with time to reflect strength regaining after remolding.

During pile installation, soil adjacent to the pile surface is remolded. The remolded zone has
a width 1 to 4 times pile diameter (Yang, 1970; Elias, 2008; and Steward, 2011). In this study, the
effective remolded zone adjacent to the pile in the radial direction was assumed to be 4 pile width
(i.e. the distance at which the soil displacement is equal to 5% of the pile width). The soil
disturbance in the vicinity of the pile affects the soil properties, as well as the soil-pile interface
friction angle. In this study, a critical state MCC constitutive model was used to simulate soil
behavior. The most important soil parameter for the MCC model is the critical state parameter \(M\).
Therefore, for a comprehensive modeling of the thixotropic behavior that considers both aspects,
the time-dependent \(\beta\) parameter was used to define the time change in \(M\) and \(\mu\), as follows:

\[
M(t) = \beta(t)M \quad \text{and} \quad \mu(t) = \beta(t)\mu
\]

(3.2)
By following the similar formulation of Hymann et al. (1977), the evolution of the parameter \( \beta \) with time can be defined using as follows:

\[
\beta(t) = \beta(\infty) - [\beta(\infty) - \beta(0)] e^{-\frac{t}{\tau}}
\]  

(3-3)

where \( \beta(0) \) is a reduction parameter immediately after pile installation, and is related to soil sensitivity. \( \beta(\infty) \) is the regained strength a long time after soil disturbance. Shui-long et al. (2005) showed that the strength regaining after a long time could be 1, or a value less than 1, for pure clayey soils, to a value greater than 1 for soil with a specific percentage of salt or cement slurry. Figure 3.4 shows the evolution of \( \beta \) with time after disturbing and remolding the soil structure. Back calculation of \( \beta(0) \) from pile resistance obtained for TP1 immediately after EOD revealed a value of \( \beta(0) = 0.75 \). Besides, the average sensitivity \( S_r \) value for the Bayou Lacassine site, as obtained from the field vane shear tests was about 3. For TP1, the relation between \( \beta(0) \) and \( S_r \) is \( \beta(0) = (S_r)^{-0.3} \). In this paper, the \( \beta(\infty) \) value was assumed to be 1. The time constant \( \tau \) is related to the soil properties, especially the soil sensitivity. In this study, \( \tau \) was assumed to be equal to the soil \( t_{90} \), which is the time for 90% dissipation of the excess porewater pressure at the pile surface. This assumption means that consolidation due to induced excess porewater pressure and thixotropic behavior are assumed to be completed at the same time after the remolding and disturbance of the soil structure. In this study, the \( t_{90} \) was measured using the piezometers installed at pile faces for the instrumented piles at Bayou Lacassine. However, a direct value for \( t_{90} \) at pile surface is usually not available in the absence of field measurement. Liyanapathirana (2008) proposed the following relation between the \( t_{50} \) value obtained from piezocone dissipation tests and the value for dissipation at pile surface:

\[
\frac{t_{50}^{\text{pile}}}{t_{50}^{\text{piez}}} = \left( \frac{R_{\text{pile}}}{R_{\text{piez}}} \right)^n
\]  

(3-4)
where $R$ represents pile or piezocone radius, and $n$ is an exponential constant. However, Liyanapathirana (2008) suggested $n=2$. However, Based on analysis of the data obtained from field tests on instrumented test piles TP1 and TP3 at the Bayou Lacassine bridge site, the $n$ value found to be equal to 1.50. After finding $t_{50}^{\text{pile}}$, the corresponding $t_{90}^{\text{pile}}$ was obtained using the following relation:

$$\frac{t_{90}}{t_{50}} = \frac{T_{90}}{T_{50}}$$

(3-5)

where $T_{50}$ and $T_{90}$ are time factors for 50% and 90% consolidation, respectively.

Figure 3-4: Change in parameter $\beta$ with time.

3.2.3 Aging Setup

Aging is a phenomenon, which occurs after dissipation of the induced excess porewater pressure. Dissipation time for the induced excess porewater pressure varies depending on soil permeability. For a single pile driven in clay soils, this time varies from one week to six months (Karlsrud and Haugen, 1985; Fellenius, 2008; and Haque et al., 2014). Some studies show that pile capacity increases even after dissipation of the excess porewater pressure (Bullock, 1999 and Augustesen, 2006). The aging effect in cohesive soils is very small and may not contribute
significantly to ultimate pile capacity (Komurka et al., 2003). In this study, the effect of aging was not considered.

3.3 Soil Constitutive Models

A constitutive model defines the stress-strain relationship of material during loading through a mathematical formulation (Elias, 2008). Geo-materials like soils show nonlinear behavior during loading because of changes in properties under stress state; therefore, an appropriate and advanced elastoplastic constitutive model is required to model soil response.

In the present study, based on critical state (CS) concept, the MCC model was selected to study the soil response in the framework of the disturbed state concept (DSC). The model will be referred to as the Critical State and Disturbed State Concept (CSDSC) model. Combination of the CS theory and DSC yields a new elastoplastic constitutive model, which can capture soil response appropriately. The proposed model was evaluated for both NC and OC soils, and it has good capability of defining saturated soil behavior. In the next section, both components of the proposed model (the MCC model and The DSC theory) will be reviewed.

3.3.1 Modified Cam Clay Model

Based on the CS theory, the Cam Clay model was developed by Roscoe and Schofield (1963) and then modified by Roscoe and Burland (1968) to the MCC model in order to study the clayey soil behavior under loading, unloading and reloading. The MCC model has been widely used in the past decades to define soil behavior because: first, it captures realistic soil behavior better than other conventional models, such as Mohr-Coulomb or Von-Misses models; second, it is a simple model with less parameters than other advanced soil models.
The modified Cam-clay yield locus is assumed to have an elliptical shape, as shown in Figure 3.5. The equation of the yield surface in triaxial stress space is:

\[ F = q^2 - M^2[p'_0(p' - p')] = 0 \]  

(3 - 6)

Where:

- \( p' \) is the general volumetric stress or hydrostatic stress, which is \( p' = (\sigma'_1 + 2\sigma'_3)/3 \) for triaxial stress state.
- \( q \) is deviatoric stress, which is \( q = \sigma'_1 - \sigma'_3 \) for the triaxial condition.
- \( M \) is the slope of the critical state line on the \( p-q \) plane.
- \( p'_0 \) is the pre-consolidation pressure.

Figure 3-5: Elliptical yield surface for Modified Cam Clay model in \( p'-q \) plane.

The Cam Clay model was developed based on the volumetric behavior of saturated soil under shear loading, unloading, and re-loading as shown in Figure 3.6. In this figure, the virgin consolidation or normally consolidation line (NCL) represents the loading behavior, and the unloading-reloading line (URL) represents the shear unloading and reloading behavior.
Based on Figure 3.6, the normal compression line (NCL) can be written in the following form:

\[ \nu = \nu_\lambda^0 - \lambda \ln p' \quad (3 - 7) \]

and the unloading-reloading line (URL) can be expressed as:

\[ \nu^e = \nu_\kappa^0 - \kappa \ln p' \quad (3 - 8) \]

where \( \lambda \) and \( \kappa \) are slope of the NCL and URL, respectively, \( \nu_\lambda^0 \) and \( \nu_\kappa^0 \) are the intercepts on the lines at \( p' = 1 \), and \( \nu \) is specific volume (\( \nu = 1 + \epsilon \)).

### 3.3.2 Disturbed State Concept (DSC)

The conceptual framework of the disturbed state model is based on the cyclical behavior of a material from its “cosmic state” to the “engineering materials” state and its tendency then to return to its initial cosmic state under applied loads. Desai (2001) represented this concept with an historical convolution shown in Figure 3.7, which depicts material in its densest “cosmic” state (point “o”). For purpose of this concept, the cosmic material has been changed to the engineering material state under various loads during its history (middle part of figure). Then, with application
of more load, it fails and changes to the “fully adjusted” state, and then it tends to become in position pointed ‘o’. “Perhaps the state “o” and ‘o’ are the same” (Desai, 2001).

In DSC, a deforming material element is assumed to consist of various components. For soils, it is assumed to have two components: continuum or relative intact (RI) and dis-continuum or fully adjusted (FA) phases. These components interact and merge into each other, transforming the initial RI phase to the ultimate FA phase. The transformation occurs due to continuous modifications in the microstructure of the material (Desai, 2012). The disturbance or microstructural changes act as a coupling mechanism between the RI and FA phases.

![Material existence in nature](image)

Figure 3-7: Material existence in nature (from Desai, 2001).

3.3.2.1 Relative Intact State (RI)

The initial continuum or theoretical maximum density state defines the material intact state. As materials in most cases do not exist in their theoretical maximum density condition, their status in the initial continuum case is called “Relative Intact”. An elastoplastic model usually is used to define the relative intact behavior. The Hierarchical single surface (HISS) model was used
repeatedly in prior research (e.g. Desai and Ma, 1992; Katti, 1991; Shao, 1998; Titi and Wathugala, 1999; Desai, 2005; Desai, 2007; Desai et al., 2011), but in this study the MCC model is adopted to define the relative intact behavior because the proposed model has less and easier to extract parameters than the HISS model.

3.3.2.2 Fully Adjusted State (FA)

The final condition material will approach under applied loads is called fully adjusted (FA). The FA state is an asymptotic state in which material may not be further disturbed. In this condition, engineering materials may disintegrate into a loose cluster form, with no strength in unconfined situation (Desai, 2001). This state is not measurable in the laboratory because testing material fails before reaching this state, and test machine stops. For soils, the critical state can be used to represent FA behavior (Desai, 2001). For soils at critical state, the void ratio under shear loading is a function of the hydrostatic stress $p'$ with the following relation:

$$ e = e^0 - \lambda \ln p' $$ \hspace{1cm} (3-9)

where $e^0$ is the void ratio at $p' = 1$. At the critical state, the maximum shear stress that material can carry is given by:

$$ q = Mp' $$ \hspace{1cm} (3-10)

3.3.2.3 Observed (average) state

Actual behavior of material is a combination of theoretical behavior of two interacting material in the RI and FA reference states. The actual (observed or average) behavior is a weighted average material response obtained from these two reference states (RI and FA). At the beginning of application of external load, the RI response has more effect on the overall response of the material, but with increased application of load, material particles are displaced and the overall
material response approaches to the FA state. This transition from RI response to the FA response is depicted in Figure 3.8. The average response can be obtained by a linear combination of the RI and FA states and by using the disturbance function $D$ with the following relation:

$$\sigma_{ij}^a = (1 - D)\sigma_{ij}^l + D\sigma_{ij}^c$$

(3-11)

or in the incremental form:

$$d\sigma_{ij}^a = (1 - D)d\sigma_{ij}^l + Dd\sigma_{ij}^c + dD(\sigma_{ij}^c - \sigma_{ij}^l)$$

(3-12)

where $a$, $i$, and $c$ represent the observed, intact and fully adjusted (critical state) responses, respectively, and $D$ is the disturbance function, which combines the intact and critical state responses to obtain the averaged (or observed) response.

Figure 3-8: Schematic presentation of stress-strain curve for DSC (from Shao, 1998).
Figure 3.9 explains the DSC schematically: material starts from the RI state (when $D=0$), and then starts to disturb until it reaches the FA state (when $D=1$).

![Figure 3.9: Representation of DSC (from Desai, 2001).](image)

### 3.3.2.4 Disturbance parameter $D$

As the material deforms disturbance occurs, resulting in an increase in the disturbance function $D$. At the beginning of loading, $D$ is zero (or a small value depending on the initial condition); as the load increases, deformation increases and $D$ increases. When $D$ approaches 1, the soil is in the critical state. The disturbance function $D$ can be related to the plastic strain trajectory ($\xi$) with an exponential equation proposed by Desai (1984):

$$D = 1 - e^{-A\xi^B} \quad (3-13)$$

where $A$ and $B$ are the material parameters that can be obtained from triaxial test results, and $\xi$ is plastic strain trajectory, which is related to the plastic strain with $\xi = \int (\varepsilon_p^a \cdot \varepsilon_p^p)^{1/2}$. The parameters $A$ and $B$ control the evolution pattern of the disturbance parameter $D$ as shown in Figure 3.10, which indicates that increasing either of these parameters yields an increase in $D$ function.
Figure 3-10: Effect of disturbance parameters $A$ and $B$ on soil disturbance function $D$.

Incremental change of disturbance function $dD$ can be obtained by:

$$dD = \frac{\partial D}{\partial \xi} \frac{\partial \xi}{\partial \varepsilon^p} \, d\varepsilon^p$$

(3.14)

Deviatoric plastic strain is the most common strain component used to define disturbance function. Using deviatoric plastic strain, and combining e. (3.13) and (3.14) the following relation is obtained:

$$dD = AB \xi_d^{B-1} e^{-A \cdot \xi_d} d\xi_d^p$$

(3.15)

Also

$$d\xi_d = \frac{d\xi_d}{dE^p_{ij}} \cdot dE^p_{ij}$$

(3.16)

where $d\xi_d = (dE^p_{ij}, dE^p_{ij})^{1/2}$, and $dE^p_{ij} = d\varepsilon^p_{ij} - 1/3d\varepsilon^p_{kk}\delta_{ij}$. Based on the plasticity theory, for a specific yield surface $F$ and in the case of associated flow rule, the plastic strain increment is related to the deferential of the yield function $F$ with respect to the stress tensor by the following equation:
\[ d \varepsilon_{ij}^p = \lambda \frac{\partial F}{\partial \sigma_{ij}} \] (3.17)

where \( \lambda \) is the scalar consistency parameter or plastic multiplier. By combining Equations (3.16) and (3.17), we can obtain the following expression for \( dE_{ij}^p \):

\[ dE_{ij}^p = d \varepsilon_{ij}^p - \frac{1}{3} d \varepsilon_{kk}^p \delta_{ij} = \lambda \left( \frac{\partial F}{\partial \sigma_{ij}} - \frac{1}{3} \frac{\partial F}{\partial \sigma_{kk}} \delta_{ij} \right) \] (3.18)

In addition, \( d \xi_d \) will be obtained as:

\[ d \xi_d = (dE_{ij}^p, dE_{ij}^p)^{1/2} = \lambda \left[ \left( \frac{\partial F}{\partial \sigma_{ij}} - \frac{1}{3} \frac{\partial F}{\partial \sigma_{kk}} \delta_{ij} \right) \cdot \left( \frac{\partial F}{\partial \sigma_{ij}} - \frac{1}{3} \frac{\partial F}{\partial \sigma_{kk}} \delta_{ij} \right) \right]^{1/2} \] (3.19)

Equation (3.19) after some mathematical rearrangement yields to:

\[ d \xi_d = \lambda \left[ \frac{\partial F}{\partial \sigma_{ij}} \frac{\partial F}{\partial \sigma_{ij}} - \frac{1}{3} \frac{\partial F}{\partial \sigma_{ij}} \frac{\partial F}{\partial \sigma_{kk}} \right]^{1/2} \] (3.20)

By combining Equation (3.15) and Equation (3.20), the incremental change in the disturbance function is obtained as:

\[ dD = AB \xi_d B^{-1} e^{-A^T \xi B} \lambda \left[ \frac{\partial F}{\partial \sigma_{ij}} \frac{\partial F}{\partial \sigma_{ij}} - \frac{1}{3} \frac{\partial F}{\partial \sigma_{ij}} \frac{\partial F}{\partial \sigma_{kk}} \right]^{1/2} \] (3.21)

In Equation (3.21), the plastic multiplier \( \lambda \) is obtained based on the chosen constitutive model for intact response. For example, in the case of using the HISS model for intact behavior, \( \lambda \) can be defined as:

\[ \lambda = \frac{\frac{\partial F}{\partial \sigma_{ij}^e} C_{ijkl} e_{kl}^e}{\frac{\partial F}{\partial \sigma_{ij}^e} C_{ijkl} \frac{\partial F}{\partial \sigma_{kl}} \lambda^{1/2}} \] (3.22)
In the above, $C_{ijkl}^e$ denotes the elastic stress-strain matrix, and $d\varepsilon_{kl}^i$ the incremental intact strain. If the MCC model is used to represent intact material response, the $\lambda$ is modified to:

$$\lambda = \frac{\partial F}{\partial \sigma_{ij} \varepsilon_{ijkl} \partial \sigma_{kl}} \left[ 1 + e^{\frac{1}{\lambda}} \right] \frac{\partial F}{\partial \sigma_{ij}} \frac{\partial F}{\partial \sigma_{kl}} \frac{\partial F}{\partial \sigma_{ij}} \frac{\partial F}{\partial \sigma_{kl}}$$

(3-23)

In this section, $\lambda^*$ is selected for the slope of the NCL to avoid confusion; $e$ is the void ratio.

