

Evaluating Pile Setup Using Numerical Simulation and Introducing an Elastoplastic Constitutive Model for Clays

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Abstract. An elastoplastic constitutive model was developed to define cohesive soil behavior. During pile installation in saturated ground, the soil adjacent to the pile disturbs causing large displacements and numerous variation in the porewater pressure in the soil-pile interface zone. Therefore, the soil disturbance and the corresponding decline in the soil shear strength were included in the developed constitutive model. After end of pile driving (EOD), the surrounding disturbed soil tends to regain its strength over time due to both consolidation and thixotropic effects. In this paper, the soil thixotropy was simulated by applying a time-dependent reduction parameter, β , which affects both the interface friction and the soil shear strength parameters. In order to examine the proposed model, numerical simulation of pile installation and the following increase in the pile capacity over time (pile setup) was performed for a full-scale pile load test case study. Finite element (FE) software Abaqus utilized to simulate the pile installation and following pile load tests. Dissipation of the induced excess porewater pressure was modeled through applying conventional consolidation theory. The proposed model was developed based on disturbed state concept and application of the modified Cam-Clay model. Pile installation was modeled by combination of two phases in an axisymmetric FE model: creating a volumetric cavity expansion followed by applying a vertical shear displacement (penetration). The FE simulation results included: (1)-developed excess porewater pressure in the soil body during pile installation and its dissipation over time after EOD, (2)-increase in effective lateral stresses at the pile-soil interface, and (3)-the pile setup values attributed to both the soil consolidation and its thixotropic responses. Comparison of the FE simulation results with the measured values obtained from load tests conducted on a full-scale instrumented pile indicated that the developed constitutive model is able to appropriately predict pile installation and following setup.

1 Introduction

In Geotechnical Engineering cases such as deep penetration, soil disturbance and remolding of the soil particle occur during shear loading and it significantly affect the general soil behavior. For engineering problems that involve deep foundations, soil

type usually changes over depth due to differences in the stress history. Therefore, in such cases, incorporating an appropriate constitutive model that can capture the actual behavior for both normally consolidated (NC) and over consolidated (OC) soils is necessary. There are several elastoplastic constitutive model available in the literature attempted to model the soil response under different loading conditions. Most of the developed constitutive models for clays are based on the critical state soil mechanics (CSSM) concept (Pestana and Whittle 1999). The modified Cam-Clay (MCC) model proposed by Roscoe and Burland (1968) is the most well-known critical state model, which is able to appropriately describe the isotropic NC clay behavior. Since the MCC model indicates elastic response inside the yield surface, its prediction for OC clay is poor (Likitlersuang 2003). Series of bounding surface models have been developed to overcome this deficiency (e.g., Dafalias and Herrmann 1986). The bounding surface plasticity concept was used later to develop the MIT-E3 model by Whittle (1993). The bounding surface plasticity has been developed to provide smooth transition from elastic to fully plastic state for soils under general loading. Application of the critical state models for heavily OC clays is limited, and it needs specific consideration. Yao et al. (2007) and (2012) introduced a unified hardening model using Hvorslev envelope to capture the heavily OC clay behavior. Linear and parabolic form of the Hvorslev envelope were adopted to adjust the conventional MCC model for heavily OC clay response under shear loads. Based on CSSM and bounding surface theory, Chakraborty et al. (2013a, b) developed a two surface elastoplastic constitutive model to define strain rate dependent behavior for clay. Chakraborty et al. (2013a, b) and Basu et al. (2014) used two-surface plasticity constitutive model for the clays, and it was implemented for analysis of shaft resistance in piles. Although, some of these models are able to describe both NC and OC clay behavior, but they usually include plenty of model parameters that requires performing several lab tests for their estimation. Furthermore, these models don't address directly the disturbance occurs in the soil body during shear loading in their elastoplastic formulation.

The disturbed state concept (DSC) developed by Desai and Ma (1992) is a powerful technique that is directly formulated based on the soil disturbance. In the DSC model, the soil response is obtained using two boundary (reference) state responses, which are named as relative intact (RI) state and fully adjusted (FA) or critical (c) state. The real or observed soil behavior is defined as a linear combination of RI and FA responses (Desai 2001).

