Numerical modeling of pile installation effects on stress state in clay

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ABSTRACT

During installation of prefabricated piles in saturated clayey soils, excess pore water pressure is generated around the pile shaft and tip. The induced excess pore pressure is important to evaluate the stress state variations in clay and the subsequent pile bearing capacity. The main objective of this paper is to present a finite element numerical model to simulate a mini-cone installation (representing pile) and subsequent pore water pressure dissipation with time. The effect of OCR on soil stress state is also investigated. The numerical model is validated using the results of a physical model in which a mini-cone is penetrated into a consolidation chamber containing saturated clay. The results show that higher OCR develops less excess pore water pressure during installation. It is shown, however, that radial and vertical effective stresses as well as the coefficient of lateral earth pressure increase with increasing OCR during the excess pore water pressure dissipation process.

Keywords: pile installation, numerical modeling, soil stress state, clay, OCR.

1. INTRODUCTION

Penetration of piles in saturated clays leads to significant changes in the main state variables of the surrounding soil during and after installation. The stress state variations results in significant changes in the pile capacity with time which is of great importance in engineering practice.

In common numerical modelling of pile foundations, the true installation phase is often not taken into account. In most of the cases, the pile or the pile group is placed in a pre-bored hole in the soil. The obvious shortcoming of this kind of approach is that the effects of the stress changes due to pile installation are not clear (Fakharian and Khanmohammadi, 2012; Lee at al., 2010). Some have tried to model pile installation effects through specifying a coefficient of lateral earth pressure different than \( K_0 \), for example higher values than \( K_0 \) for driven piles (Baars and Niekerk, 1999; Broere and Tol, 2006; Said et al., 2009). The well-documented results of pile load tests are necessary for reliable inclusion of the installation effects. To overcome such problems, some researchers have carried out numerical modelling for simulation of pile installation or CPT cone insertion (Abu-Farsakh et al., 1998; Dijkstra et al., 2011; Pucker and Grabe, 2012; Sheng et al., 2005, Tolooiyan and Gavvin, 2011).

This paper presents a numerical model to analyse the cone (or pile) penetration in cohesive soils. In addition, the paper deals with the details of the distribution of the excess pore water pressure induced by cone penetration in clay with wide ranges of OCR. The numerical model in this paper is validated using the physical modelling results of a mini-cone penetration carried out by Kurup et al. (1994).

2 NUMERICAL MODEL

This section explains the procedures used to simulate the penetration of a pile using ABAQUS/Standard (Hibbitt et al., 2012). Because of generation of excess pore water pressure during pile installation in saturated clays and subsequent dissipation with time, a numerical model based on the coupled theory of nonlinear porous media has been used to simulate the installation. Numerical simulations are carried out during different installation stages of the pile: (1) pile installation, and (2) subsequently soil consolidation.

All the models are built in two steps. In the first step, the gravity load is applied to the soil to establish the initial stress states prior to the pile installation. Such initial stresses are controlled by the given soil unit weights and lateral stress coefficients, \( K_0 \). In the second step, the pile is pushed into the soil to the desired depth by prescribed displacements.

The material behavior of clay is simulated using the modified Cam-Clay model. An isotropic elastic model with Poisson’s ratio 0.25 and elastic modulus of 20 GPa is used for the cone.
The model geometry is generated in 2D axisymmetric condition. Soil is modelled using a four-node axisymmetric element with pore pressure degrees of freedom (CAX4P in ABAQUS). Cone is modelled with CAX4. According to studies carried out by Sheng et al. (2005), the conical pile end is chosen with cone angle of 60º. A typical mesh used in the proposed model to simulate the mini-cone penetration to the embedment depth is shown in Fig. 1.

In ABAQUS, the contact algorithm is based on the concept of a master surface and a slave surface. The master surface pushes into the slave surface and contact forces are generated to prevent nodes penetrating into the master surface. The pile–soil interaction during pile installation is modeled by contact kinematics. Contact conditions between the two surfaces are governed by kinematic constraints in the normal and tangential directions. The normal stress at contact is either zero when there is a gap between the pile and the soil, or compressive when the pile is in contact with the soil. The frictional sliding at the pile–soil interface is modeled by the Coulomb friction contact law. The normal stress at the interface is proportional to the normal stress at the interface. When the shear stress is less than this maximum, no relative displacements (sliding) take place. When the shear stress reaches the maximum, sliding takes place in the direction of the shear stress.

To validate this numerical algorithm, a model was produced based on Test#1 carried out by Kurup et al. (1994). Some analyses were carried out to investigate the effects of OCR on the soil stress state. The analysis procedure and results are described in the following sections.

3 MODEL VALIDATION

In this section, one of the eight tests carried out by Kurup et al. (1994) is simulated using the numerical model. Tests are about penetrating a miniature piezocone in cohesive soil in a calibration system in the laboratory. Rate of penetration is 2 cm/s. Factors such as soil type, stress history and boundary conditions have been investigated in these tests.