Substituting Equation (3-23) into Equation (3-21) yields:

$$dD = \left( A B C D E \right) \frac{\partial F}{\partial \sigma_{ij} \varepsilon_{ijkl} \partial \sigma_{kl}} \left[ 1 + e^{\frac{1}{\lambda}} \right] \frac{\partial F}{\partial \sigma_{ij}} \frac{\partial F}{\partial \sigma_{kl}} \frac{\partial F}{\partial \sigma_{ij}} \frac{\partial F}{\partial \sigma_{kl}} \frac{\partial F}{\partial \sigma_{ij}} \frac{\partial F}{\partial \sigma_{kl}}$$

(3-24)

### 3.3.3 Proposed Constitutive Model (CSDSC)

The MCC model is generally suitable to describe the NC clay behavior, but results obtained for OC clay using the MCC model is not satisfactory (Yao et al., 2007). The MCC model gives poor predictions for heavily OC clay, in particular for shear strains, because it assumes a purely elastic and reversible behavior inside state boundary surface (Likitlersuang, 2003). To improve the OC clay response, some researchers have proposed advanced constitutive models, such as:

1) The bounding surface model proposed by Dafalias and Hermann, (1982) and Dafalias (1986);

2) The sub-loading surface models proposed by Hashiguchi (1978); and

3) The three-dimensional unified hardening model using the Hvorslev surface, developed by Yao et al. (2007) and Yao et al. (2012).
In this study, an elastoplastic constitutive model is proposed in the framework of the DSC. The proposed model can simulate both NC and OC clay behavior and provides an elastoplastic response inside the state boundary surface (initial yield surface), with a smooth transition from elastic to plastic response. The MCC model was adopted in the DSC as a constitutive model to describe the intact behavior. For each increment $d\varepsilon$, constitutive equation for the MCC model is solved in the framework of DSC. For MCC model, the following differential equations define the material behavior:

\[ d\sigma = C^{ep} d\varepsilon \]

And

\[ dp_0 = \lambda \frac{\partial g}{\partial \sigma_{ij}} \]

where $d\sigma$ is the incremental stress tensor, $d\varepsilon$ is the incremental strain tensor, $C^{ep}$ is the elastoplastic constitutive matrix, $dp_0$ is rate of the hardening parameter (which is preconsolidation mean stress for MCC model), $\lambda$ is the plastic multiplier was defined in Equation (3-23), and $g$ denotes the plastic potential function. Since the associative flow rule is used in MCC model, $g$ can be replaced by the yield function, $F$, described in Equation (3-6). For the MCC model, the elastoplastic constitutive matrix, $C^{ep}$, can be obtained in the following form:

\[ C_{ijkl}^{ep} = C_{ijkl}^e - \frac{c^{e}_{ijkl} \frac{\partial F}{\partial \sigma_{ij}} \frac{\partial F}{\partial \sigma_{kl}} C_{pqkl}^{p}}{H + \frac{\partial F}{\partial \sigma_{ij}} C_{ijkl}^{e} \frac{\partial F}{\partial \sigma_{kl}}} \]  

(3-26)

where $C^e$ is the tangent modulus, and depends on the stress state through the elastic bulk modulus, $K$, and the shear modulus, $G$, in the following form:
\[ C^e = \begin{bmatrix} K + 4/3G & K - 2/3G & K - 2/3G & 0 & 0 & 0 \\ K - 2/3G & K + 4/3G & K - 2/3G & 0 & 0 & 0 \\ K - 2/3G & K - 2/3G & K + 4/3G & 0 & 0 & 0 \\ 0 & 0 & 0 & G & 0 & 0 \\ 0 & 0 & 0 & 0 & G & 0 \\ 0 & 0 & 0 & 0 & 0 & G \end{bmatrix} \]  

(3-27)

where

\[ K = \frac{\delta p_r}{\delta \varepsilon^p} = \frac{1+\epsilon}{\kappa} p' \]  

(3-28)

and

\[ G = \frac{3(1-2\nu)}{2(1+\nu)} \]  

(3-29)

In Equation (3-26), H is the parameter related to hardening behavior and is defined as:

\[ H = -\frac{\partial F}{\partial p_{r0}} \]  

(3-30)

Based on the plasticity theory, the following relations are valid:

\[ d\varepsilon^p_v \lambda \frac{\partial F}{\partial p'} \]  

(3-31)

\[ d\varepsilon^p_v = \frac{\lambda}{1+e} \frac{dp'_{0}}{p'_{0}} \]  

(3-32)

where \( d\varepsilon^p_v \) is rate of the volumetric plastic strain.

Combining Equations (3-30) to (3-32), the hardening parameter H will be obtained as:

\[ H = \frac{1+\epsilon}{\lambda - \kappa} p'_{0} \frac{\partial F}{\partial p'} \]  

(3-33)

The hardening parameter H can be observed in the denominator of Equation (3-23). In this study, the disturbance function was applied to the critical state parameter, \( M \), instead of stress.
states. In this condition, three states for M were defined: 1) $M_l$, which represents the critical state parameter for intact material, 2) $M_c$, the critical state parameter indication for the FA material, and 3) $M_a$, which is the actual, averaged, or observed value for the critical state parameter. The following relation between these parameters can be defined:

$$M_a = (1 - D)M_l + DM_c$$

(3.34)

where $M_a$ is the averaged or observed value for the critical state parameter at each stage of loading process. Figure 3.11 describes the evolution of the critical state parameter $M_a$ during shear loading. At the initial stage of shear, the soil is assumed to be undisturbed ($D=0$ and $\xi_d = 0$), which means Equation (3.34) reduces to $M_a = M_l$. However, with the proceeding of the applied load, the soil disturbs; the plastic strains develop in the soil body; the values of $D$ and $\xi_d$ increase; and eventually the $D$ value approaches 1. At this point, the soil reaches the critical state (i.e. $M_a = M_c$) condition. Values of $M_c$ and $M_l$ were assumed to be constant, so the incremental form for Equation (11) can be expressed as follows:

$$dM_a = (M_c - M_l)dD$$

(3.35)

By combining Equation (3-24) and (3-35), the incremental change for observed critical state parameter $dM_a$, is obtained as:

$$dM_a = (M_c - M_l) \frac{\partial F}{\partial \sigma_{ij}} \left[ 1 + \frac{1}{\lambda^2} \right] d\sigma_{i} \left[ \frac{1}{\lambda^2} \right] d\sigma_{m}$$

(3.36)

\[\frac{\partial F}{\partial \sigma_{ij}} \left[ \frac{1}{\lambda^2} \right] d\sigma_{i} \left[ \frac{1}{\lambda^2} \right] d\sigma_{m} = \frac{1}{\lambda^2} \frac{\partial F}{\partial \sigma_{ij}} d\sigma_{i} \left[ \frac{1}{\lambda^2} \right] d\sigma_{m} \]
As indicated by Sloan et al., (2001) and Zhang (2012), tangent modulus $C^\iota$ cannot be used directly in numerical analysis of critical state models, because $K$ and $G$ are non linear within the finite strain increment. Therefore, the secant modulus, which is obtained from integration of Equation (3.28), replaced with tangent modulus as:

$$\tilde{K} = \frac{p'}{\Delta\varepsilon^\iota} \left( \exp \left( \frac{1+\nu}{\kappa} \Delta\varepsilon^\iota - 1 \right) \right)$$

(3-37)

where $p'$ is the effective hydrostatic stress at the start of the strain increment $\Delta\varepsilon^\iota$. By assuming that the Poisson’s ratio stays constant during loading, the secant shear modulus can be obtained as:

$$\tilde{G} = \frac{3(1-2\nu)}{2(1+\nu)} \tilde{K}$$

(3-38)

In the proposed model, MCC model runs in each increment (or sub-increment). However, while the soil shears, the critical state parameter $M$ evolves gradually from $M_i$ value to $M_c$ value based on the amount of developed plastic strain in each increment, in accord with the DSC theory.
Figure 3.12 shows formulation of the proposed model in the $p' - q$ space. The point $A$ represents the stress state at the beginning of the strain increment $d\varepsilon_n$. The MCC model is used to solve the governing equations for $d\varepsilon_n$ using $M^n_a$, and the new stress state is obtained at point $B$, which is located on the yield surface $\alpha_n$. The updated value for the average critical state parameter $M^{n+1}_a$ is then obtained from the incremental value of $dM_a$ from Equation (3.36) for use in the next increment. The imaginary yield surface $i_{n+1}$ will then be defined using the updated critical state parameter $M^{n+1}_a$ and the hardening parameter $p'^{n+1}_c$ (the prime index in $p'_c$ removed for simplicity). The current stress state (point B) is located inside the imaginary yield surface $i_{n+1}$, which results an elastoplastic behavior for material in the next steps, until the stresses reach critical state. The MCC model is then solved using the new strain increment $d\varepsilon_{n+1}$ to reach point C and so on.

![Diagram showing stress state transitions](image)

Figure 3-12: The proposed (CSDSC) model representation in p'-q space.

The main advantage of this approach is the possibility of specifying a small value close to zero for $M_i$ since the actual material behavior is captured by the disturbance parameters regardless of the chosen value for $M_i$. By choosing a very small value for $M_i$, the plastic behavior inside the yield surface is achieved; leading to a smooth transition between the elastic and plastic behavior.
For each strain increment, $d\varepsilon$, the elastic and the plastic portions are determined using the yield surface intersection parameter $\alpha_{\text{inter}}$ as follows:

$$d\varepsilon^e = \alpha_{\text{inter}}. d\varepsilon$$

and

$$d\varepsilon^p = (1 - \alpha_{\text{inter}}). d\varepsilon$$

(3-39)

In Equation (3-39), $\alpha_{\text{inter}}$ is a parameter that represents the intersection of stress state with yield surface, and its value is obtained using an appropriate technique explained in next section. Higher values of $\alpha_{\text{inter}}$ indicate dominant elastic response, and lower $\alpha_{\text{inter}}$ show dominant plastic response. A value of $\alpha_{\text{inter}} = 0$ indicates that under strain increment $d\varepsilon$ pure plastic deformation occurs, while a value of $\alpha_{\text{inter}} = 1$ results in pure elastic deformation. At the initial stage after loading (D=0), the elastic behavior is dominant (point B is far from yield surface $t_{n+1}$).

As loading increases the $\alpha_{\text{inter}}$ value decreases, the elastic response disappears, and the plastic behavior becomes dominant until it reaches the fully plastic response at D=1 (point B locates on the yield surface and critical state line).

3.3.4 Incorporating Thixotropy Effect in the Proposed (CSDSC) Model

Severe soil remolding under shear loading is obvious during pile installation. Values of reduction in soil strength due to the remolding process, which vary in soil body depending on the amount of soil disturbance. A similar formulation to the disturbance function D is also proposed in this study, which relates the initial reduction parameter $\beta(0)$ to the deviatoric plastic strain trajectory:

$$\beta(0) = \beta_R + (1 - \beta_R) e^{-A \xi_d \beta}$$

(3-40)
where $\beta_R$ is the $\beta$ value for the fully remolded soil, at which there is maximum reduction of the soil strength during shearing, and its value can be related to soil sensitivity. In order to reduce complexity, the disturbed state parameters $A$ and $B$ were used to introduce a relation between $\beta(0)$ and $\xi_d$ in Equation (3.40). Figure 3.13,a and 3.13,b are schematic representations of the variations of $D$ and $\beta(0)$ versus the deviatoric plastic strain trajectory, and show that as the soil disturbs, the $D$ value approaches unity by proceeding the plastic strain, and the $\beta(0)$ approaches to $\beta_R$.

![Diagram of disturbance function D and soil strength reduction factor $\beta(0)$](image)

**Figure 3.13:** Variation of soil characteristics during shear loading: (a) disturbance function $D$, and (b) the soil strength reduction factor immediately after remolding, $\beta(0)$.

In the original DSC models, correction of $D$ is usually required to conform to the observed response. One advantage of this proposed model is that the obtained stresses in each increment are equal the observed response. Therefore, the plastic strain and disturbance function are calculated from the observed stress state, and no correction is required for $D$. A summary of the steps required to implement the CSDSC model is:

1. For a given strain increment $d\varepsilon$, solve the constitutive equations using the MCC model and implement an appropriate integration scheme to determine the current stress state (Point B in Figure 3.12) and corresponding $P_c$. 

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2. Calculating the disturbance function increment, $dD$, based on the induced plastic strain values using the resulting formulation. Then, calculate $dM_a$ using equation (3.36) to update the $M_a$ value for use in the next increment.

3. An imaginary yield surface is defined based on the updated $M_a$ and $p_c$ values. This step causes the current stress states (point B in Figure 3.12) to stay inside the imaginary yield surface.

4. Run the MCC model using the imaginary yield surface and the $M_a$ value obtained from step 2, which yields a new stress state at point C and a new hardening parameter $p_c$.

5. Repeat Steps 2 to 4 until the stress state reaches the critical state ($M_a = M_c$) condition.
4 IMPLEMENTATION AND VERIFICATION

4.1 Implementation of the Proposed Model

For an elastoplastic constitutive model, usually the constitutive matrix that defines the stress-strain relationship varies during analysis. Because the constitutive matrix is not constant, special solution technique is required to solve the governing deferential equation (Elias, 2008). To implement a constitutive model in Abaqus software, the user defined material behavior (UMAT) subroutine for Abaqus/Standard interface is used. The subroutine updates stresses and state variables at the end of every increment, and it returns the updated stresses, the material Jacobian matrix, and the state variables. Based on the Abaqus manual (2012), Any UMAT includes series of variables. In this study, the following variables have been used:

1- **DDSDDE (i,j)**: The Jacobian matrix of the constitutive model, \( J = \frac{\partial \Delta \sigma}{\partial \Delta \varepsilon} \), where \( \Delta \sigma \) is the stress increment and \( \Delta \varepsilon \) is the strain increment. These variables must return at the end of any increment. Here, the Jacobian matrix can be defined as the elastoplastic material constitutive matrix, or \( J = C^{ep} \).

2- **STRESS (NTENS)**: this is a one-dimensional array, which is passed in as the stress tensor at the beginning of the increment and must be updated in this routine to be the stress tensor at the end of the increment (Abaqus user manual, 2012). \( NTENS \) indicates the number of stress components; for example \( NTENS=6 \) for three-dimensional problems and \( NTENS=4 \) for axisymmetric problems.

3- **STATEV (NSTATV)**: The solution-dependent state variables are stored in this array. The solution-dependent state variables are passed in as the values at the beginning of the increment and should return as the values at the end of any increment. \( NSTATV \) declares the number of state variables. For the proposed model, state variables include:
a. Void ratio, $e$,

b. The pre-consolidation pressure, $p'_0$,

c. The averaged (observed) critical state parameter, $M^a$,

d. The Plastic strain trajectory, $\xi$, and

e. The plastic strain increment, $de^p$.

The procedure for solution includes determining the strain increment $\Delta \varepsilon$ at each Gauss point based on the external loading and element boundary condition, and then integrating the material constitutive equation using an appropriate algorithm with respect to the obtained strain increment $\Delta \varepsilon$. The following section presents the integration scheme used in this study.

4.1.1 Integration of Elastoplastic Equations

As mentioned earlier, the core of the proposed model is the critical state plasticity. Two sets of algorithms have been used to integrate the MCC model to obtain the unknown increment in the stresses: 1) implicit algorithms (e.g., Borja and Lee, 1990; Borja, 1991) and 2) explicit algorithms (e.g., Sloan et al., 2001). In the first method, the hardening law and gradients are obtained at unknown stress states, and the method yields a system of non-linear equations, which should be solved using an appropriate iterative process such as Newton Raphson scheme, and which requires determination of second derivatives of yield surface in the iterative analysis. Implicit methods have two main advantages: First, the resulting stress state at the end of analysis satisfies the yield criterion, and there is no need to correct stress state. Second, there is no need to find the intersection point of the stress state with yield surface where that stress state changes from elastic state to plastic state. However, the implicit schemes for critical state family models yield to the complicated formulation because of complexity of the soil plasticity model (Sloan et al., 2001).
The explicit algorithm requires only the first derivative of yield function and potential function, and it follows directly the elastoplastic constitutive equation. Therefore, it is applicable for most complicated constitutive models because, unlike the implicit method, it does not need to solve a system of non-linear equations for every Gauss point. Sloan (1978) proposed an explicit scheme to implement an elastoplastic constitutive equation. For integration of the model, the modified Euler algorithm was implemented to find and control errors during integration. This model was suitable for constitutive models where all deformation inside the yield surface is linear elastic. Sloan (2001) proposed a new version of the explicit sub-stepping method to cover generalized critical state models with non-linear elastic response inside the yield surface. In this study, the modified Euler algorithm with the explicit sub-stepping technique proposed by Sloan et al., (2001) was used to solve governing differential equations.

4.1.2 Yield Surface Intersection

For a given strain increment $\Delta \varepsilon$, the stress state is updated using the integration schemes and the secant modulus described in section 3.3.3. When a stress state locates and stays the inside yield surface, stresses can be updated using only the secant modulus. If a stress state, which is initially located inside the yield surface, exceeds the yield surface under the strain increment $\Delta \varepsilon$, the intersection point of the stress state with the yield surface must be found. Figure 4.1 graphically represents the yield surface intersection. The intersection point is obtained by defining a multiplier $\alpha$, which defines strain increment portion $\alpha_{inter.}$ that stays inside the yield surface and satisfies the following non-linear equation:

$$f((\sigma + \Delta \sigma), P_0) = 0$$

where

$$\Delta \sigma = \alpha_{inter.} C : \Delta \varepsilon$$
Several numerical techniques that were developed to find the $\alpha_{\text{inter}}$, including bisection, Regula-falsi, Newton-Raphson, and Pegasu schemes. The Pegasu intersection scheme was used in this study because it is unconditionally convergent and there is no need for derivatives of the yield surface or the plastic potential functions (Sloan et al., 2001 and Zhang, 2012).