Cohesive soils show certain degrees of thixotropic response under constant effective stress and constant void ratio. Thixotropy is defined as the “process of softening caused by remolding, followed by a time-dependent return to the original harder state” (Mitchell 1960). Thixotropy is a reversible process, which is mainly related to the rearrangement of the remolded soil particles, and it must be considered in constitutive models that deal with shear failure at the soil-structure interface such as driven piles, and the following increase in pile capacity with time after end of driving (or pile setup). Fakharian et al. (2013) described a reduction factor to incorporate soil remolding during pile installation in the numerical simulation. Barnes (1997) introduced a time-dependent exponential function to formulate inks thixotropic (strength regaining) response after remolding.

In this paper, a new constitutive model is developed and examined based on the combination of the critical state theory and the DSC [named as CSDSC model], which can describe behavior of both NC and OC soils as well as the soil disturbance caused by shear loading during deep penetration. In this study, thixotropic response of the clayey soils was formulated similar to the ink's thixotropy formulation presented in Barnes (1997). The proposed model requires only six model parameters, which it is less than the numbers of parameters of models that have been previously developed based on DSC. Furthermore, the proposed model predicts a smooth transition from the elastic state of the soil to the plastic state, which is usually observed during laboratory tests performed on soil samples.

2 Proposed Constitutive Model

The proposed model was developed in the framework of DSC formulation. Therefore, in this study, the relative intact (RI) state behavior was modeled using the MCC model. The fully adjusted (FA) response was defined based on CSSM concept, and it was assumed that the soil is in critical state when it becomes fully adjusted. Adopting critical state concept for both RI and FA behaviors imposes the proposed model to have two different values of critical state parameter (M) in order to describe the behaviors of RI and FA reference states: A value for intact material response (M_i) and a value for the fully adjusted response (M_c). The former one is not a real soil properties, and it can be specified based on the proposed model requirement (as will be described in the following sections). The latter one represents the critical state parameter of the soil, and it is obtained from laboratory test results. Since the proposed model was obtained by combining the DSC and the critical state MCC model, it is called the Critical State and Disturbed State Concept (CSDSC) model. In this model, the actual soil response in each load increment is related to the values of M_i and M_c using disturbance function, D with following equation:

$$M_a = (1 - D)M_i + DM_c \quad (1)$$

where M_a is the averaged (linearly combined) value for the M at each stage of loading process. The disturbance function D , which is related to the produced plastic strain in the soil body under shear is used to define the averaged response. The following exponential equation was proposed by Desai (2001), which relates soil disturbance D to the developed plastic strain in the soil body under applied load:

$$D = 1 - e^{-A \cdot \xi_d^B} \quad (2)$$

where ξ_d is the trajectory of deviatoric plastic strain dE_{ij}^p , defined as $\xi_d = \int \left(dE_{ij}^p \cdot dE_{ij}^p \right)^{1/2}$; and A and B are material parameters, which are obtained from results of laboratory soil Triaxial tests.

At the initial stage of shear, the soil is assumed to be undisturbed ($D = 0$ and $\xi_d = 0$), which means Eq. 1 yields to $M_a = M_i$. However, with the proceeding of the

applied load, the soil disturbs, the plastic strains develop in the soil body, the values of D and ξ_d increase, and eventually the D value approaches to 1. At this point, the soil reaches the critical state (i.e., $M_a = M_c$) condition.

M_c and M_i can be assumed to be constant, so the incremental form for Eq. 1 can be expressed as follows:

$$dM_a = (M_c - M_i)dD \quad (3)$$

From plasticity conceptual formulation and using the MCC model to represent the material response, the plastic multiplier λ^* can be defined as follows:

$$\lambda^* = \frac{\frac{\partial F}{\partial \sigma_{ij}} C_{ijkl}^e d\varepsilon_{kl}^i}{\frac{\partial F}{\partial \sigma_{mn}} C_{mnpq}^e \frac{\partial F}{\partial \sigma_{pq}} - \left[\frac{1+e}{\lambda-\kappa} p' p'_0 \frac{\partial F}{\partial p'} \right]} \quad (4)$$

where $d\varepsilon_{kl}^i$ is the incremental intact strain. On the other hand, for a specific yield surface F and in the case of associated flow rule used for plasticity formulation, the plastic strain increment is related to the deferential of the yield function F with respect to the stress tensor by the following equation:

$$d\varepsilon_{ij}^p = \lambda^* \frac{\partial F}{\partial \sigma_{ij}} \quad (5)$$

where λ^* is the plastic multiplier. The star sign in λ^* is used here for plastic multiplier to remove confusion. After mathematical manipulation incremental variation of D can be obtained as:

$$dD = \frac{\left(AB \xi_d^{B-1} e^{-A \xi_d^B} \right) \frac{\partial F}{\partial \sigma_{ij}} \cdot C_{ijkl}^e \left[\frac{\partial F}{\partial \sigma_{rs}} \frac{\partial F}{\partial \sigma_{rs}} - \frac{1}{3} \frac{\partial F}{\partial \sigma_{uu}} \frac{\partial F}{\partial \sigma_{vv}} \right]^{1/2}}{\frac{\partial F}{\partial \sigma_{mn}} C_{mnpq}^e \frac{\partial F}{\partial \sigma_{pq}} - \left[\frac{1+e}{\lambda-\kappa} p' p'_0 \frac{\partial F}{\partial p'} \right]} \cdot d\varepsilon_{kl}^i \quad (6)$$

Equation 6 indicates that the incremental change in the disturbance function in related strain increment of the intact material.

In the proposed model, MCC formulations run in each increment (or sub-increment). However, while the soil shears, the critical state parameter M evolves gradually from M_i value and toward M_c value depending on the amount of developed plastic strain in each increment and obeying the DSC theory. Figure 1 presents the formulation of the proposed model in the $p' - q$ space. The point A represents stress state at the beginning of strain increment $d\varepsilon_n$. The MCC model is used to solve the governing equations for $d\varepsilon_n$ using averaged critical state parameter, M_a^n , and the new stress state is obtained at point B , which is located on the yield surface a_n . Then, updated value for the averaged critical state parameter M_a^{n+1} is obtained from the incremental value of dM_a by using Eqs. 3 and 6 for use in the next increment. The imaginary yield surface i_{n+1} will then be defined using the updated critical state parameter M_a^{n+1} and the hardening parameter p_c^{n+1} (the prime index in p'_c removed for

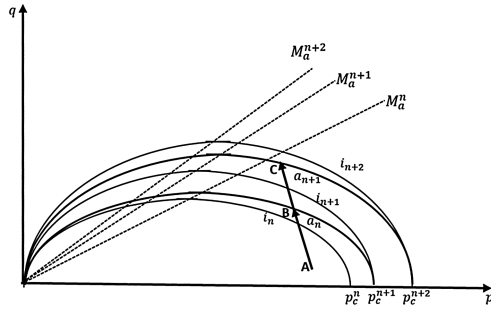


Fig. 1. Proposed (CSDSC) model representation in $p' - q$ space

simplicity). The current stress state (point B) is located inside the imaginary yield surface i_{n+1} , which causes the elastoplastic behavior for the material in the next steps until stress state reaches the critical state. The MCC model is then solved using the new strain increment $d\epsilon_{n+1}$ to reach point C and so on. The main advantage of this approach is the possibility of specifying a small value close to zero for M_i since the observed 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.

3 CSDSC Model Parameters

The proposed CSDSC model has six parameters, including the following four critical state (MCC) model parameters: (1) The Poisson ratio ν , (2) the slope of the critical state line M , (3) slope of the normal compression line λ , and (4) slope of unloading-reloading line κ ; and two parameters related to the disturbed state concept, defining the disturbance function D . The first four parameters can be obtained directly from laboratory tests (i.e., consolidation and Triaxial tests). The other two parameters namely, A and B define disturbance function D , and they can be obtained from Triaxial test results by some mathematical manipulation (Desai and Ma 1992).

4 Soil Thixotropy and the CSDSC Model

For cases like driven piles, which deal with change in the soil properties during different steps of installation and the following setup, it is necessary to adopt appropriate material properties at each step. Numerical simulation of pile setup using properties obtained from laboratory tests like Triaxial or consolidation tests on undisturbed soil samples yields unrealistic results. Therefore, the time-dependent reduction parameter $\beta(t)$ was applied in this study on the critical state parameter M and the soil-structure interface friction coefficient μ to incorporate the effect of soil remolding during pile installation and the following strength regaining with time after that:

$$M(t) = \beta(t)M, \quad \mu(t) = \beta(t)\mu, \quad (7)$$

Based on research performed by Barnes (1997) on the thixotropic strength regaining over time for inks, Abu-Farsakh et al. (2015) proposed the following definition for $\beta(t)$:

$$\beta(t) = \beta(\infty) - [\beta(\infty) - \beta(0)]e^{-\frac{t}{\tau}} \quad (8)$$

Where, the parameter t is time after soil remolding. $\beta(0)$ is the initial value for reduction parameter β immediately after soil shearing ($t = 0$), which its value depends on the degree of remolding occurs in the soil during shear. $\beta(\infty)$ is the β value after long time from soil disturbance ($t = \infty$); and τ is a time constant that controls the rate of evolution of β . Abu-Farsakh et al. (2015) related τ to the soil t_{90} , which is the time for 90% dissipation of the excess pore water pressure at pile surface.