4.1.3 Correction of Stress State to Yield Surface

Due to the linearization technique of the explicit integration algorithms, the stress states usually drift away from the yield surface at the end of each step. This drift may be very small compared to the stress increment in that step, but can accumulate to a large error value after huge number of steps of solution (Zhang, 2012). A combined consistent correction scheme, which provides an enhanced stability of the whole correction procedure and it was developed by Sloan (2001) was used in the present study. However, regardless which correction method is used, the elastic predictor followed by the plastic corrector controls the whole correction process. Geometrical presentation of the elastic predictor and plastic corrector is shown in Figure 4.2.
Figure 4-2: Graphical explanation for general correction concept (from Anandarajah, 2010).

Based on Figure 4.2, the following relation is valid for a strain increment from time $t$ to time $t + \Delta t$:

$$d\sigma = C d\varepsilon - \lambda_{corr} \cdot C \frac{\partial f}{\partial \sigma}$$

or

$$\Delta \sigma = \Delta \sigma^{ep} + \Delta \sigma^{pc}$$

or

$$\sigma_{n+1} = \sigma_n + \Delta \sigma^{ep} + \Delta \sigma^{pc}$$

or

$$\sigma_{n+1} = \sigma_{n+1}^{tr} + \Delta \sigma^{pc}$$
The uncorrected stresses and hardening parameters will be processed through a consistent correction scheme. The developed model is an isotropic critical state concept constitutive model, and the first Taylor polynomial of the yield function $f$ about the point $(\sigma, p_0)$ for this model can be written as:

$$f = f(\sigma, p_0) + \frac{\partial f}{\partial \sigma} \delta \sigma + \frac{\partial f}{\partial p_0} \delta p_0 = 0 \quad (4.2)$$

Here $\delta \sigma$ and $\delta p_0$ will be viewed as a small correction to the current $\sigma$ and $p_0$. Such corrections make the change of stress and hardening parameters simultaneously while leaving the total strain increment $d\varepsilon_{kl}$ unchanged, which is consistent with the philosophy of the displacement finite element procedure (Potts and Gens, 1985). Assume a correction index $\delta \lambda_c$ defined as:

$$\delta \varepsilon^p = \lambda_{corr} \frac{\partial f}{\partial \sigma} \quad (4.3)$$

By defining Tensor $M = \frac{\partial f}{\partial \sigma}$, and since the strain increments remain unchanged and noticing $\delta \varepsilon = 0$, the stress correction can be obtained as:

$$\delta \sigma = -\lambda_{corr} C : M \quad (4.4)$$

The hardening parameter correction can be simply obtained from:

$$\delta p_0 = \frac{1 + e}{\lambda - \kappa} p_0 d\varepsilon^p = \lambda_{corr} \frac{1 + e}{\lambda - \kappa} p_0 |trM| \quad (4.5)$$

Substituting (4.4) and (4.5) into (4.2), the expression for the correction index is obtained as:

$$\lambda_{corr} = \frac{f(\sigma, p_0)}{M : C : M - \frac{\partial f}{\partial \varepsilon^p} trM} \quad (4.6)$$
After determination of $\lambda_{corr.}$, the correction of the hardening parameter can be obtained using Equations (4.4) and (4.5), respectively. Furthermore, if convergence could not achieved during the correction scheme mentioned above, the backup normal correction scheme can be used (Sloan et al., 2001). In this simplified scheme, the hardening parameters $p_0$ is assumed to be constant and stresses are corrected only back to the yield surface using the formula:

$$\delta\sigma = -\frac{f(\sigma, p_0): M}{M: M}$$

(4.7)

4.2 Calibration of the Proposed Model

The number of model parameters and their determination are important issues in developing a constitutive model. Usually, an advanced model with few parameters, which are easy to extract, is more applicable in engineering practice than a complicated model. The proposed model has six material constants:

a) There are four parameters related to MCC model: 1) The Poisson ratio $\nu$; 2) the slope of the critical state line $M$; 3) the slope of the normal compression line $\lambda$; and 4) the slope of the unloading-reloading line $\kappa$. All these parameters can obtained directly from the laboratory consolidation test and triaxial test results.

b) There are two parameters in the disturbance function, namely, $A$ and $B$, which can be obtained from triaxial test results and application of Equation (3.11), when the disturbance function, $D$ can be expressed as:

$$D = \frac{q_i - q_a}{q_i - q_c}$$

(4.8)

where $q_i$, $q_c$ and $q_a$ are deviatoric stress for RI, FA and averaged material, respectively.

Rearranging and taking natural logarithms of the disturbance function, Equation (3-13) yields to:
\[
\ln(1 - D) = -A \xi_d^B 
\] (4-9)

Rearranging and taking natural logarithms of Equation (4-9) leads to:

\[
\ln(A) + B\ln(\xi_d) = \ln(-\ln(1 - D)) 
\] (4-10)

Now, we can plot the value for D obtained from Equation (4-8) and, using triaxial test results versus values obtained for \(\xi_d\), obtain a straight line shown in Figure 4.3, which determines A and B.

![Figure 4-3: Determination of disturbance function parameters.](image)

### 4.3 Verification of the Proposed Model

To verify the proposed model, the triaxial compression test was simulated numerically using the Abaqus software, selecting the three-dimensional model with a cubic porous element for soil. The coupled porewater pressure analysis was used, to define the multi-phase characteristic of the saturated soil. Triaxial stress state applied using prescribed stresses for confining stress and using the prescribed displacement for deviatoric stress. The sample top surface was assumed free for
drainage. Both drained and undrained responses were modeled, and drainage condition was controlled by the value specified for soil permeability. The model was first run using the Abaqus built-in MCC model, and the obtained results are presented in Figure 4.4. Then same model was run using the proposed model through implementation via UMAT, and the obtained results were compared with the results of the MCC model as shown in Figure 4.5. As can be seen in Figure 4.4, the MCC model prediction for OC soil is not realistic, especially for the heavily OC soil since it shows mostly elastic response during undrained shearing. On the other hand, the proposed model provides a complete elastoplastic response with smooth transition from elastic to plastic responses even for the heavily OC soils.

Figure 4-4: Stress path in triaxial compression obtained from numerical simulation using MCC model.
4.3.1 Case Study 1: Kaolin Clay

To verify the predictive capability of the proposed model, experimental data on Kaolin Clay from triaxial tests performed by other researchers (e.g., Yao et al., 2012) has been used. The shear responses from underained triaxial compression tests for different stress histories (OCR=1, 1.20, 5, 8, 12) have been simulated. Four model parameters which are related to the MCC model were obtained from Dafalias and Herrmann (1986). Two remaining parameters which are related to the disturbed state concept (i.e. $A$ and $B$) were obtained from the triaxial test results and the method explained in Section 4.2. The calculated parameters are presented in Table 4-1. As shown in Figure 4.6, the obtained values for parameters $A$ and $B$ control the disturbance pattern in the soil body due to induced plastic strain during applied shear loads.
Table 4-1: Model parameters for Kaolin Clay used for implementation.

<table>
<thead>
<tr>
<th></th>
<th>M</th>
<th>λ</th>
<th>κ</th>
<th>ν</th>
<th>A</th>
<th>B</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1.04</td>
<td>0.14</td>
<td>0.05</td>
<td>0.20</td>
<td>14.43</td>
<td>0.47</td>
</tr>
</tbody>
</table>

Using the model parameter presented in Table 4-1, the FE model was run with the MCC model and results for stress path in the undrained condition are presented in Figure 4.7, which shows that the MCC model is not able to capture both NC and OC clay soil responses under undrained shearing. In the proposed model, the strong capability of the DSC to model the real material behavior is used, and the numerical simulation results for stress path using the proposed model are presented in Figure 4.8. Based on these results, we can conclude that the proposed model predicts the real soil behavior for both NC and OC soils with good agreement when compare its prediction...
with experimental test results. It can capture the strain softening behavior in heavily OC soils, and Figure 4.9 shows the proposed model results for undrained stress-strain relation at different over-consolidation ratios, which represents satisfactory agreement. In this figure, values for stress were normalized based on the initial pre-consolidation pressure \( p_0 \). Figure 4.10 represents the numerical simulation for porewater pressure generated during undrained triaxial test using the proposed model. Figure shows that for NC soil and lightly OC soil the generated porewater pressure is positive, which indicates the soil contraction during undrained shear. On the other hand, for heavily OC soils, the numerical simulation shows generation of the positive porewater pressure at the initial stage of the test, followed by negative porewater pressure until failure. This clearly indicates the soil dilative behavior which is common in heavily OC soils. Based on the obtained results, soil dilation in the undrained condition increases with increasing OCR values.

Figure 4-7: Numerical simulation of undrained triaxial test on Kaolin Clay using MCC model.
Figure 4-8: Numerical simulation of undrained triaxial test on Kaolin Clay using the proposed model.

Figure 4-9: Stress-strain relation for undrained triaxial test on Kaolin Clay using the proposed model.
Figure 4.10: Simulation of the porewater pressure generated in undrained triaxial test on Kaolin Clay using the proposed model.

4.3.2 Case Study 2: Boston Blue Clay

The results of undrained triaxial tests on normally consolidated Boston Blue Clay which are available in the literature (Ling et al. 2002) were also used to verify the proposed CSDSC model predictions. Table 4.2 presents the model parameters for the Boston Blue Clay. Figure 4.11 shows variation of the soil disturbance as a function of plastic strain, using the specified $A$ and $B$ values. Figures 4.12, 4.13 and 4.14 present the results obtained from the proposed model and those measured using triaxial tests. These figures demonstrate very good agreement between the model predictions and the test results for hydrostatic and deviatoric stress paths, and the stress-strain curve and excess porewater pressure generated during applied shear load, respectively.
Table 4.2: Model parameters for Boston Blue Clay.

<table>
<thead>
<tr>
<th>M</th>
<th>M</th>
<th>( \lambda )</th>
<th>( \kappa )</th>
<th>( \nu )</th>
<th>A</th>
<th>B</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.04</td>
<td>0.14</td>
<td>0.05</td>
<td>0.20</td>
<td>4.70</td>
<td>0.35</td>
<td></td>
</tr>
</tbody>
</table>

Figure 4.11: Evolution of disturbance in the soil body as function of plastic strain for Boston Blue Clay.

4.3.3 Case Study 3: Bangkok Clay

The results available in the literature (Likitlersuang, 2003) of undrained triaxial test on Bangkok Clay at different load histories (OCR=1 and 1.24), were used to verify the proposed CSDSC model. Table 4.3 presents the model parameters for this soil. Figure 4.15 shows variation of soil disturbance as a function of plastic strain using the specified \( A \) and \( B \) values. The results obtained from the numerical simulation using CSDSC model, including the stress paths and stress-strain relations, are compared with the laboratory test results as shown in Figures 4.16 and 4.17 for OCR=1 and OCR=1.24, respectively. There is good agreement between the prediction results using the proposed model and the laboratory triaxial test results.
Figure 4-12: Comparison of the stress paths b/w traixial test result and the proposed model prediction for Boston Blue Clay.

Figure 4-13: Comparison of the stress-strain relation b/w traixial test result and the proposed model prediction for Boston Blue Clay.
Figure 4.14: Comparison of the generated excess porewater pressure b/w traixial test result and the proposed model prediction for Boston Blue Clay.

Table 4.3: Model parameters for Bangkok Clay.

<table>
<thead>
<tr>
<th>M</th>
<th>λ</th>
<th>κ</th>
<th>ν</th>
<th>A</th>
<th>B</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.90</td>
<td>0.34</td>
<td>0.045</td>
<td>0.20</td>
<td>11.94</td>
<td>0.52</td>
</tr>
</tbody>
</table>

Figure 4-15: Evolution of disturbance in the soil body as function of plastic strain for Bangkok Clay.
Figure 4.16: Comparison of stress paths between the proposed model prediction and test result for Bangkok Clay.

Figure 4.17: Comparison of stress-strain curves between the proposed model prediction and test result for Bangkok Clay.
5 NUMERICAL SIMULATION OF CASE STUDIES

Numerical simulation of pile installation and following setup at different times from EOD is an objective of this study. In this research, five pile-driving sites were selected to study including: 1- Bayou Lacassine Bridge site; 2- Sabin River Case Study; 3- Bayou Zouri Site; 4-Bayou Bouef Site; and 5- Baton Rouge Cajun site.

5.1 Case Study 1: Bayou Lacassine Site

The Louisiana Department of Transportation and Development (LADOTD) replaced the old Bayou Lacassine Bridge, located at Highway 14 in Jefferson Davis Parish, Louisiana, with a new 585 m long span supported by 152 square pre-stressed concrete (PSC) piles. Three precast, pre-stressed concrete (PPC) test piles (TP1, TP2, and TP3) were driven at Bayou Lacassine Bridge site using a single acting diesel hammer. Figure 5.1 shows the layout of this site, depicting the old and new bridges and the test piles location. All the test piles were square PPC piles with 0.76 m width. The total length of TP1, TP2, and TP3 were 22.87 m, 25 m, and 22.87 m, respectively. A 1.14 m diameter and 6.4 m length casings were installed prior to pile installation to represent the scour effect at shallow depth. All test piles were monitored with Pile Driving Analyzer (PDA), followed by Case Pile Wave Analysis Program (CAPWAP) analysis to obtain the pile driving criteria. TP1 and TP3 were fully instrumented, while test pile TP2 was driven in the waterway and could not be instrumented. Figures 5.2,a and 5.2,b show the instrumentation layout of test piles TP1 and TP3. Four sets of pressure cells and vibrating wire piezometers were installed flush to the face of the piles at different depths that target specific soil layers.

“This chapter previously appeared as Abu-Farsakh M., Rosti F., and Souri A., Evaluating pile installation and subsequent thixotropic and consolidation effects on setup by numerical simulation for full-scale pile load tests., 2015. It is reprinted by permission of NRC Research Press.”
In addition, surrounding soils were instrumented with nine multi-level piezometers located at the same depths as the pressure cells and piezometers installed at the piles’ faces. These instruments were used to measure the total stress, effective stress, and developed excess porewater pressure during pile installation and its dissipation with time after EOD. Each of the two test piles was instrumented with eight pairs of strain gages in order to evaluate the load transfer distribution along the pile length. An extensive load test program was carried out after pile installation, using both static load tests (SLTs) and dynamic load tests (DLTs). The test program began with a DLT conducted within 1 hour after EOD, followed by five SLTs conducted at different times after EOD. One additional DLT was conducted on the piles after the completion of SLTs.

Figure 5-1: Bayou Lacassine Bridge site layout (from Haque et al. 2014).
5.1.1 Soil Properties and Model Parameters

Soil properties were obtained via laboratory testing of the specimens obtained from two boreholes drilled on site, near the test piles TP1 and TP3, and from three cone penetration tests (CPT) conducted on the site near the test piles. Model parameters were obtained using the consolidation test results and adopting appropriate correlation equations exist in literature. As there is no direct test result for TP2, model parameters required in the numerical simulation were estimated, based on the information obtained for the other two test piles. Soil layering, soil properties and model parameters are presented in Tables 5-1, 5-2, and 5-3. Soil layering was performed based on CPT profiles, and 5, 8, and 8 layers were selected for TP1, TP2, and TP3, respectively.
Table 5.1: Soil material parameters for the test pile 1 (TP1) site.

<table>
<thead>
<tr>
<th>Layer No.</th>
<th>Depth (m)</th>
<th>w (%)</th>
<th>$e_0$</th>
<th>$k_0$</th>
<th>$S_u$ (kPa)</th>
<th>OCR</th>
<th>M</th>
<th>$\lambda$</th>
<th>$\kappa$</th>
<th>K (m/s) 10^{-9}</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0-6</td>
<td>28</td>
<td>0.74</td>
<td>0.60</td>
<td>18</td>
<td>2.50</td>
<td>0.95</td>
<td>0.078</td>
<td>0.013</td>
<td>8.15</td>
</tr>
<tr>
<td>2</td>
<td>6-9</td>
<td>22</td>
<td>0.57</td>
<td>0.60</td>
<td>18</td>
<td>2.46</td>
<td>1.00</td>
<td>0.078</td>
<td>0.013</td>
<td>6.09</td>
</tr>
<tr>
<td>3</td>
<td>9-11</td>
<td>24</td>
<td>0.65</td>
<td>0.60</td>
<td>56</td>
<td>2.30</td>
<td>1.10</td>
<td>0.078</td>
<td>0.013</td>
<td>16.50</td>
</tr>
<tr>
<td>4</td>
<td>11-14</td>
<td>22</td>
<td>0.60</td>
<td>0.70</td>
<td>49</td>
<td>1.40</td>
<td>0.90</td>
<td>0.056</td>
<td>0.019</td>
<td>5.86</td>
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<tr>
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<td>14-21</td>
<td>34</td>
<td>1.00</td>
<td>0.70</td>
<td>66</td>
<td>1.00</td>
<td>0.75</td>
<td>0.093</td>
<td>0.014</td>
<td>1.54</td>
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Table 5.2: Soil material parameters for the test pile 2 (TP2) site.

<table>
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<tr>
<th>Layer No.</th>
<th>Depth (m)</th>
<th>w (%)</th>
<th>$e_0$</th>
<th>$k_0$</th>
<th>$S_u$ (kPa)</th>
<th>OCR</th>
<th>M</th>
<th>$\lambda$</th>
<th>$\kappa$</th>
<th>K (m/s) 10^{-9}</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0-5.50</td>
<td>37</td>
<td>1.20</td>
<td>1.40</td>
<td>130</td>
<td>4</td>
<td>1.16</td>
<td>0.08</td>
<td>0.03</td>
<td>4.36</td>
</tr>
<tr>
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<td>5.50-7.60</td>
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<td>0.70</td>
<td>0.70</td>
<td>85</td>
<td>3</td>
<td>1.23</td>
<td>0.08</td>
<td>0.03</td>
<td>4.36</td>
</tr>
<tr>
<td>3</td>
<td>7.60-10.40</td>
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<td>0.05</td>
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<td>0.11</td>
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<td>0.13</td>
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<td>22</td>
<td>0.60</td>
<td>0.60</td>
<td>150</td>
<td>1</td>
<td>0.93</td>
<td>0.12</td>
<td>0.04</td>
<td>1.05</td>
</tr>
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</table>

Table 5.3: Soil material parameters for the test pile 3 (TP3) site.

<table>
<thead>
<tr>
<th>Layer No.</th>
<th>Depth (m)</th>
<th>w (%)</th>
<th>$e_0$</th>
<th>$k_0$</th>
<th>$S_u$ (kPa)</th>
<th>OCR</th>
<th>M</th>
<th>$\lambda$</th>
<th>$\kappa$</th>
<th>K (m/s) 10^{-9}</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0-6.40</td>
<td>21</td>
<td>0.50</td>
<td>1.20</td>
<td>120</td>
<td>4</td>
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<td>0.104</td>
<td>0.035</td>
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<td>72</td>
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<td>0.100</td>
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</tr>
<tr>
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<td>7.60-10</td>
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<td>0.091</td>
<td>0.026</td>
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<tr>
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<td>0.75</td>
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<td>0.90</td>
<td>0.108</td>
<td>0.035</td>
<td>0.12</td>
</tr>
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<td>0.80</td>
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<td>0.62</td>
<td>0.108</td>
<td>0.035</td>
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<td>0.80</td>
<td>150</td>
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<td>0.147</td>
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<td>1</td>
<td>0.93</td>
<td>0.056</td>
<td>0.013</td>
<td>0.66</td>
</tr>
</tbody>
</table>
5.1.2 Numerical Model

Three test piles (TP1, TP2, and TP3) driven at the Bayou Lacassine Bridge site were modeled in this study. The geometry of soil and pile and the applied boundary conditions for a typical pile are shown in Figure 5.3,a; while the finite element mesh is presented in Figure 5.3,b. Finer mesh was used for the soil elements adjacent to the pile surface. Mesh sensitivity analysis was performed to determine the soil element type and size. Figures 5.4 and 5.5 demonstrate two curves regarding the numerical simulation prediction for pile shaft resistance using different element size and type. Both linear and quadratic elements were used for soil domain at the pile interface zone, because the objective was to identify linear sets of element, which can be used to predict the pile installation and setup behavior. The linear quadrilateral coupled porewater element was used for the whole soil domain to avoid shear locking and to provide more accurate results than other elements (Shao, 1998; Walker and Yu, 2006). Two sizes of elements were selected based on the literature (Sheng et al., 2011) and the ratio of the finest element width (W) to the pile radius (R) were selected to be 0.25 and 0.125. As the Figures 5.4 and 5.5 indicate, the FE mesh using quadratic elements with the ratio of W/R=0.25 and W/R=0.125 show very close prediction, which indicates that these W/R=0.25 is best size to use in FE analysis. The equivalent mesh using linear elements with W/R=0.125 reached same prediction as the optimum quadratic mesh. Therefore, the linear elements with W/R=0.125 was selected in FE analysis to reduce the possible numerical simulation problems. A curved shape was adopted for the pile tip to minimize the effect of sharp corner problem during penetration in the numerical simulation.

In this study, the surface to surface master-slave contact model was used to simulate the pile-soil interface. The contact between the two surfaces is controlled by kinematic constraints in the normal and tangential directions. When the pile is in contact with the soil, the normal stress at
contact is compressive, and it is zero when there is a gap between the pile and the soil. The classical isotropic Coulomb frictional contact law was used to model the frictional sliding at the pile-soil interface. The friction coefficient, \( \mu \), was related to the soil internal friction angle, \( \varphi \), using relation

\[
\mu = \frac{2}{3} \tan(\varphi).
\]

Figure 5.3: Numerical simulation domain: (a) geometry and boundary conditions and (b) FE mesh.
Figure 5-4: Mesh sensitivity analysis: prediction of pile shaft resistance immediately after end of pile installation for different mesh sizes at the pile-soil interface zone (R: pile radius, W: finest soil element width).
Figure 5.5: Mesh sensitivity analysis: prediction of pile shaft resistance for 217 days after end of pile installation for different mesh sizes at the pile-soil interface zone (R: pile radius, W: finest soil element width).
5.1.3 Results and Analysis

During pile driving, the stress states in the surrounding soil change. Figure 5.6 presents the change in hydrostatic stress and deviatoric stress in the soil due to the pile installation effect. The figure shows that the deviatoric stress increases significantly in the soil adjacent to the pile, especially near the pile tip; however, the hydrostatic stress does not change significantly in comparison to deviatoric stress during pile installation. Pile installation in saturated clayey soils is usually associated with the development of excess porewater pressure, which dissipates with time after EOD. The normalized change in excess porewater pressure with time after EOD obtained from field test results for TP1 and the corresponding numerical simulation values are presented in Figures 5.7,a and 5.7,b for two depths 12.20 m and 16.47 m, respectively, which correspond to soil layers three and four. Similar results obtained for TP3 at depths 8.54 m and 18.30 m are shown in Figure 5.8. The presented depths in these figures correspond to the instrumented depths for the full-scale field test piles. Figures 5.7 and 5.8 demonstrate good agreement between the field measurements and the results of FE numerical simulation. The spikes seen in Figures 5.7 and 5.8 represent the generated excess porewater pressure during the installation of reaction frame, the field static load tests, and the dynamic load test restrikes. Figure 5.9,a presents the soil displacement obtained in the radial direction after the second phase of installation. The pile installation effect in the surrounding soil beyond the radial distance (d) greater than 10 times the equivalent pile diameter (D) is almost negligible, which is supported by the results reported in the literature (e.g., Randolph, 1979). In this study, the influence zone for soil disturbance is assumed to be equal to the radial distance that corresponds to soil displacement equal to 5% of the pile diameter. The influence zone obtained from Figure 5.8,a is equal to 4D, and this value was used in the FE numerical simulation. Figure 5.9,b presents the displacement field around the pile tip
during installation. This figure shows that the displacement of soil in the pile vicinity occurs in both vertical and horizontal directions and decreases as distance increases from the pile.

Figure 5.6: Stress change in soil during pile driving: a) hydrostatic stress (kPa) and b) deviatoric stress (kPa).

Figure 5-7: Comparison of numerical simulation and measured excess porewater pressure dissipation with time after EOD for TP1 obtained at different depths: (a) at Z=12.20 m. and (b) at Z=16.47 m.
Figure 5.8: Comparison of numerical simulation and measured excess porewater pressure dissipation with time after EOD for TP3 obtained at different depths: (a) at $Z=8.54$ m. and (b) at $Z=18.30$ m.

Figure 5.9: Displacement in soil body during pile installation for TP1: (a) horizontal displacement, (b) general displacement field.
The shear stress in the soil body changes during the entire process of numerical simulation. Figure 5.10 shows variation of the shear stress in each step of pile installation and the following setup for a typical horizontal path (e.g., path 1) in the soil body. The figure depicts that no shear was developed in geostatic; the cavity expansion and consolidation steps develop low values of shear stress in the soil body; and that the initial shear and load test steps develop most of the shear stress in the soil body.

Figure 5-10: Changes in shear stress in soil body during different steps of pile installation and setup.
Figure 5.11 shows that increases in pile resistance with time after pile installation, obtained from both the field tests and predicted by the FE numerical simulation for TP1. The results demonstrate that the readers can observe that setup is predominant in pile shaft resistance rather than in tip resistance. In Figure 5.12, pile tip resistance immediately after EOD is compared with its value at 217 days after EOD (i.e.: the time at which the soil excess porewater pressure is fully dissipated). Variation of pile tip resistance with the pile penetration was compared, and Figure 5.12 indicates negligible difference in tip resistance over time after EOD. The model was first run using undisturbed soil properties, and the pile capacities at different times after EOD are depicted in Figure 5.11 (dashed line with triangles). The figure also shows that the predicted shaft resistance using the undisturbed soil properties is overestimated at early stages, which then reaches the field measured values after 120 days from EOD. To clarify this problem, the model was run using the remolded soil properties with $\beta(0)=0.75$, and the pile capacities at different time after EOD were obtained (dashed line with stars). This curve shows that results are in good agreement with field results only at the early stage after EOD, but then deviate from the field results. This observation can be explained by the soil disturbance occurs in the vicinity of the pile-soil interface during pile installation, followed by the effect of thixotropic behavior of the soil in regaining its strength with time after EOD. Based on this observation, the thixotropic behavior was incorporated into setup simulation (in combination with consolidation effect) using the time-dependent parameter $\beta(t)$ (equations 3-2 and 3-3). The model run under evolution of $\beta(t)$ for different times from EOD, and the obtained results of setup with time for TP1, are also presented in Figure 5.11 (solid line with circles). Figure 5.11 clearly shows that the results obtained from the numerical simulation, using thixotropic effects along with consolidation, are in good agreement with the results obtained from field measurements.
Similar results with the same observation can be seen in Figures 5.13 and 5.14 for TP2 and TP3, respectively. The results of numerical simulation demonstrate that the use of parameters related to the undisturbed soil cannot capture the actual setup behavior with time, especially at the early stages of setup after EOD. Therefore, the consideration of disturbance effects during pile driving and its evolution with time after EOD (thixotropy) is necessary to simulate actual behavior of pile setup in cohesive soils, which is demonstrated in Figures 5.11, 5.13, and 5.14.

Figure 5.11: Increase in pile capacity with time after EOD for TP1.

Figure 5.15 presents the load-settlement curves for static load test of TP1, obtained at 13 and 208 days after EOD. Similar results were obtained for the other two test piles. The figure compares the field results from the static load test with the prediction values from the FE numerical simulation including the consolidation and thixotropic effects, and demonstrates satisfactory agreement between the field and numerical results. The results of field static load tests show that TP1 was failed at 2354 kN and 2845 kN loads for load tests performed at 13 and 208 days after
EOD, respectively, while the FE results show 2109 kN and 2790 kN ultimate capacities for load tests performed at the same time intervals.

Figure 5-12: Tip resistance variation over time after EOD.

Figure 5-13: Increase in pile capacity with time after EOD for TP2.
Since TP2 was not instrumented in the field, only the results of TP1 and TP3 will be presented. Figure 5.16,a compares the profiles of pile shaft resistance for TP1 with depths obtained from numerical simulation and field test measurements (from strain gauges) at two different times: shortly after pile installation (i.e., t=0.01 day) and a long time after EOD (i.e., t=208 days). The figure demonstrates good agreement between the FE prediction and the field measured shaft resistance profiles. The variation of the shaft resistance with depth for TP3 is presented in Figure 5.16,b, which also shows good agreement between the result from FE numerical simulation and the field test results. No shaft resistance was observed on the top 6 m for both test piles due to casing. Figures 5.16,a and 5.16,b demonstrate that shaft resistance increases in all soil layers due to setup phenomena.
The ratio of the side pile resistance at time (t) after installation ($R_t$) to the side resistance determined immediately after pile installation ($R_0$), known as the setup ratio, was calculated for different soil layers along the piles. The variations of setup ratio with depth for TP1 obtained from the numerical simulation and from those calculated from the field test measurements at 208 days after EOD are presented in Figure 5.17,a. The variation of setup ratios with depth for TP3, obtained 175 days after EOD, is shown in Figure 5.17,b. Contribution of soil thixotropy has been included in the analyses in Figures 5.17,a and 5.17,b.

Figures 5.16 and 5.17 clearly support the conclusion that considering the soil disturbance and thixotropy effects in combination with the consolidation setup effect, provides good agreement between the field measurements and the numerical simulation of setup, especially a long time after EOD. The results of numerical simulation for shaft resistance at $t=0.01$ day (Figure 13) also demonstrate that considering soil disturbance a short time after EOD provides better agreement between numerical simulation and field test measurements.

Figure 5.15: Comparison between the measured load-settlement curves for TP1 with numerical simulation obtained at 13 and 208 days after EOD.
Figure 5.16: Variation of the pile shaft resistance with depth obtained at two times after EOD for (a) TP1, and (b) TP3.

The percentage increases in the effective horizontal stress obtained from the FE numerical simulation and those calculated from the measured field test data (using pressure cells and piezometers), along with corresponding excess porewater pressure dissipation, are compared in Figures 5.18 and 5.19 for TP1 and TP3, respectively. The results of changes in effective stress with time during the setup process exhibit satisfactory agreement between the numerical simulation and the field test results. Figures 5.18 and 5.19 demonstrate that with time from EOD, the induced excess porewater pressure dissipates, and the effective stress increases, until they reach constant values after setup is almost completed.
Figure 5.17: Comparative setup ratio at different depth: (a) for TP1 calculated at time $t=208$ days (b) for TP3 calculated at time $t=175$ days.

Figure 5.18: Comparative results for horizontal effective stress and excess porewater pressure analysis for TP1 at depth $Z=16.47$ m.
Figure 5-19: Comparative results for horizontal effective stress and excess porewater pressure analysis for TP3 at depth $Z=8.54$ m.

5.1.4 Results for Disturbance Function and Strength Reduction Parameter

During pile installation, soil disturbance is most predominant, especially at the pile-soil interface area. This section presents the results of obtained amount of soil disturbance. Figure 5.20 represents the soil disturbance occurs immediately after pile installation for a typical horizontal path (path 1 in Figure 5.3), obtained from numerical simulation using the CSDSC model. The figure shows that $\beta$ has its maximum value $\beta_R = 0.75$ for soil adjacent to the pile face and approaches unity at a radial distance equal to 8 times the pile size. At the same time, the disturbance function has a maximum value ($D=1$) at the soil-pile interface, and it approaches to $D=0$ at a radial distance equal to 8 times the pile size along the same path.
Figure 5.20: Variation of $\beta$ and $D$ for a typical horizontal path in soil body immediately after pile installation.

Figure 5.21 shows variation of the initial soil strength reduction $\beta_0$ and disturbance function $D$ immediately after pile installation. SDV is refers to the state dependent variables, which were defined in the user-defined subroutine (UMAT) and were updated at the end of each increment. This figure indicates that soil has maximum disturbance and remolding at the pile interface and reaches its un-remolded condition while it approaches the far right boundary. The numerical simulation using CSDSC model was compared with predictions of the other soil constitutive models such as built-in Abaqus MCC model and AMCC model. Figure 5.22,a is a comparison between the predictions of these models for unit shaft resistance immediately after end of driving. The cumulative values of shaft resistance obtained from numerical simulation using the models were compared with the calculated values obtained from field tests, and the results are shown in Figure 5.22,b. These figures indicate that the CSDSC model is able to predict the pile resistance appropriately.
Figure 5.21: Variation of $\beta_0$ (SDV8) and $D$ (SDV9) in soil body immediately after pile installation.

Figure 5.22: Comparison between the proposed CSDSC model prediction with MCC model and AMCC models (a) unit shaft resistance, and (b) cumulative shaft resistance.
5.2 Case Study 2: Bayou Zouri and Bayou Bouef Sites

Two driven piles at two different sites (Bayou Zouri Bridge and Bayou Bouef Bridge) were simulated using the proposed technique. Both test piles were fully instrumented. The piles had square cross sections; however, an equivalent circular shape was adopted to facilitate the FE modeling of the cavity expansion. The FE software Abaqus was used for numerical modeling. The geometry of the soil and the pile and the applied boundary conditions for the Bayou Zouri Bridge site and the corresponding soil layering are shown in Figure 5.23,a. The information for the Bayou Bouef Bridge site are presented in Figure 5.23,b. Curved shape was adopted for the pile tip to minimize the effect of sharp corner in the numerical solution. Figure 5.24 shows the typical finite element mesh and the different phases used to model the pile installation and following pile setup.

![Figure 5-23: Numerical simulation domain geometry and boundary conditions: (a) Bayou Zouri Bridge site (b) Bayou Bouef Bridge site.](image-url)
Figure 5.24: Changes in porewater pressure during various steps of pile installation: (a) cavity expansion, (b) pile placement, (c) initial vertical penetration, (d) consolidation and (e) final vertical penetration.