In this study, a similar formulation to the disturbance function D (i.e., Eq. 2) was also proposed, which relates the initial reduction parameter $\beta(0)$ to the deviatoric plastic strain trajectory using the following exponential function:

$$\beta(0) = \beta_R + (1 - \beta_R)e^{-A\zeta_d^B} \quad (9)$$

where β_R is the β value for the fully remolded soil, which indicates a maximum reduction of the soil strength during shearing, and its value is related to the soil sensitivity. In order to reduce complexity, the disturbed state parameters A and B were used to introduce a relation between $\beta(0)$ and ζ_d in Eq. 9. Figure 2a and b present the schematic representations of the variations of D and $\beta(0)$ versus the deviatoric plastic strain trajectory, respectively. These Figures show that while the soil disturbs, the D value approaches unity, and the $\beta(0)$ yields to β_R by proceeding the plastic strain.

5 Verification of the Proposed Model

In order to verify the developed CSDSC model, elastoplastic formulation of the model was coded in Fortran computer language and it the was implemented in FE software, Abaqus, using a user defiend material (UMAT) subroutine. Two case studies including a consolidated undrained (CU) Triaxial test and a full-scale pile load test program were then simulated as discribed in following sections, and the obtained results compared with measured values from the experimental tests.

6 Case Study 1: Kaolin Clay

To verify the predictive capability of the proposed model, the results of laboratory Triaxial tests on Kaolin Clay performed by Yao et al. (2012) was simulated using the proposed CSDSC model. Three-dimensional model with a cubic porous element for soil specimen was used. The coupled porewater pressure analysis was used to define the multi-phase characteristic of the saturated soil. Triaixal stress state was applied

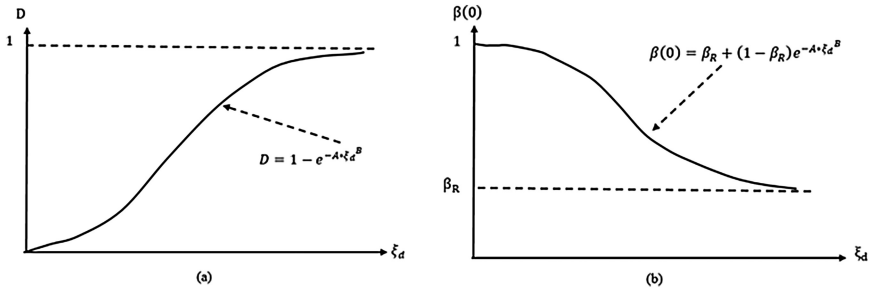


Fig. 2. Variation of soil characteristics during shear loading: (a) disturbance function D , and (b) soil strength reduction factor immediately after remolding, $\beta(0)$

using prescribed stresses for confining stress and using the prescribed displacement to apply deviatoric stress. The sample top surface was assumed to be free for drainage. The shear responses from undrained Triaxial compression test for different stress history ($OCR = 1, 1.20, 5, 8, 12$) were simulated. The four model parameters that are related to the MCC model were obtained from Yao et al. (2012). The remained two model parameters that are related to the disturbed state concept (i.e. A and B) were obtained from the Triaxial test results following the procedure explained in a previous section. The calculated parameters are presented in Table 1.