5.2.1 Bayou Zouri Bridge Site Description

The construction project consists in building a two-lane highway bridge on the northbound lane of U.S. 171 over Bayou Zouri in Vernon Parish, Louisiana. The existing bridge required replacement due to substandard load carrying capacity and the embankment protection is severely undetermined. The plan view of the site is schematically illustrated in Figure 5.25. Prestressed square precast concrete (PPC) pile foundation having a width of 61 cm were selected to support the bridge structure. One pile with a 16.8 m embedded length was selected to perform two static load tests (SLT) and four dynamic load tests (DLT) to study the setup magnitude over 77 days from end of driving (EOD).
The ground water level was about one meter below ground surface. The subsurface soil was characterized using in-situ Standards Penetration Test (SPT), Cone Penetration Test (CPT), and Piezocone Penetration Test (PCPT). Laboratory soil tests such as triaxial test and consolidation test were also performed by the research team on undisturbed soil samples. The PCPT data were used to classify subsurface soil for Bayou Zouri Bridge site. The subsurface soil consists mainly of stiff clay with some loose sandy soil interlayers in the top 10 m below ground surface. The estimated undrained shear strength of the clayey layers varies from 150 to 490 kPa. Site characterization was described in Chen et al. (2014) in detail. Standard Penetration Tests (SPT), Cone Penetration Tests (CPT), Piezocone Penetration Tests (PCPT), as well as laboratory soil tests such as triaxial and consolidation tests, were used for site characterization. A summary of soil characterization can be seen in Figure 5.26. The liquid limit ($LL$), plasticity index ($PI$), particle size distribution, undrained shear strength ($S_u$), SPT $N$-values, and vertical coefficient of consolidation are also shown in Figure 5.26. Figure 5.27 shows the cone tip resistance, friction ratio, Porewater pressure, and soil classification which was obtained based on the PCPT data.
Figure 5.26: Soil properties and soil classification in Bayou Zouri Bridge site (Chen et al. 2014).

Figure 5.27: Soil profile and classification depicted from PCPT results (Chen et al. 2014).
5.2.2 Bayou Bouef Bridge Site Description

The long-term pile capacity study was conducted during the construction of Bayou Bouef Bridge extension on relocated U.S. 90, east of Morgan City, Louisiana. This project consisted of constructing approximately 3.54 km of bridge structure over swampy terrain. The site conditions required the contractor to build a temporary haul road to gain access to the project site. Four 76.2 cm square PPC piles per bent were typically used to support the elevated structure. The pile lengths ranged from 38.1 m to 45.7 m. The project required that three test piles be driven and load tested. The long-term pile capacity study, which included pile setup capacity, was conducted next to Test Pile No. 3 of this project, between pile bents 210 and 211. The tested pile had a 43.6 m length, and it was driven 40.1 m beneath the subsurface soil. The subsurface conditions were characterized during the pre-design phase of the project by performing in-situ CPT tests and laboratory tests on soil samples obtained from soil boring. The subsurface soil at Bayou Bouef Bridge site consists of normally consolidated soft clayey soil to an approximate depth of -38 m, underlying by medium sand to maximum depth of soil boring. Figure 5.28 shows the PCPT test results and the soil properties obtained using the PCPT data. The Osterberg Cell was used to perform the static load tests at different times after pile installation, as long as 2 years after EOD. The ground water level was about 0.80 m below ground surface. The estimated undrained shear strength of the clayey layers varies from 20 to 90 kPa.

5.2.3 Numerical Model

Pile installation was modeled by the combination of volumetric cavity expansion, followed by applying vertical shear displacement (penetration) in an axisymmetric FE model. The theory of consolidation followed by shearing at the pile-soil interface was used to model the pile setup phenomenon. In this model, a series of prescribed displacements in the soil’s axisymmetric
boundary were first applied in order to create a displaced volume in the soil equal to the size of the pile (volumetric cavity expansion).

![Soil properties and soil classification in Bayou Bouef Bridge site.](image)

Figure 5-28: Soil properties and soil classification in Bayou Bouef Bridge site.

The pile was then placed inside the cavity, and the interaction between the pile and soil surrounding soil was activated. The prescribed boundary conditions to create cavity expansion were released, and an additional vertical penetration was applied (initial shear step). This step provides porewater pressure generation around the pile tip, which was not mobilized appropriately during the previous step. Figures 5.29,a and 5.29,b represent the porewater pressure distribution around the Bayou Zouri Bridge pile tip before and after the initial shear step, respectively. These figures show that the porewater pressure values beneath the pile tip increased from 50 kPa before the initial shear step to 800 kPa after this step.
Figure 5-29: Porewater pressure mobilization during initial shear step beneath Bayou Zouri site pile tip: (a) before initial shear, (b) after initial shear.

The developed excess porewater pressure during the installation was allowed to dissipate for different elapsed times after installation to simulate static load tests at different times. The static load test was then simulated by applying an additional penetration to the pile and hence additional vertical shear displacement at the pile-soil interface, until failure (final shear step).

In this study, the previously introduced time-dependent strength reduction parameter $\beta(t)$ was first applied to the cohesive soil strength parameter $M$, as well as the pile-soil interface friction coefficient $\mu$ to incorporate the effect of soil remolding during pile installation:

\[
\begin{align*}
M(t) &= \beta(t)M \\
\mu(t) &= \beta(t)\mu
\end{align*}
\]  

(5)

An evolution function, presented in Equation (3.3), was then introduced to capture the increase in strength over time for the remolded soil around the pile. As discussed earlier, in
Equation (3.3), the term $\beta(0)$ is usually related to the soil sensitivity and $\beta(\infty)$ is the $\beta$ value a long time after soil disturbance. Information regarding the soil sensitivity for these test sites were not available; however, based on the study that was performed on another test site in a similar soil in Louisiana, values of $\beta(0) = 0.75$ were reasonably adopted for both sites. This value for $\beta(0)$ is obtained from $\beta(0) = (S_r)^{-0.3}$, adopting a sensitivity value equal to 3. A detailed description regarding the thixotropy formulation in pile installation and setup is available in Abu-Farsakh et al. (2015). For naturally non-structured soils with low sensitivity, long-term strength regaining during thixotropic behavior might be equal to the undisturbed strength values. On the other hand, $\beta(\infty)$ can be 1 for low sensitive clay (as adopted here) and it can reach a value greater than 1 for soils artificially structured with cement slurry or salt after remolding. In Equation 6, $\tau$ is a time constant and it can be defined in relation to $t_{90}$, which is the time for 90% dissipation of the excess porewater pressure at the pile surface. Values for $t_{90}$ were derived from PCPT dissipation curves. More investigation is required to find the real value for $\tau$; however, here it was simply assumed that $\tau = t_{90}$.

5.2.4 Results and Analysis

Figure 5.30 shows variations of the initial soil strength reduction $\beta_0$ and disturbance function $D$ immediately after Bayou Bouef Bridge pile installation. This figure indicates that soil has maximum disturbance and remolding at the pile interface and reaches its un-remolded condition while it approaches the far right boundary. In Figure 5.30, “SDV” refers to the state dependent variables, which were defined in user-defined subroutine (UMAT) and were updated at the end of each increment.
Figure 5.30: Variation of $\beta_0$ (SDV8) and $D$ (SDV9) the soil body immediately after pile installation in Bayou Bouef Bridge site.

The increase in pile shaft resistance with time after EOD, obtained from the field load tests and predicted from numerical simulation, (solid line) are presented in Figure 5.31. The field results for the two sites were obtained from both the SLT and DLT results. Figure 5.31 shows that the predicted shaft resistances (solid line) are overestimated for a short period of time, but then attain the field measured values after a long time (i.e., after 100 days for the Bayou Zouri test pile). This observation can be explained by the disturbance that occurs at the pile-soil interface during pile installation and the effect of thixotropic behavior of the soil in regaining its strength with time. For accurate prediction, numerical simulation was performed using reduced properties for remolded soil immediately after EOD, and then adjusting properties to capture the soil thixotropic response with time after EOD. The soil remolding during pile installation, and the subsequent strength regaining due to the soil thixotropic response, were applied using a time-dependent reduction factor and its evolution with time using an exponential function as described in previous sections.
The predicted results by including the soil thixotropic response are shown in Figure 5.31 by a dashed line. As one can see, the predicted response obtained by considering soil disturbance during pile installation and thixotropic behavior demonstrated better agreement with the measured results from field tests. Figure 5.31 demonstrates lower setup ratio for the Bayou Zouri Bridge site compared to the Bayou Bouef Bridge site. This is may be due to higher stiffness of clayey soil, presence of sandy layers, and high over-consolidation ratios for the subsurface soil of bayou Zouri Bridge site as compared to the subsurface soil condition at Bayou Bouef Bridge site.

Changes in the porewater pressure in the surrounding soil domain is one of the main results of pile installation in saturated clay soils. Pile installation usually results in the development of excess porewater pressure, which dissipates with time after EOD. The change in excess porewater pressure with time after EOD for the Bayou Zouri site, obtained from field test measurements through the piezometers installed on the pile face, and the corresponding numerical simulation values, are presented in Figures 5.32,a and 5.32,b for the two depths 7.60 m and 10.70 m, respectively, corresponding to soil layers three and five. The figure shows satisfactory agreement between the field measurements and results of numerical simulation. The generated porewater pressure and its dissipation with time obtained from numerical simulation for Bayou Bouef Bridge site at two different depths are shown in Figure 5.33. Because the field measurement data was not available, in Figure 5.33 no result has been shown for field results.

The load transfer distribution along the Bayou Zouri Bridge pile during SLT for selected loads, and their corresponding values obtained from numerical simulation, are presented in Figure 5.34. The figure shows that the results obtained from the FE numerical model are able to appropriately predict the load distribution along the pile, especially for load distribution at the pile shaft.
Figure 5.31: Increase in pile shaft resistance with time after EOD: (a) Bayou Zouri bridge site, and (b) Bayou Bouef bridge site.

Figure 5.32: Comparison between numerical and measured excess porewater pressure dissipation with time after EOD for Bayou Zouri Bridge site obtained at different depths: (a) at $Z=7.60$ m, and (b) at $Z=10.70$ m.
Figure 5-33: Excess porewater pressure dissipation with time after EOD for Bayou Bouef Bridge site obtained from numerical simulation for two different depths (Z) below the ground surface.

Figure 5-34: Comparison between the load transfer distributions along Bayou Zouri Bridge pile obtained from numerical simulation and calculated from SLT for selected loads.
5.3 **Case Study 3: Sabin River Site**

Small diameter instrumented steel pile segments (x-probe) were driven in Sabin River Clay by the Earth Technology Corporation in 1986 to study soil setup behavior. The Sabin River site location shown in Figure 5.35 consists of highly plastic fat clay with properties described in Table 5-4. The Sabin Clay properties were first obtained from several extensive laboratory tests conducted on undisturbed soil samples by Katti (1990). Two x-probes with 1.72 inch and 3 inch diameters were penetrated in the soil depths of pre bored boreholes, and the data from instrumentation were collected. The measured data included the water porewater pressure, the pile side resistance, and pile displacement.

<table>
<thead>
<tr>
<th>Soil properties</th>
<th>Unit</th>
<th>Value</th>
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<tbody>
<tr>
<td>Liquid Limit (LL)</td>
<td>%</td>
<td>100</td>
</tr>
<tr>
<td>Plastic Index (PI)</td>
<td>%</td>
<td>72</td>
</tr>
<tr>
<td>Water Content (w)</td>
<td>%</td>
<td>73</td>
</tr>
<tr>
<td>Unit Weight (γ)</td>
<td>$KN/m^3$</td>
<td>15.4</td>
</tr>
<tr>
<td>Permeability (k)</td>
<td>$m/sec$</td>
<td>$2.4 \times 10^{-9}$</td>
</tr>
<tr>
<td>$\lambda$ and $\kappa$</td>
<td></td>
<td>0.34 and 0.09</td>
</tr>
<tr>
<td>$\mu$</td>
<td></td>
<td>0.50</td>
</tr>
</tbody>
</table>
In this study, the 3 inch (7.6 cm) x-probe was selected to perform the numerical study. The numerical model, similar to the previous case studies, was an axisymmetric FE model. A 20 m long pile was assumed prebored for the first 15 m, and the elements from depth 15 m to 17 m were used to study the pile segment response. The x-probe tip was located far enough from these elements to minimize the numerical deficiency related to the tip in the numerical modeling. Figure 5.36 shows the soil and pile geometries that were used in FE numerical model.
5.3.1 Results and Analysis

Several results were extracted from the numerical model and are presented here. The porewater pressure in the soil body generated from penetration of the x-probe at different times after end of probe penetration is shown in Figure 5.37. In this figure, the letter $d$ represents horizontal distance in the soil body from x-probe surface, and $D$ is the x-probe diameter. The figure
indicates that after two weeks from end of probe installation, the generated excess pore pressure dissipates and the consolidation step is completed.

![Variation of porewater pressure in soil body over time](image.png)

**Figure 5-37:** Variation of porewater pressure in soil body over time.

The obtained values of porewater normalized against the initial value of porewater pressure (i.e. the porewater pressure developed in soil body immediately after installation). The normalized values then were compared with the field measurements and with those obtained from other
numerical modeling techniques using different constitutive models, which are described in Shao and Desai (2000). They simulated this case study using HISS and DSC models, and compared the simulated results with the field measurements. A summary and comparison of these techniques can be seen in Figure 5.38.

Figure 5-38: Comparison between different models in predicting porewater pressure.

The induced stresses in the soil body during x-probe penetration were extracted from FE software to evaluate variation of shear and normal stresses in the soil body after installation. Figure 5.39 shows variation of shear stress and three main normal stresses in the soil body for the axisymmetric FE model. This figure indicates that maximum shear stress occurs at the soil-probe interface, and it reduces to the geostatic values for a horizontal distance equal to 10 times the probe diameter.
Similar to the previous case studies, the soil thixotropic effect was applied by assuming that the soil sensitivity is six, which yielded to a value of 0.59 for the strength reduction factor of fully remolded soil, $\beta_R$. The results obtained for the probe shaft resistance using the numerical study, with and without consideration of soil thixotropic response, are presented in Figure 5.40, which indicates that the shaft resistance at initial stage of consolidation is over-predicted in comparison with the field measurement. Furthermore, the shaft resistance for the long times after end of installation is under-predicted. These differences might be related the soil properties, especially the soil sensitivity value.
Case Study 4: Baton Rouge Cajun Site

Numerical simulation of an ongoing full-scale pile instrumentation and pile load test case study was conducted. The test site is located beside I-10 highway at 10 miles west from Baton Rouge, Louisiana. The test piles are square nominally 305 mm (12 in) wide, 18.29 m (60 ft) long, prestressed concrete pile. However, 1.22 m (4 feet) soil will be excavated before driving. The designed embedment depth of the pile is 18.29 m (60 ft) and 0.91 m (3 ft) will be left at top for performing static load tests and dynamic load tests. The arrangement of test piles and instrumentation layout are sketched in Figure 5.41.
The subsurface soil was investigated by the piezocone penetration test (PCPT), Field Vane shear test (VST) and extensive laboratory tests were performed on undisturbed soil samples. Information regarding the subsurface soil and pile instrumentation layout are shown in Figure 5.42. The laboratory tests included consolidated undrained (CU) triaxial test, consolidation test, etc.
Detailed information regarding laboratory test results are presented in appendix II. A summary of soil properties obtained from CU triaxial test can be seen in Table 5-5 and Figures 5.43 and 5.44. Figure 5.43 compares the undrained shear strength obtained from VST and calculated from PCPT measurement with the values obtained from CU triaxial test. The values of the critical state parameter $M$ at different depths of soil, calculated from CU test results, are shown in Figure 5.44.

Figure 5.42: Pile instrumentation layout and subsurface soil profile for Cajun site.
Table 5-5: Results obtained from triaxial test performed on Cajun site soils.

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Depth (ft)</th>
<th>Water content (%)</th>
<th>( \gamma' ) (lb/ft(^3))</th>
<th>( \Phi' )</th>
<th>( t_{90} ) (min)</th>
<th>( q_f ) (psi)</th>
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\( t_{90} \): Time for 90\% consolidation in the consolidation step of CU test.

\( q_f \): Deviatoric stress at the soil failure in CU test.

\( \Phi' \): Soil friction angle at critical state condition.

A comparison between VST and the triaxial test results is presented in Figure 5.45, which indicates that prediction by the triaxial test under-estimates soil undrained shear strength. The soil properties, which were used in the numerical simulation, are presented in Table 5-6. The geometry of the soil, the pile, the applied boundary conditions, and the corresponding soil layering for the Baton Rouge Cajun site are shown in Figure 5.46. Similar to other case studies, very fine element size was used at the pile-soil interaction zone. Figure 5.47 shows FE mesh for this site consists of 28974 linear rectangular elements.
Figure 5-43: Undrained shear strength obtained from VST, PCPT and CU triaxial test for Cajun site.

Figure 5-44: Critical state parameter along the soil depth for Cajun site.
Figure 5.45: Prediction of undrained shear strength from VST in comparison with Triaxial test for Cajun site.

Table 5.6: Soil properties for Baton Rouge Cajun site.

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<th>$k_0$</th>
<th>$S_u$ (kPa)</th>
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<th>M</th>
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Figure 5-46: Schematic representation of pile and soil domain for Cajun site.

Figure 5-47: FE mesh of Cajun site.
5.4.1 Results and Analysis

The porewater pressure change in the soil body during pile installation and over time after the pile installation was completed was evaluated. Figure 5.48 shows the generated porewater pressure immediately after pile placement in the cavity expansion (Figure 5.48,a) and its values after initial shear step (Figure 5.48,b). The induced cavity expansion in the soil following by an additional shear step (initial shear) generated excess porewater pressure in the soil, and it dissipated over time after EOD. At same time, the effective stress in the soil body increased. This variation of excess porewater pressure and the effective stress in the soil is shown in Figure 5.49. In order to study the pile setup phenomenon, variation of the pile shaft, tip, and total resistance over time after EOD, was extracted from numerical simulation results and plotted in Figure 5.50.