Table 1. Model parameters for Kaolin Clay used for implementation (Yao et al. 2012)

M	λ	κ	ν	A	B
1.04	0.14	0.05	0.20	14.43	0.47

Using the model parameters presented in Table 1, the FE model was run with MCC model and the results for different stress paths in the undrained condition are presented in Fig. 3a. The figure shows that the MCC model is not able to capture appropriately the actual soil response under undrained shear loads, especially for OC clays. In the proposed model, the strong capability of the CSDSC in modeling the actual behavior of soils was demonstrated, and the results of numerical simulation for different stress paths using the proposed model are presented in Fig. 3b. The figure clearly indicates that the proposed model can predict the actual soil behavior for both the NC and OC soils with good agreement. The model is also able to capture the strain softening behavior of heavily OC soils. Figure 4 shows the results of proposed model for stress-strain relations and excess porewater pressure generated during triaxial CU test at different over-consolidation ratios, which represents good agreement between the model prediction and measured lab results. In this figure, the stress values are normalized with respect to the initial pre-consolidation pressure p'_0 . Figure 4b shows that, for NC soil and lightly OC soil, the generated porewater pressure is positive, which is representation of soil contraction during undrained shearing. On the other hand, for heavily OC soils, the numerical simulation shows the generation of positive porewater pressure at the initial stage of the test followed by negative pore water pressure until

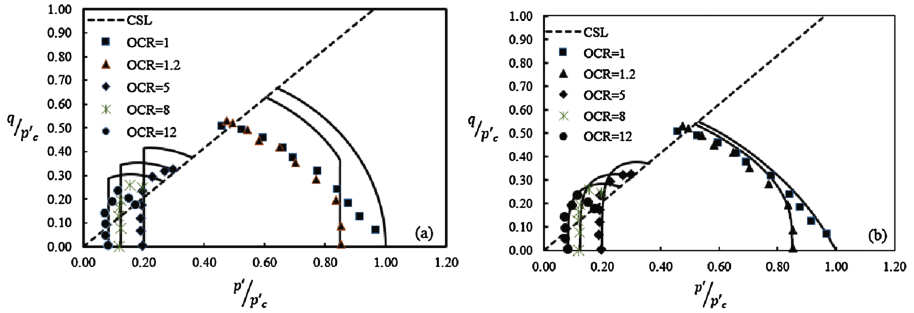


Fig. 3. Prediction of numerical simulation of undrained Triaxial test on Kaolin Clay (Yao et al. 2012) using (a) MCC model, and (b) proposed CSDSC model

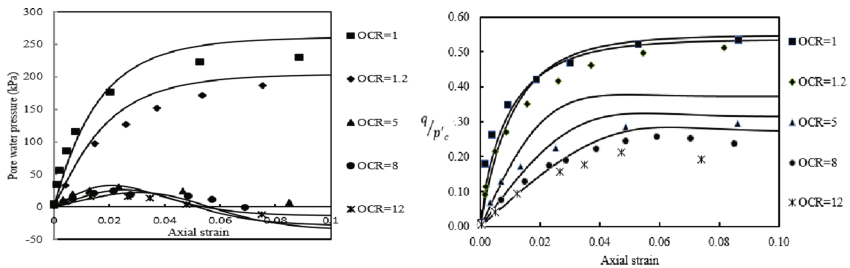


Fig. 4. Prediction of numerical simulation of undrained Triaxial test on Kaolin Clay (Yao et al. 2012) (a) excess porewater pressure, and (b) stress-strain curves

failure. This is an indication of soil dilative behavior, which is common in heavily OC soils. Based on the obtained results, the soil dilation in undrained condition increases by increasing OCR values.

7 Case Study 2: Full-Scale Pile Installation

The load test results for a full-scale instrumented test pile that was conducted at Bayou Laccasine Bridge site, Louisiana (Haque et al. 2014) was simulated using the proposed CSDSC model. The test pile was square concrete pile with 0.76 m width and a total length of 22.87 m. A 6.4 m long casing was installed and driven prior to pile installation to represent the scour effect at shallow depth. The test pile was fully instrumented with pressure cells, vibrating wire piezometers and sister bar strain gages that were installed at different depths of pile length, targeting specific soil layers. In addition, the surrounding soils were instrumented with nine multi-level piezometers located at the same depths as the pressure cells and piezometers installed at the pile's face. Both static load tests (SLTs) and dynamic load tests (DLTs) were conducted to obtain the pile resistance at different times after end of driving.

In this paper, the numerical simulation of the pile installation and following setup were performed using the Abaqus software and adopting the techniques described in Abu-Farsakh et al. (2015). The geometry of the soil and pile, the applied boundary conditions, and finite element mesh are shown in Fig. 5. Numerical simulation of pile installation was achieved by applying prescribed displacement to the soil nodes to create volumetric cavity expansion. The pile was then placed inside the cavity followed by applying a vertical penetration until the steady state condition is reached.