Figure 5-48: Change in porewater pressure at the pile tip: (a) before initial shear step, (b) and after initial step.
Figure 5.49: Variation of excess porewater pressure and effective stress in soil over time after EOD.
Figure 5.50: Pile resistance over time after EOD for Cajun site.
6 PARAMETRIC STUDY AND REGRESSION ANALYSIS

6.1 Parametric Study for Pile Setup Factor

In this section, numerical simulation techniques are used to investigate influence of the soil properties in the pile setup factor. Initially, engineering judgment was used to determine the soil properties as independent variables, which significantly affect pile setup. The stepwise procedure of variable selection was used to evaluate the significance of each variable. The soil properties selected for parametric study are: plasticity index $P_I$, shear strength $S_u$, coefficient of consolidation $C_v$, sensitivity $S_r$, and over-consolidation ratio OCR. More than one hundred regional soil properties (presented in Table 6.1), collected from the available literature, were used to simulate the FE modeling. A typical FE model of pile installation and the following setup, as described in a previous section, was used to conduct this parametric study. The selected soil properties have PI values which vary between 84% and 4%; the shear strength changes between 0.07 to 4.41 tsf; soil hydraulic conductivity values vary between 0.003 to 4.62 $ft^2/day$; the soil sensitivity values range between 1 to 13; and the soil is mostly normally consolidated and, for some cases, the maximum value reaches OCR=12.

The parametric study FE model includes a 85 cm diameter and 20 m long cylindrical pile, and the subsurface soil consists of four layers. The parametric study was performed by assigning specified soil properties to one layer (layer 3) only, while properties of other layers were kept constant and consistent with the properties of layer 3. Figure 6.1 is a schematic representation of the pile and soil domain with specified boundary conditions. Table 6-2 presents statistical analysis of the selected parameters for parametric study. Frequency analysis was performed on the obtained data to clarify distribution of each soil property, and the frequency histogram for each variable is shown in Figure 6.2.
Table 6-1: Soil properties used for numerical parametric study.

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<th>Soil No.</th>
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<th>$S_u$ (tsf)</th>
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<th>$S_r$</th>
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<td>0.15</td>
<td>0.25</td>
<td>0.018</td>
<td>2.34</td>
<td>1.00</td>
</tr>
<tr>
<td>66</td>
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<td>0.15</td>
<td>0.009</td>
<td>2.34</td>
<td>1.00</td>
</tr>
<tr>
<td>67</td>
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<td>0.40</td>
<td>0.046</td>
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</tr>
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<td>1.664</td>
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<td>1.00</td>
</tr>
<tr>
<td>69</td>
<td>0.28</td>
<td>0.80</td>
<td>4.622</td>
<td>2.63</td>
<td>1.00</td>
</tr>
<tr>
<td>70</td>
<td>0.16</td>
<td>0.58</td>
<td>0.555</td>
<td>6.90</td>
<td>1.00</td>
</tr>
<tr>
<td>71</td>
<td>0.07</td>
<td>0.61</td>
<td>4.622</td>
<td>9.15</td>
<td>1.10</td>
</tr>
</tbody>
</table>

(Table 6-1 continued)
(Table 6-1 continued)

<table>
<thead>
<tr>
<th>Soil No.</th>
<th>PI (%)</th>
<th>$S_d$ (tsf)</th>
<th>$C_v$ ($ft^2/day$)</th>
<th>$S_r$</th>
<th>OCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>72</td>
<td>0.09</td>
<td>1.02</td>
<td>0.370</td>
<td>3.12</td>
<td>4.00</td>
</tr>
<tr>
<td>73</td>
<td>0.50</td>
<td>0.70</td>
<td>0.832</td>
<td>13.00</td>
<td>8.60</td>
</tr>
<tr>
<td>74</td>
<td>0.25</td>
<td>0.39</td>
<td>0.092</td>
<td>2.00</td>
<td>2.50</td>
</tr>
<tr>
<td>75</td>
<td>0.18</td>
<td>1.20</td>
<td>2.126</td>
<td>3.00</td>
<td>2.80</td>
</tr>
<tr>
<td>76</td>
<td>0.05</td>
<td>0.65</td>
<td>0.018</td>
<td>1.50</td>
<td>3.60</td>
</tr>
<tr>
<td>77</td>
<td>0.08</td>
<td>3.33</td>
<td>0.647</td>
<td>6.25</td>
<td>5.10</td>
</tr>
<tr>
<td>78</td>
<td>0.05</td>
<td>4.41</td>
<td>0.740</td>
<td>9.30</td>
<td>11.80</td>
</tr>
<tr>
<td>79</td>
<td>0.20</td>
<td>0.55</td>
<td>0.185</td>
<td>1.00</td>
<td>4.20</td>
</tr>
<tr>
<td>80</td>
<td>0.45</td>
<td>0.50</td>
<td>0.028</td>
<td>3.00</td>
<td>4.80</td>
</tr>
<tr>
<td>81</td>
<td>0.22</td>
<td>0.30</td>
<td>0.185</td>
<td>2.50</td>
<td>2.00</td>
</tr>
<tr>
<td>82</td>
<td>0.48</td>
<td>0.30</td>
<td>0.028</td>
<td>2.50</td>
<td>1.00</td>
</tr>
<tr>
<td>83</td>
<td>0.15</td>
<td>0.90</td>
<td>0.028</td>
<td>2.65</td>
<td>4.00</td>
</tr>
<tr>
<td>84</td>
<td>0.18</td>
<td>0.50</td>
<td>0.462</td>
<td>2.65</td>
<td>5.00</td>
</tr>
<tr>
<td>85</td>
<td>0.14</td>
<td>0.40</td>
<td>0.740</td>
<td>2.80</td>
<td>3.00</td>
</tr>
<tr>
<td>86</td>
<td>0.30</td>
<td>1.20</td>
<td>0.055</td>
<td>2.00</td>
<td>1.00</td>
</tr>
<tr>
<td>87</td>
<td>0.25</td>
<td>0.50</td>
<td>0.185</td>
<td>2.00</td>
<td>3.50</td>
</tr>
<tr>
<td>88</td>
<td>0.45</td>
<td>0.20</td>
<td>0.185</td>
<td>2.30</td>
<td>1.00</td>
</tr>
<tr>
<td>89</td>
<td>0.18</td>
<td>0.30</td>
<td>0.277</td>
<td>2.50</td>
<td>1.00</td>
</tr>
<tr>
<td>90</td>
<td>0.22</td>
<td>0.25</td>
<td>0.009</td>
<td>3.40</td>
<td>10.00</td>
</tr>
<tr>
<td>91</td>
<td>0.19</td>
<td>0.20</td>
<td>0.009</td>
<td>7.00</td>
<td>3.00</td>
</tr>
<tr>
<td>92</td>
<td>0.18</td>
<td>0.17</td>
<td>0.009</td>
<td>6.00</td>
<td>2.00</td>
</tr>
<tr>
<td>93</td>
<td>0.40</td>
<td>0.30</td>
<td>0.009</td>
<td>3.50</td>
<td>1.00</td>
</tr>
<tr>
<td>94</td>
<td>0.23</td>
<td>0.40</td>
<td>0.018</td>
<td>2.50</td>
<td>1.00</td>
</tr>
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<td>95</td>
<td>0.15</td>
<td>1.10</td>
<td>0.555</td>
<td>1.60</td>
<td>12.00</td>
</tr>
<tr>
<td>96</td>
<td>0.20</td>
<td>0.50</td>
<td>0.277</td>
<td>2.00</td>
<td>5.00</td>
</tr>
<tr>
<td>97</td>
<td>0.24</td>
<td>0.40</td>
<td>0.370</td>
<td>2.90</td>
<td>3.50</td>
</tr>
<tr>
<td>98</td>
<td>0.15</td>
<td>1.20</td>
<td>0.185</td>
<td>4.70</td>
<td>2.50</td>
</tr>
<tr>
<td>99</td>
<td>0.32</td>
<td>0.25</td>
<td>0.037</td>
<td>3.00</td>
<td>6.00</td>
</tr>
<tr>
<td>100</td>
<td>0.40</td>
<td>0.20</td>
<td>0.037</td>
<td>2.50</td>
<td>4.00</td>
</tr>
<tr>
<td>101</td>
<td>0.20</td>
<td>0.30</td>
<td>0.028</td>
<td>3.25</td>
<td>2.50</td>
</tr>
<tr>
<td>102</td>
<td>0.25</td>
<td>0.45</td>
<td>0.018</td>
<td>2.90</td>
<td>3.50</td>
</tr>
<tr>
<td>103</td>
<td>0.23</td>
<td>0.38</td>
<td>0.009</td>
<td>3.70</td>
<td>1.50</td>
</tr>
<tr>
<td>104</td>
<td>0.05</td>
<td>3.75</td>
<td>0.740</td>
<td>9.30</td>
<td>11.80</td>
</tr>
</tbody>
</table>
Figure 6-1: Numerical simulation domain used for parametric study.

Table 6-2: Statistical analysis of the selected parameters for parametric study.

<table>
<thead>
<tr>
<th>Statistic</th>
<th>PI</th>
<th>$S_u$ (tsf)</th>
<th>$C_{v} \left( \frac{t^2}{\text{day}} \right)$</th>
<th>$S_r$</th>
<th>OCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum</td>
<td>0.04</td>
<td>0.07</td>
<td>0.003</td>
<td>1.00</td>
<td>0.25</td>
</tr>
<tr>
<td>Maximum</td>
<td>0.84</td>
<td>4.41</td>
<td>4.62</td>
<td>13.00</td>
<td>12.00</td>
</tr>
<tr>
<td>Range</td>
<td>0.80</td>
<td>4.34</td>
<td>4.62</td>
<td>12.00</td>
<td>11.75</td>
</tr>
<tr>
<td>Average</td>
<td>0.35</td>
<td>0.64</td>
<td>0.35</td>
<td>5.18</td>
<td>2.26</td>
</tr>
<tr>
<td>Std. Deviation</td>
<td>0.19</td>
<td>0.67</td>
<td>0.81</td>
<td>2.79</td>
<td>2.57</td>
</tr>
</tbody>
</table>
Figure 6.2: Frequency histogram for soil properties used for parametric study.

Table 6.3 represents correlation coefficient between the model variables, which were used to evaluate the setup factor. This table indicates that these variables can be divided into two groups: the first includes the plasticity index and sensitivity, and the second includes soil shear strength,
coefficient of consolidation, and OCR. The variables of group one and group two have inverse relation with each other.

Table 6-3: Correlation between regression model potential variables.

<table>
<thead>
<tr>
<th></th>
<th>PI</th>
<th>$S_u$</th>
<th>$C_v$</th>
<th>$S_r$</th>
<th>OCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>PI</td>
<td>1</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$S_u$</td>
<td>-0.4719</td>
<td>1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$C_v$</td>
<td>-0.30202</td>
<td>0.287694</td>
<td>1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$S_r$</td>
<td>0.493141</td>
<td>-0.03414</td>
<td>-0.12748</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>OCR</td>
<td>-0.42772</td>
<td>0.541487</td>
<td>0.070131</td>
<td>-0.24363</td>
<td>1</td>
</tr>
</tbody>
</table>

In order to evaluate pile/soil setup for the different soils presented in Table 6-1, the values of setup factor $A$ introduced by Skov and Denver (1988) were determined using FE numerical simulation. The pile resistance $R$ obtained from numerical simulation at four times after end of pile driving ($t=1, 10, 100, 1000$ days) were used to calibrate the following equation:

$$\frac{R_t}{R_0} = A \log\left(\frac{t}{t_0}\right) + 1 \quad (6.1)$$

In this study, the value of $R_0$ was considered to be the pile resistance at time $t_0 = 1 \text{ day}$. Therefore, the setup factor $A$ is the slope of best fit line applied to the four points corresponding to $t=1$, $10$, $100$, and $1000$ days, and it force to have intercept value of 1. A sample explanation of this method for a typical soil is presented in Figure 6.3. The FE numerical model was run for each case, which were presented in Table 6-1, for four specified times after EOD, and the shaft resistance corresponds to the pile segment adjacent to layer 3. The numerical simulation was performed, and values of $A$ factor were obtained for all of the 104 different soil types by calculating the relation between normalized shaft resistance and the elapsed time after EOD (similar to Figure 6.3). The obtained values for $A$ were initially analyzed indicating that a minimum value of $0.10$
and a maximum value of 0.50 for the A factor were obtained from the FE model. The frequency histogram for A factor is shown in Figure 6.4.

![Figure 6.3: Determination of setup factor A for a typical soil sample.](image)

![Figure 6.4: Frequency histogram for setup parameter A obtained from numerical simulation.](image)

6.1.1 Effect of Soil Properties on Setup Factor A

In order to evaluate the correlation between Factor A and each soil parameters, the corresponding values for A and each independent parameter were drawn in graphic form in Figures
6.5 to 6.9. These figures indicate that $A$ has a direct relation with the soil plasticity index $PI$ and the sensitivity ratio $S_s$, and it has an inverse relation with soil shear strength $S_u$, consolidation coefficient $C_v$, and over-consolidation ratio $OCR$. These trends between the $A$ and the soil properties will be used to conduct nonlinear regression analysis in the next section.

Figure 6-5: Relation between setup factor $A$ and soil shear strength.

Figure 6-6: Relation between setup factor $A$ and soil plasticity index.
Figure 6.7: Relation between setup factor $A$ and soil coefficient of consolidation.

Figure 6.8: Relation between setup factor $A$ and soil overconsolidation ratio.

Figure 6.9: Relation between setup factor $A$ and soil sensitivity ratio.
6.2 Regression analysis

As explained earlier, the selected soil properties for parametric study were based on engineering judgment. However, evaluation of significance level for each independent variables is necessary, which requires an appropriate correlation technique. Application of T-test with obtaining P-value is a common technique in order to evaluate degree of correlation between dependent and independent variables. P-value represents the significance level within a statistical hypothesis test, and it indicates the probability of the occurrence of a given event. In this research, the P-values were obtained using T-test, and their magnitudes were compared with significance level (α=0.05). First, statistical analysis was applied to correlate each independent parameter individually with setup factor $A$, and the obtained P-values are shown in Table 6-4. The backward stepwise procedures were then used to examine the significance levels of the independent variables. The summary of P-value and other statistical parameters obtained from this analysis is shown in Figure 6.10, which demonstrates that all five selected variables are significant and can be used as an independent variable in regression analysis.

Table 6-4: Evaluation of correlation of individual independent variables.

<table>
<thead>
<tr>
<th></th>
<th>Coefficients</th>
<th>Standard Error</th>
<th>t-Stat</th>
<th>P-value</th>
<th>Lower 95%</th>
<th>Upper 95%</th>
</tr>
</thead>
<tbody>
<tr>
<td>PI</td>
<td>0.331</td>
<td>0.033</td>
<td>10.034</td>
<td>6.81E-17</td>
<td>0.266</td>
<td>0.397</td>
</tr>
<tr>
<td>$S_u$</td>
<td>-0.079</td>
<td>0.011</td>
<td>-7.245</td>
<td>8.46E-11</td>
<td>-0.100</td>
<td>-0.057</td>
</tr>
<tr>
<td>$C_v$</td>
<td>-0.0410</td>
<td>0.0102</td>
<td>-4.006</td>
<td>0.0001</td>
<td>-0.061</td>
<td>-0.020</td>
</tr>
<tr>
<td>OCR</td>
<td>-0.0181</td>
<td>0.0029</td>
<td>-6.083</td>
<td>2.07E-08</td>
<td>-0.024</td>
<td>-0.012</td>
</tr>
<tr>
<td>$S_r$</td>
<td>0.019</td>
<td>0.002</td>
<td>7.487</td>
<td>2.58E-11</td>
<td>0.0141</td>
<td>0.0243</td>
</tr>
</tbody>
</table>
6.2.1 Regression Analysis with Two Independent Variables

The regression analysis was divided into four phases. In the first phase, the setup factor $A$ was correlated to the soil shear strength $S_u$ and plasticity index $PI$. These two parameters were selected based on engineering judgment and the fact that these parameters are the most effective and easily measured soil properties. Non-linear regression analysis was conducted using Statistical Analysis System (SAS) and CurveExpert Professional (CE-P) softwares. The latter was used because it can
easily perform non-linear regression for several models simultaneously. Candidate models were selected and offered in non-linear regression analysis based on the rational relations exist between $A$ and the variables. Table 6-5 presents the models for two-variable non-linear regression analysis, which reflected the best correlation. It is notable that the squared R, ($R^2$), calculated here it pseudo $R^2$, because the actual values for cannot be attained (i.e., is meaningless) in non-linear regression analysis.

6.2.2 Regression Analysis with Three Independent Variables

In the second phase of regression analysis, the coefficient of consolidation $C_v$ was first considered as a third independent variable, OCR then replaced with $C_v$, and regression analysis was repeated. $C_v$ and OCR have an inverse relation with the setup factor $A$, and they were therefore used as denominators in the proposed models. Non-linear regression analyses using three variables were performed and the results are presented in Tables 6-6 and 6-7.

As an option for regression analysis with three parameters, the soil sensitivity $S_r$ was used with PI and $S_u$. The statistical analyses were conducted for several models, and the best models based on correlation coefficient are presented in Table 6-8.