The subsurface soil condition at the pile site is mainly consists of clay soil, and the natural water table is 2.24 m below the ground surface. The subsurface soil domain was divided into eight layers based on the soil type and properties as presented in Table 2. In the table, w is the soil water content (%), S_u is the undrained shear strength (kPa), and K is the soil permeability (m/s).

The proposed CSDSC model was used to describe the elastoplastic behavior of the surrounding clay soil. The soil remolding during pile installation was incorporated in the constitutive model, and relating β_R to the soil sensitivity S_r with $\beta_R = (S_r)^{-0.3}$. This relation was depicted for Bayou Laccasine Bridge site based on available data for S_r and the pile resistance values obtained from field load tests, which yields a value of $\beta_R = 0.75$ (Haque et al. 2014).

Figure 6 represents the disturbance occurs in the soil immediately after pile installation for a typical horizontal path (path 1 in Fig. 5), which was obtained from numerical simulation using the CSDSC model. The figure shows that β has its maximum value $\beta_R = 0.75$ for soil adjacent to the pile face and approaches unity at a radial distance equal to eight times the pile size. At the same time, the disturbance function has a maximum value ($D = I$) at the soil-pile interface, and it approaches to $D = 0$ at a radial distance equal to eight times the pile size along the same path.

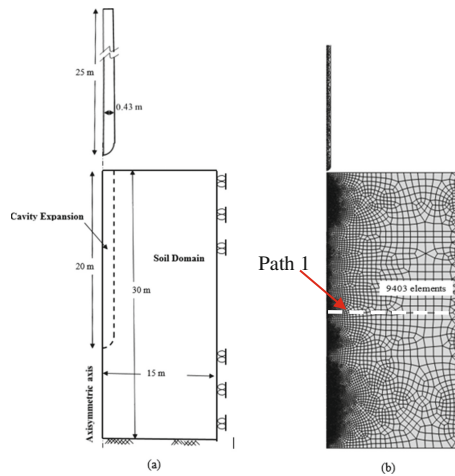
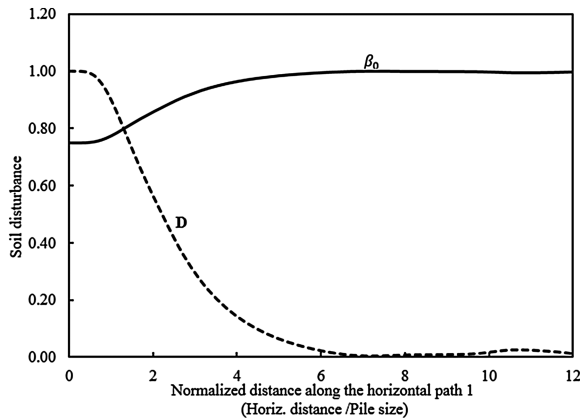


Fig. 5. Numerical simulation domain: (a) geometry and boundary conditions and (b) FE mesh (Abu-Farsakh et al. 2015)

Table 2. Soil material parameters for the test pile site (Abu-Farsakh et al. 2015)

Layer No.	Depth (m)	w (%)	S_u (kPa)	OCR	M	λ	κ	K (m/s) 10^{-9}
1	0–6.40	21	120	4	0.61	0.104	0.035	3.80
2	6.40–7.60	26	72	2.5	1.17	0.100	0.029	4.20
3	7.60–10	25	68	2	0.90	0.091	0.026	0.62
4	10–11.60	29	104	1.7	0.90	0.108	0.035	0.12
5	11.60–13	23	94	1.45	0.62	0.108	0.035	7.60
6	13–16	52	150	1.40	1.12	0.147	0.061	8.90
7	16–20	24	112	1.3	0.92	0.100	0.030	0.17
8	20–23	29	101	1	0.93	0.056	0.013	0.66

**Fig. 6.** Variation of β and D for a typical horizontal path in soil body immediately after pile installation

The numerical simulation using CSDSC model was compared with the measured field test results of the pile load tests. Figure 7a shows the comparison between the predictions of unit shaft resistance one hour after end of driving obtained using the CSDSC model and the measured values from the field load tests. The cumulative values of shaft resistance obtained from numerical simulation were also compared with the calculated values obtained from field tests, and the results are shown in Fig. 7. These figures clearly indicate that the CSDSC model is able to predict the pile resistance appropriately. For more verification, the increase in pile capacity after end of driving (or pile setup) was obtained from numerical modeling using the CSDSC model, and the model predictions were compared with the measured values from field load test results as shown in Fig. 8. The figure demonstrates that the proposed model is able to simulate pile setup with the model predictions of the pile resistance are slightly over-predicted the measured values.