6.2.3 Regression Analysis with Four Independent Variables

In third phase, regression analyses using four independent variables were performed. The first selected four parameters are: PI, $S_u$, $C_v$ and OCR. Regression analysis was conducted based on reasonable relations between each independent variable and the setup factor. Table 6-9 presents regression models, which describe the appropriate setup phenomenon. Another set of selected four parameters including PI,$S_u$, $C_v$ and $S_r$ were analyzed using non-linear regression analysis. The models that yielded the best correlation between parameters are shown in Table 6-10.
6.2.4 Regression Analysis with Five Independent Variables

In the last phase of regression analysis, all five independent variables PI, $S_u$, $C_v$, OCR and $S_r$ were used. Similar regression analyses were performed to evaluate different models, and those with the best correlation are presented in Table 6-11.

Table 6-5: Predicted regression models with two variables (PI and $S_u$).

<table>
<thead>
<tr>
<th>Model No.</th>
<th>Model</th>
<th>R2</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$A = 0.76 * \frac{PI + 1}{S_u^{1.14} + 2.97}$</td>
<td>0.62</td>
</tr>
<tr>
<td>2</td>
<td>$A = \frac{0.76PI + 0.73}{S_u^{1.13} + 2.86}$</td>
<td>0.62</td>
</tr>
<tr>
<td>3</td>
<td>$A = \frac{1.04PI^{0.24}}{S_u^{0.84} + 2.16}$</td>
<td>0.61</td>
</tr>
<tr>
<td>4</td>
<td>$A = \frac{1.02PI + 1}{1.46S_u + 3.93}$</td>
<td>0.62</td>
</tr>
<tr>
<td>5</td>
<td>$A = 0.63PI^{0.12} - 0.29S_u^{0.18}$</td>
<td>0.59</td>
</tr>
<tr>
<td>6</td>
<td>$A = 0.45PI^{0.25}e^{-0.29S_u}$</td>
<td>0.60</td>
</tr>
<tr>
<td>7</td>
<td>$\ln(A) = 0.7PI - 0.2S_u - 1.37$</td>
<td>0.61</td>
</tr>
<tr>
<td>8</td>
<td>$A = 0.34 \frac{PI^{0.24}}{S_u^{0.14}}$</td>
<td>0.59</td>
</tr>
<tr>
<td>9</td>
<td>$A = 0.31\left(\frac{PI + 1}{S_u + 1}\right)^{0.69}$</td>
<td>0.60</td>
</tr>
<tr>
<td>10</td>
<td>$A = 0.31\left(\frac{PI}{S_u}\right)^{0.18}$</td>
<td>0.59</td>
</tr>
</tbody>
</table>
Table 6-6: Predicted regression models with three variables (PI, $S_u$ and $C_v$).

<table>
<thead>
<tr>
<th>Model No.</th>
<th>Model</th>
<th>R²</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$A = 0.38 \frac{PI^{0.7} + 1}{S_u^{0.48} \times C_v^{0.2} + 1.51}$</td>
<td>0.65</td>
</tr>
<tr>
<td>2</td>
<td>$A = \frac{PI^{1.4} + 1}{S_u^{1.22} \times C_v^{0.2} + 3.92}$</td>
<td>0.63</td>
</tr>
<tr>
<td>3</td>
<td>$A = 0.34 \frac{PI^{0.68} + 1}{(S_u \times C_v)^{0.23} + 1.27}$</td>
<td>0.65</td>
</tr>
<tr>
<td>4</td>
<td>$A = \frac{1.5PI + 1.78}{S_u^{0.6} \times (\log(C_v - 0.01 \ln^2/hr)^{1.28} + 6.73}$</td>
<td>0.65</td>
</tr>
<tr>
<td>5</td>
<td>$A = \frac{0.37PI + 0.45}{S_u^{0.54} \times C_v^{0.21} + 1.58}$</td>
<td>0.65</td>
</tr>
<tr>
<td>6</td>
<td>$A = 0.45PI^{0.24}e^{-(0.26S_u + 0.06C_v)}$</td>
<td>0.62</td>
</tr>
<tr>
<td>7</td>
<td>$A = 0.43PI^{0.32}e^{-(0.11S_u \times C_v}$</td>
<td>0.55</td>
</tr>
<tr>
<td>8</td>
<td>$A = 0.26(\frac{PI + 1}{S_u \times C_v + 1})^{0.68}$</td>
<td>0.53</td>
</tr>
<tr>
<td>9</td>
<td>$A = 0.29 \frac{PI^{0.2}}{(S_u \times C_v)^{0.06}}$</td>
<td>0.64</td>
</tr>
<tr>
<td>10</td>
<td>$A = 0.23(\frac{PI}{S_u \times C_v})^{0.08}$</td>
<td>0.60</td>
</tr>
</tbody>
</table>
Table 6-7: Predicted Regression models with three variables (PI, $S_u$ and OCR).

<table>
<thead>
<tr>
<th>Model No.</th>
<th>Model</th>
<th>R2</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$A = 0.46 \frac{PI^{0.46} + 1}{S_u^{0.44} \times OCR^{0.31} + 1.73}$</td>
<td>0.70</td>
</tr>
<tr>
<td>2</td>
<td>$A = 0.46 \frac{PI^{0.46} + 1}{(S_u \times OCR)^{0.33} + 1.68}$</td>
<td>0.67</td>
</tr>
<tr>
<td>3</td>
<td>$A = 0.65 \frac{PI^{0.47} + 1}{S_u^{0.99} + OCR^{0.30} + 1.98}$</td>
<td>0.67</td>
</tr>
<tr>
<td>4</td>
<td>$A = \frac{PI^{0.63} + 1}{S_u^{1.29} + OCR^{0.39} + 3.56}$</td>
<td>0.67</td>
</tr>
<tr>
<td>5</td>
<td>$A = 0.45PI^{0.2}e^{-(0.25S_u+0.04OCR)}$</td>
<td>0.64</td>
</tr>
<tr>
<td>6</td>
<td>$A = 0.43PI^{0.25}e^{-0.07S_u \times OCR}$</td>
<td>0.66</td>
</tr>
<tr>
<td>7</td>
<td>$\ln(A) = 0.7PI - 0.2S_u - 0.03OCR - 1.32$</td>
<td>0.64</td>
</tr>
<tr>
<td>8</td>
<td>$A = 0.33(\frac{PI + 1}{S_u \times OCR + 1})^{0.35}$</td>
<td>0.63</td>
</tr>
<tr>
<td>9</td>
<td>$A = 0.32 \frac{PI^{0.14}}{(S_u \times OCR)^{0.12}}$</td>
<td>0.66</td>
</tr>
<tr>
<td>10</td>
<td>$A = 0.31(\frac{PI}{S_u \times OCR})^{0.12}$</td>
<td>0.66</td>
</tr>
</tbody>
</table>
Table 6-8: Predicted regression models with three variables (PI, $S_u$ and $S_r$).

<table>
<thead>
<tr>
<th>Model No.</th>
<th>Model</th>
<th>R2</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$A = 0.74 \frac{PI^{0.42} + S_r^{0.25} + 1}{S_u^{1.45} + 3.94}$</td>
<td>0.74</td>
</tr>
<tr>
<td>2</td>
<td>$A = 0.53 \frac{PI^{0.22} \times S_r^{0.38} + 1}{S_u^{1.41} + 3.86}$</td>
<td>0.73</td>
</tr>
<tr>
<td>3</td>
<td>$A = 0.31(PI \times S_r)^{0.18}e^{-0.24S_u}$</td>
<td>0.71</td>
</tr>
<tr>
<td>4</td>
<td>$A = 0.44PI^{0.36}e^{-0.005S_u \times S_r}$</td>
<td>0.50</td>
</tr>
<tr>
<td>5</td>
<td>$\ln(A) = 0.7PI - 0.2S_u + 0.04S_r - 1.60$</td>
<td>0.67</td>
</tr>
<tr>
<td>6</td>
<td>$A = 0.23\left(\frac{PI}{S_u}\right)^{0.14} \times S_r^{0.19}$</td>
<td>0.68</td>
</tr>
<tr>
<td>7</td>
<td>$A = 0.24\left(\frac{PI \times S_r}{S_u}\right)^{0.15}$</td>
<td>0.68</td>
</tr>
</tbody>
</table>
Table 6-9: Predicted regression models with four variables (PI, $S_u$, $C_v$ and OCR).

<table>
<thead>
<tr>
<th>Model No.</th>
<th>Model</th>
<th>R²</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$A = 0.31 \frac{PI^{0.43} + 1}{(S_u \times OCR \times C_v)^{0.18} + 1.18}$</td>
<td>0.69</td>
</tr>
<tr>
<td>2</td>
<td>$A = 0.35 \frac{PI^{0.37} + 1}{S_u^{0.56} \times C_v^{0.12} \times OCR^{0.28} + 1.34}$</td>
<td>0.70</td>
</tr>
<tr>
<td>3</td>
<td>$A = \frac{PI^{0.42} + 1}{S_u^{1.17} + C_v^{0.41} + OCR^{0.40} + 3.58}$</td>
<td>0.69</td>
</tr>
<tr>
<td>4</td>
<td>$A = 0.45PI^{0.19} e^{-0.21S_u + 0.07C_v + 0.04OCR}$</td>
<td>0.65</td>
</tr>
<tr>
<td>5</td>
<td>$A = 0.43PI^{0.31} e^{-0.04S_u \times OCR \times C_v}$</td>
<td>0.54</td>
</tr>
<tr>
<td>6</td>
<td>$\ln(A) = 0.72PI - 0.15S_u - 0.07C_v - 0.03OCR - 1.35$</td>
<td>0.66</td>
</tr>
<tr>
<td>7</td>
<td>$\ln(A) = 0.7PI - 0.2S_u - 0.07C_v - 0.03OCR - 1.31$</td>
<td>0.65</td>
</tr>
<tr>
<td>8</td>
<td>$A = 0.27 \frac{PI^{0.14}}{(S_u \times OCR \times C_v)^{0.06}}$</td>
<td>0.67</td>
</tr>
<tr>
<td>9</td>
<td>$A = 0.24(\frac{PI}{S_u \times OCR \times C_v})^{0.07}$</td>
<td>0.66</td>
</tr>
</tbody>
</table>
Table 6-10: Predicted regression models with four variables ($PI, S_u, C_v$ and $S_r$).

<table>
<thead>
<tr>
<th>Model No.</th>
<th>Model</th>
<th>R2</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$A = 0.37 \frac{(PI \times S_r)^{0.31} + 1}{(S_u \times C_v)^{0.41} + 2.4}$</td>
<td>0.72</td>
</tr>
<tr>
<td>2</td>
<td>$A = \frac{0.43PI \times S_r + 4.48}{S_u^{0.97} \times (\log(\frac{C_v}{0.01 \text{in2/hr}}))^{1.83} + 16.73}$</td>
<td>0.71</td>
</tr>
<tr>
<td>3</td>
<td>$A = \frac{0.07PI \times S_r + 0.7}{S_u^{0.99} \times C_v^{0.29} + 2.61}$</td>
<td>0.71</td>
</tr>
<tr>
<td>4</td>
<td>$A = 0.31(PI \times S_r)^{0.17}e^{-(0.22S_u + 0.05C_v)}$</td>
<td>0.74</td>
</tr>
<tr>
<td>5</td>
<td>$A = 0.28(PI \times S_r)^{0.2}e^{-0.13S_u \times C_v}$</td>
<td>0.69</td>
</tr>
<tr>
<td>6</td>
<td>$A = 0.44PI^{0.24}e^{-0.06S_u \times S_r \times C_v}$</td>
<td>0.62</td>
</tr>
<tr>
<td>7</td>
<td>$\ln(A) = 0.7PI - 0.2S_u - 0.07C_v + 0.03S_r - 1.62$</td>
<td>0.50</td>
</tr>
<tr>
<td>8</td>
<td>$A = 0.23 \frac{(PI \times S_r)^{0.16}}{(S_u \times C_v)^{0.04}}$</td>
<td>0.70</td>
</tr>
<tr>
<td>9</td>
<td>$A = 0.21(PI \times S_r)^{0.08}S_u \times C_v$</td>
<td>0.65</td>
</tr>
</tbody>
</table>
Table 6-11: Predicted regression models with Five variables (PI, $S_u$, $C_v$, OCR and $S_r$).

<table>
<thead>
<tr>
<th>Model No.</th>
<th>Model</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$A = 0.82 \frac{(PI \times S_r)^{0.22} + 1}{(S_u \times C_v \times OCR)^{0.40} + 3.69}$</td>
<td>0.74</td>
</tr>
<tr>
<td>2</td>
<td>$A = \frac{0.77 PI \times S_r + 0.91}{S_u^{0.61} \times C_v^{0.26} \times OCR^{0.41} + 3.2}$</td>
<td>0.72</td>
</tr>
<tr>
<td>2</td>
<td>$A = 0.32(PI \times S_r)^{0.16}e^{-(0.2S_u + 0.05 C_v + 0.02 OCR)}$</td>
<td>0.74</td>
</tr>
<tr>
<td>3</td>
<td>$A = 0.28(PI \times S_r)^{0.21}e^{-0.04S_u \times C_v \times OCR}$</td>
<td>0.70</td>
</tr>
<tr>
<td>4</td>
<td>$\ln(A) = 0.22PI - 0.22S_u - 0.07C_v - 0.02OCR + 0.04S_r - 1.35$</td>
<td>0.73</td>
</tr>
<tr>
<td>5</td>
<td>$\ln(A) = 0.7PI - 0.2S_u - 0.07C_v - 0.03OCR + 0.04S_r - 1.55$</td>
<td>0.67</td>
</tr>
<tr>
<td>6</td>
<td>$A = 0.23 \frac{(PI \times S_r + 1)^{0.31}}{(S_u \times C_v \times OCR + 1)^{0.21}}$</td>
<td>0.72</td>
</tr>
<tr>
<td>7</td>
<td>$A = 0.24 \frac{PI \times S_r + 1}{S_u \times C_v \times OCR + 1}^{0.27}$</td>
<td>0.71</td>
</tr>
<tr>
<td>8</td>
<td>$A = 0.24 \frac{(PI \times S_r)^{0.13}}{(S_u \times C_v \times OCR)^{0.04}}$</td>
<td>0.71</td>
</tr>
<tr>
<td>9</td>
<td>$A = 0.22 \frac{PI \times S_r}{S_u \times C_v \times OCR}^{0.06}$</td>
<td>0.69</td>
</tr>
</tbody>
</table>
6.3 Analyzing the Developed Models

The results obtained from regression analysis showed that increasing the number of independent variables increases the correlation coefficient. The final selected models were arranged in three different sets of equations that relate the setup factor \( A \) to the corresponding soil properties that were specified as independent variables in the regression analyses. Set-1, which is shown in Table 6-12 presents the set of fractional relation obtained between the A factor and the different soil variables. Tables 6-13 and 6-14 present the set-2 and set-3 of correlation models, which have exponential and power relation between the A factor and different soil variables, respectively. In the tables the values \( R^2 \) represents the pseudo correlation of correlation \( R_2 \) since the actual values for it is not directly reachable in the nonlinear regression analysis. In addition, the Cross-Validated Standard Error of Prediction (CVSEP) and Cross-Validated Average Error of Prediction (CVAEP) were added to these tables in order to clarify the level of error in each model. External evaluation technique was adopted on the data to obtain CVSEP and CVAEP values. This technique was achieved from application of the regression equations of these tables (which were obtained based on 67% randomly selected data out of all data) to the remained 33% data out of all data, which yielded a value for predicted setup factor, \( \hat{A} \). These errors were calculated based on the variation of the externally predicted \( \hat{A} \) from the A values obtained directly from numerical simulation, using the following equations:

\[
CVSEP = \sqrt{\frac{\sum (\hat{A} - A)^2}{n}}
\]

(6-1)

\[
CVAEP = \sum \left| \frac{(\hat{A} - A)}{n} \right|
\]
In Equation 6-1, \( n = 34 \) (or \( 104 \times 33\% = 34 \)) representing number of data set used to perform the external evaluation.

Table 6-12: Regression model set-1.

<table>
<thead>
<tr>
<th>Model No.</th>
<th>Number of variables</th>
<th>Model description</th>
<th>( R^2 )</th>
<th>CVSEP</th>
<th>CVAEP</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5</td>
<td>( A = 0.82 \frac{(PI \times S_r)^{0.22} + 0.37}{(S_u \times C_v \times OCR)^{0.40} + 3.69} )</td>
<td>0.73</td>
<td>0.0412</td>
<td>0.0320</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
<td>( A = 0.28 \frac{PI^{0.67} + 1.44}{(S_u \times OCR \times C_v)^{0.19} + 1.28} )</td>
<td>0.69</td>
<td>0.0476</td>
<td>0.0397</td>
</tr>
<tr>
<td>3</td>
<td>3</td>
<td>( A = 0.34 \frac{PI^{0.66} + 0.97}{(S_u \times C_v)^{0.22} + 1.26} )</td>
<td>0.65</td>
<td>0.0542</td>
<td>0.0449</td>
</tr>
<tr>
<td>4</td>
<td>3</td>
<td>( A = 0.43 \frac{PI^{0.61} + 1.28}{(S_u \times OCR)^{0.35} + 1.82} )</td>
<td>0.67</td>
<td>0.0498</td>
<td>0.0415</td>
</tr>
<tr>
<td>5</td>
<td>2</td>
<td>( A = 0.76 \frac{PI + 0.96}{S_u^{1.14} + 2.96} )</td>
<td>0.62</td>
<td>0.0556</td>
<td>0.0493</td>
</tr>
</tbody>
</table>

The results of regression analyses, as presented in Tables 6-12, 6-13 and 6-14, indicate that \( R^2 \) increase, while CVSEP and CVAEP decrease with increasing number of independent soil variables. By comparing, the values of \( R^2 \), CVSEP and CVAEP presented in the last two columns of these tables, the reader can realize that the correlation equation in these three sets have almost
the same level of accuracy. Furthermore, each set of models presented in Tables 6-12, 6-13, and 6-14 includes five regression models, which are ranked from 1 to 5 based on the corresponding value of errors. The model number 1 in each set represents the best equation to estimate the setup factor $A$, which can be used to estimate the $A$ values if all the required soil properties (i.e., PI, $S_u$, $C_v, S_r$ and OCR) are available. However, in the case not all the required soil properties are available, the reader can use models 2 to 5 of each set with acceptable accuracy to estimate the setup factor $A$, depending on availability of the soil properties. This concept can be applied in order to evaluate the three sets of models presented in Tables 6-12, 6-13, and 6-14.

Table 6-13: Regression model set-2.

<table>
<thead>
<tr>
<th>Model No.</th>
<th>Number of variables</th>
<th>Model description</th>
<th>$R^2$</th>
<th>CVSEP</th>
<th>CVAEP</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5</td>
<td>$A = 0.21 \times e^{0.7\text{PI}-0.2S_u-0.07C_v-0.03\text{OCR}+0.04S_r}$</td>
<td>0.68</td>
<td>0.0539</td>
<td>0.0437</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
<td>$A = 0.27 \times e^{0.7\text{PI}-0.2S_u-0.07C_v-0.03\text{OCR}}$</td>
<td>0.65</td>
<td>0.0534</td>
<td>0.0442</td>
</tr>
<tr>
<td>3</td>
<td>3</td>
<td>$A = 0.26 \times e^{0.7\text{PI}-0.2S_u-0.07C_v}$</td>
<td>0.63</td>
<td>0.0563</td>
<td>0.0469</td>
</tr>
<tr>
<td>4</td>
<td>3</td>
<td>$A = 0.27 \times e^{0.7\text{PI}-0.2S_u-0.03\text{OCR}}$</td>
<td>0.65</td>
<td>0.0558</td>
<td>0.0464</td>
</tr>
<tr>
<td>5</td>
<td>2</td>
<td>$A = 0.25 \times e^{0.7\text{PI}-0.2S_u}$</td>
<td>0.61</td>
<td>0.0578</td>
<td>0.0483</td>
</tr>
</tbody>
</table>
Table 6-14: Regression model set-3.

<table>
<thead>
<tr>
<th>Model No.</th>
<th>Number of variables</th>
<th>Model</th>
<th>$R^2$</th>
<th>CVSEP</th>
<th>CVAEP</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5</td>
<td>$A = 0.22 \left( \frac{\text{PI} \times S_r}{S_u \times OCR \times C_v} \right)^{0.06}$</td>
<td>0.68</td>
<td>0.0471</td>
<td>0.0395</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
<td>$A = 0.24 \left( \frac{\text{PI}}{S_u \times OCR \times C_v} \right)^{0.07}$</td>
<td>0.66</td>
<td>0.0509</td>
<td>0.0423</td>
</tr>
<tr>
<td>3</td>
<td>3</td>
<td>$A = 0.23 \left( \frac{\text{PI}}{S_u \times C_v} \right)^{0.08}$</td>
<td>0.60</td>
<td>0.0549</td>
<td>0.0458</td>
</tr>
<tr>
<td>4</td>
<td>3</td>
<td>$A = 0.31 \left( \frac{\text{PI}}{S_u \times OCR} \right)^{0.12}$</td>
<td>0.66</td>
<td>0.0517</td>
<td>0.0467</td>
</tr>
<tr>
<td>5</td>
<td>2</td>
<td>$A = 0.31 \left( \frac{\text{PI}}{S_u} \right)^{0.18}$</td>
<td>0.59</td>
<td>0.0585</td>
<td>0.0482</td>
</tr>
</tbody>
</table>

6.4 Verification of the proposed model

To verify the proposed regression models in Tables 6-12, 6-13 and 6-14, the information of soil properties and setup values for additional sites were collected from literature (e.g., Titi and Wathugala, 1999; Augustesen, 2006; and Ng, 2011). The selected additional sites were not included in the database used in parametric study to develop the regression models. Table 6-15 presents the additional selected sites used for verification and the corresponding soil properties as
well as the measured A factor. In this table, the A values were back-calculated from static and
dynamic field load tests. Each set of models (set-1, set-2, and set-3) was used to calculate the setup
factor A based on the availability of the soil properties presented in Table 6-15. This means that
model 1 of each set was used to predict the A if all soil properties are available, while models 2 to
5 were used if some values of the soil properties were not available. Figure 6.11 presents the
comparison between the predicted A using the proposed regression models of each set and the
back-calculated A values from static and dynamic load tests. The figure indicates that the three
sets of models proposed in Tables 6-12, 6-13 and 6-14 are able to reasonably estimate the soil
setup behavior, especially for soils with A values greater than 0.10. The figure also demonstrated
that the predictions of A values using the models set-1 (Figure 6-1,a) are slightly better than the
predictions of the other two model sets (Figure 6-11,b and 6-11,c).
Table 6-15: Site information used for verification of proposed setup models.

<table>
<thead>
<tr>
<th>No.</th>
<th>Site name</th>
<th>Reference</th>
<th>PI</th>
<th>$S_u$</th>
<th>$C_v$</th>
<th>OCR</th>
<th>Measured Field</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Sabin River</td>
<td>Titi and Wathugala (1999)</td>
<td>0.73</td>
<td>0.24</td>
<td>0.01</td>
<td>1</td>
<td>0.45</td>
</tr>
<tr>
<td>2</td>
<td>Houston</td>
<td></td>
<td>0.31</td>
<td>1.09</td>
<td>-</td>
<td>8.1</td>
<td>0.23</td>
</tr>
<tr>
<td>3</td>
<td>St. Alban</td>
<td></td>
<td>0.21</td>
<td>0.19</td>
<td>-</td>
<td>4.6</td>
<td>0.46</td>
</tr>
<tr>
<td>4</td>
<td>Drammen</td>
<td></td>
<td>0.21</td>
<td>0.21</td>
<td>-</td>
<td>1.1</td>
<td>0.34</td>
</tr>
<tr>
<td>5</td>
<td>Canons park</td>
<td></td>
<td>0.47</td>
<td>0.96</td>
<td>-</td>
<td>8.4</td>
<td>0.22</td>
</tr>
<tr>
<td>6</td>
<td>Bothkenner</td>
<td>Titi and Wathugala (1999)</td>
<td>0.40</td>
<td>0.17</td>
<td>-</td>
<td>2.9</td>
<td>0.33</td>
</tr>
<tr>
<td>7</td>
<td>Drammen Stasjon</td>
<td>Augustesen (2006)</td>
<td>0.22</td>
<td>0.82</td>
<td>-</td>
<td>1.2</td>
<td>0.32</td>
</tr>
<tr>
<td>8</td>
<td>Nitsund</td>
<td></td>
<td>0.16</td>
<td>0.68</td>
<td>-</td>
<td>14</td>
<td>0.16</td>
</tr>
<tr>
<td>9</td>
<td>Sky-Edeby</td>
<td></td>
<td>0.40</td>
<td>0.11</td>
<td>-</td>
<td>4</td>
<td>0.32</td>
</tr>
<tr>
<td>10</td>
<td>Haga</td>
<td></td>
<td>0.18</td>
<td>0.41</td>
<td>-</td>
<td>7.3</td>
<td>0.22</td>
</tr>
<tr>
<td>11</td>
<td>Algade</td>
<td></td>
<td>0.25</td>
<td>1.34</td>
<td>-</td>
<td>9.7</td>
<td>0.18</td>
</tr>
<tr>
<td>12</td>
<td>Motorvegbru</td>
<td></td>
<td>0.25</td>
<td>0.65</td>
<td>-</td>
<td>1.1</td>
<td>0.32</td>
</tr>
<tr>
<td>13</td>
<td>Sumatra</td>
<td></td>
<td>0.40</td>
<td>0.35</td>
<td>-</td>
<td>2.3</td>
<td>0.28</td>
</tr>
<tr>
<td>14</td>
<td>Cowden</td>
<td></td>
<td>0.15</td>
<td>1.36</td>
<td>-</td>
<td>25.2</td>
<td>0.16</td>
</tr>
<tr>
<td>15</td>
<td>ISU2</td>
<td>Ng (2011)</td>
<td>0.15</td>
<td>0.89</td>
<td>1.81</td>
<td>1</td>
<td>0.10</td>
</tr>
<tr>
<td>16</td>
<td>ISU3</td>
<td></td>
<td>0.10</td>
<td>1.24</td>
<td>1.44</td>
<td>1.6</td>
<td>0.05</td>
</tr>
<tr>
<td>17</td>
<td>ISU4</td>
<td></td>
<td>0.15</td>
<td>1.45</td>
<td>1.30</td>
<td>1.2</td>
<td>0.15</td>
</tr>
<tr>
<td>18</td>
<td>ISU5</td>
<td></td>
<td>0.18</td>
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Figure 6.11: Verification of proposed regression models in order to predict A factor: 
a) models set-1, b) models set-2, and c) models set-3.
7 SUMMARY, CONCLUSIONS AND FUTURE RESEARCH

7.1 Summary and Conclusions

Piles and other deep foundations are usually used to support super-structures to transfer
the load deeper stratus to increase the foundation bearing capacity and to reduce settlement.
Investigating pile capacity by experimental study is expensive in time and money. For this
reason, numerical techniques using finite element method have been implemented in many
géotechnical engineering studies, especially for simulating pile installation and the following
setup phenomenon. For the sake of scientific development, the research presented in this
dissertation was aimed at investigating pile installation and pile setup using the FE numerical
method, and studying the soil behavior during and after pile installation, by adopting an
appropriate elastoplastic constitutive model. First, a volumetric cavity expansion was created
in the soil body using prescribed displacement applied to the FE nodes at the pile-soil
interface. The pile was then installed inside the cavity, the prescribed boundary condition
released and the interaction between pile and soil surfaces was activated. Additional vertical
penetration was then applied to the pile until the steady state condition in the soil body was
reached. An elastoplastic constitutive model for saturated cohesive soils that can capture the
actual soil behavior of surrounding soil during pile driving and subsequent loading, was
developed. This model was formulated based on disturbed state concept (DSC) and critical
state (CS) theory, and is therefore called CSDSC model. The developed CSDSC model was
implemented in Abaqus software, using a Fortran User-Defined Material (UMAT) code. In
order to assess pile setup in saturated cohesive soils and to obtain an increase in pile capacity
over time after end of driving (EOD) using the numerical simulation techniques, the
consolidation theory and the thixotropic behavior of soil particles were adopted to the soil
body after EOD. During pile driving and creation of volumetric cavity expansion, excess
porewater pressure will be developed around the pile. The dissipation of excess porewater pressure (or consolidation) will result in an increase in the effective stresses and hence an increase in the pile resistance. The remolded soil particles (especially in fine grain soil) during pile penetration tends to rearrange and regain its strength (fully or partially) after EOD even under constant porewater pressure and water content. This phenomenon which is referred as thixotropy, was modeled in this study using a time-dependent reduction factor β(t) that was applied on the soil-pile interface friction coefficient μ and the critical state parameter $M$. In order to verify the FE simulation and results, five full-scale case studies (Bayou Lacassine Bridge site, Sabin River case study, Bayou Zouri Bridge site, Bayou Bouef Bridge site, and Baton Rouge Cajun site) were simulated using the FE model described in this dissertation. Since the developed model and numerical simulation techniques presented in this study were verified using different case studies, and demonstrated the ability to estimate pile setup, an extensive parametric study was conducted to evaluate the contribution of the different soil properties in pile setup phenomenon and to develop statistical regression models for evaluating pile setup in clayey soils. The soil properties that were selected to evaluate the setup rate factor $A$ introduced by Skov and Denver (1988) are: soil plasticity index (PI), undrained shear strength ($S_u$), coefficient of consolidation ($C_v$), sensitivity ratio ($S_r$), and over-consolidation ratio (OCR). Typical soil and pile geometries were selected to conduct the FE parametric study. More than 100 different actual soil properties were collected from the literature and used in FE simulation to calculate the corresponding $A$ factor. Based on findings of this research study, the following conclusions can be drawn:
• Adopting the vertical pile movement after pile placement in volumetric cavity expansion in order to simulate pile installation effects allows for accurate mobilization of the shear-induced porewater pressure and the pile tip resistance.

• The adopted FE technique in this study for pile installation in cohesive soils demonstrated its capability to capture the actual pile installation effects such as excess porewater pressure generation, displacement and shearing in the soil adjacent to the pile, and variation in the stress state due to the pile installation.

• Verification of the proposed CSDSC model using triaxial test results performed on clayey soils showed that the CSDSC model is able to predict the behavior of both normally consolidated and overconsolidated soils. This model is successful in overcoming the deficiencies of the conventional modified Cam-Clay (MCC) model, which has only two more model parameters than the MCC model.

• The results of numerical simulation of full-scale test pile case studies are in good agreement with field measurements. This demonstrated that the FE model adopted in this study is an appropriate method for modeling the pile and soil and soil-pile interaction behaviors during pile installation, and predicting the following pile setup phenomenon. In addition, the obtained results demonstrated that the use of the combination of both consolidation and thixotropic effects in the soil body can simulated pile setup phenomenon more accurately.

• The FE parametric study indicated that the setup rate factor \( A \) is directly proportional to the soil plasticity index, \( PI \), and sensitivity ratio, \( S_r \), and inversely proportional to the soil shear strength, \( S_u \), vertical consolidation of coefficient, \( C_v \), and over-consolidation ratio, \( OCR \).
Based on values of the $A$ factor for each set of soil properties obtained from the FE parametric study for individual soil layer, nonlinear multivariable regression analyses were conducted in order to develop mathematical relations between the $A$ factor and the different soil properties. The regression analyses were performed in four different phases, in which different number of soil properties were selected as independent variables in each phase. The conducted analyses yielded several regression models; however, the most accurate models were selected and grouped into three sets of equations (set-1, set-2 and set-3) based on the correlation coefficient and least square of prediction errors.

Verification of the abovementioned three regression model sets, using data available in the literature for additional sites, indicated that all the three models were able to reasonably estimate the setup behavior of individual cohesive soil layers; especially for soils with the setup factor $A$ greater than 0.10. The models of set-1 demonstrate better accuracy than the models of set-2, which are a little more accurate than the models in set-3 in estimating the setup factor $A$.

7.2 Future Research

This dissertation was an attempt to reduce expenses in deep foundation design through adopting numerical simulation technique. The numerical simulation can still be questioned due to the lack of exhaustive research in verification of case studies. Based on the study presented in this dissertation, the following recommendations for future research studies are made:

- Including thixotropy concept in the pile setup phenomenon was introduced in this dissertation for the first time, which includes some assumptions. Therefore, verification of this concept in different cohesive soils requires extensive field and
laboratory experimental study to evaluate the extent to which these assumptions are reasonable in various soil environments. Furthermore, some literature indicates that adding salt or cement to the remolded soils may increase their rate of thixotropy and the final gained strength value after long time might be higher than its unremolded strength values. This idea is worth investigating through a comprehensive experimental study.

- All case studies simulated in this study focused on single pile installation and the following setup; while in many cases piles are driven in groups with a close spacing resulting in interaction between them. Therefore, evaluating setup of piles within group would be a valuable idea.

- This dissertation focused on modeling pile penetration as a static problem. Dynamic simulation of this phenomenon would be an extension of this research and might improve the predictive capability of the model.
8 REFERENCES


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9 APPENDICES

9.1 Appendix A: Triaxial test results

Some triaxial test results performed in this study on the Cajun site clay samples are presented in this section. These results include the stress strain curves, stress paths, excess porewater pressure, and volume change in the soil sample during the consolidation step of the CU triaxial test.

Triaxial test results for Sample #1 corresponds to depth 67 feet (Undisturbed and disturbed sample results).
Triaxial test results for Sample # 3 corresponds to depth 75 feet (Undisturbed sample results).

Triaxial test results for Sample # 4 corresponds to depth 47 feet (Undisturbed sample results).
Triaxial test results for Sample # 5 corresponds to depth 53 feet (Undisturbed and disturbed sample results).

Triaxial test results for Sample # 6 corresponds to depth 58 feet (Undisturbed and disturbed sample results).
Triaxial test results for Sample #7 corresponds to depth 42 feet (Undisturbed and disturbed sample results).
Triaxial test results for Sample # 8 corresponds to depth 39 feet (Undisturbed sample results).
### Appendix B: Permissions for reprint

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VITAE

Firouz Rosti was born in Firoozabad, Fars, Iran, in 1979. He received his Bachelor of Science in Civil Engineering from Shiraz University, Iran, on August of 2002. He received his Master of Science degree in Civil Engineering from Urmia University, Iran, on May of 2005. He then immigrated to the United States in pursuing higher level of education. He expects to receive the Doctorate of Philosophy (PhD) in Civil Engineering in the area of geotechnical engineering from Louisiana State University in May 2016.