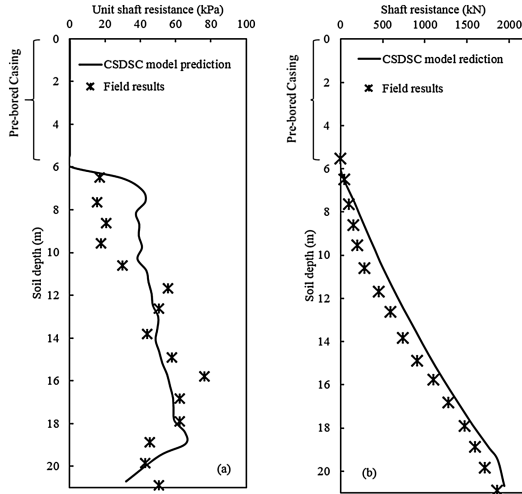


Fig. 7. Comparison between the proposed CSDSC model predictions with measured values from field test results (a) unit shaft resistance, and (b) total shaft resistance

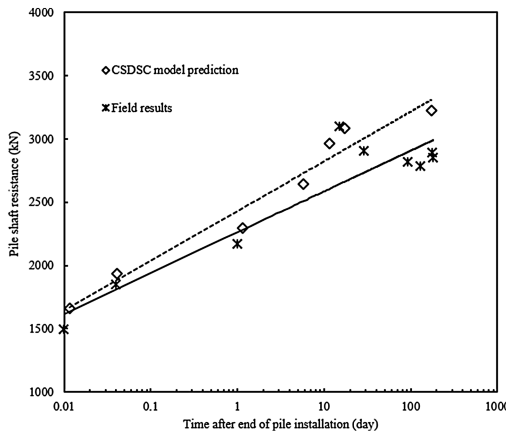


Fig. 8. Comparison between the proposed CSDSC model prediction and field measurement for pile setup

8 Summary and Conclusions

In this paper, an elastoplastic constitutive model for clay soil was developed and evaluated in application of pile installation and the following setup over time. The proposed model is based on the combination of Disturbed State Concept (DSC) and critical state concept of Modified Cam-Clay (MCC) model, which is referred as the CSDSC model. In the CSDSC model, the disturbance function D was applied to the critical state parameter M to adopt disturbed state concept. The soil remolding behavior

was related to the state of deviatoric plastic strain developed in the soil body during shear loading for use to simulate deep penetration problems such as pile installation and the corresponding remolding of surrounding soil. The soil thixotropic response was incorporated in the pile setup phenomenon using a time-dependent function, which increases exponentially with time after end of pile driving. The proposed model was implemented in Abaqus software via a user defined subroutine UMAT. The responses of Kaolin Clay under undrained monotonic loads was simulated for verification. Furthermore, a full-scale instrumented pile driven in Louisiana clayey soil and the following setup were simulated using the CSDSC model, and the results obtained from FE model and those measured from field test results were compared. Based on the results of this study, the following conclusions can be made for the numerical simulation using the CSDSC model:

- a) The developed CSDSC model has only six parameters, which is less than the previous elastoplastic models developed based on DSC, which makes it more effective in geotechnical engineering applications.
- b) The proposed CSDSC model predictions were compared with laboratory Triaxial test results, which show that the model was able to appropriately capture the undrained shear responses for NC and OC clays.
- c) The steep changes in stress paths inside yield surface, which is normally observed in MCC model, could be vanished in CSDSC model providing a smooth transition from elastic to plastic response. In other word, the proposed model is able to capture the smooth transitional behavior of the soil from elastic to elastoplastic and then to fully plastic states, which is usually observed during experimental tests performed on clayey soils.
- d) Numerical simulation of a full-scale pile installation and following pile load tests after end of driving indicated that the proposed CSDSC model is able to simulate pile installation and capture the soil disturbance, soil thixotropy and pile setup appropriately.
- e) The results demonstrated good agreement between the prediction of pile resistance and pile setup using CSDSC model and the measured values obtained from full-scale pile loads tests results.

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