The results of the numerical model are compared with the miniature piezocone penetration Test#1 conducted on normally consolidated cohesive soil specimens that were prepared in two stages: slurry consolidation in a consolidometer, followed by reconsolidation to higher stresses in the calibration chamber. Soil slurry was prepared by mixing 50% Kaolinite and 50% fine sand (by weight) named K50. The Atterberg limits of the soil mixture are shown in Table 1. The specimen was mixed at a water content equal to twice of the liquid limit. The specimen was consolidated against a backpressure of 138 kPa. Both vertical and horizontal effective stresses at initial condition for starting penetration are 207 kPa. Full details of the test procedure can be found in Kurup et al. (1994).

The modified Cam-Clay model is used to describe the plastic behavior of the clayey soil. Properties of soil used in the numerical model are presented in Table 2. The piezocone diameter is 11.3 mm, depth of penetration is 0.48 m and soil domain diameter and depth are 0.5 m and 1 m, respectively.

Table 1. Properties of Soil Mixture in lab test (Kurup et al., 1994).

| Soil     | Liquid limit (%) | Plastic limit (%) | Plasticity index (%) | Specific gravity (G,)
|----------|------------------|-------------------|----------------------|----------------------|
| Kaolinite| 54               | 28                | 26                   | 2.66                 |
| Fine sand| -                | -                 | -                    | 2.67                 |
| K50      | 30               | 16                | 14                   | -                    |

Table 2. Soil properties used in the numerical model.

| Property                              | value           |
|---------------------------------------|-----------------|
| Gradient of the normal compression line in $e$-$\ln(p')$ space, $\lambda$ | 0.11            |
| Gradient of the unloading and reloading line in $e$-$\ln(p')$ space, $\kappa$ | 0.024           |
| Gradient of the CSL in $p'$-$q$ space, $M$ | 1.2             |
| Coefficient of earth pressure at rest, $Ko$ | 1               |
| Density, $p$ (kg/m$^3$)               | 1800            |
| Void ratio, $e$                       | 1               |
| Permeability, $k$ (m/s)               | $5\times10^{-10}$|
| Overconsolidation ratio               | 1, 3, 5 & 10    |

During the piezocone test carried out in calibration chamber, excess pore water pressure (EPWP) dissipation has been observed after penetration. Figure 2 shows the distribution of pore water pressure around the piezocone at the end of installation in the numerical model. This figure indicates that the excess pore pressures at the tip and the side skin of the piezocone match quite well with the measured excess pore pressures of 550 kPa at the tip and the 625 kPa at the
shoulder of the piezocone as obtained by Kurup et al. (1994).

Figure 3 shows that the dissipation of excess pore pressure from measurements agrees well with the predictions.

4 OCR EFFECT ON SOIL STRESS STATE

In order to investigate the effect of OCR on the soil stress state during and after cone installation, a set of analyses is carried out using the numerical model with four different OCRs (i.e. OCR=1, 3, 5 and 10). The basic characteristics of the model were described in Section 3 and the effects of OCR are investigated here.

As mentioned, excess pore water pressure is generated in clay during penetration of piezocone. Excess pore water pressure generated at the end of penetration along the pile shaft for all OCR values are shown in Fig. 4. Results indicate a sharp increase in PWP just around the cone and a significant decrease in PWP (even smaller than the initial PWP) slightly below the cone tip. In other words, at a given point in the soil mass, PWP increases significantly when the cone tip penetrates to that specific point and decreases rapidly after passing the cone tip. It is noted, however, that smaller EPWP at the piezocone tip is generated with increase in OCR. Dissipation of EPWP with time is illustrated in Fig. 5. The PWP values were extracted at depth 25 cm of soil mass adjacent to the shaft (approximately half of penetration depth). It can be observed that the EPWP dissipates faster at higher OCRs.

Variations of radial and vertical effective stresses compared to initial radial and vertical effective stresses (\(\sigma_{r0}^\prime\) and \(\sigma_{v0}^\prime\)) along the shaft height are demonstrated in Fig. 6. Comparisons are made between the end of penetration condition and the time that 90% of EPWP dissipation is occurred. The plotted stress ratio profiles indicate that the increases in radial and vertical effective stresses due to installation are considerably higher in greater OCRs. However, due to the possibility of higher PWP dissipation in radial direction, the differences between EOD and 90% consolidation of the radial effective stresses are more significant compared to the vertical effective stresses.
Figure 7 shows the coefficient of lateral earth pressure variation with different OCRs. Coefficient of lateral earth pressure is defined by Equation 1.

$$K = \frac{\sigma'_r}{\sigma'_v}$$  \hspace{0.5cm} (1)

It is evident from Fig. 7 that coefficient of lateral earth pressure increases with OCR and also during the dissipation of EPWP.

What could be the practical implication of the presented results? It may be stated that the prefabricated piles driven in saturated clay induce EPWP and consolidate afterwards with dissipation of the EPWP, as is well-known in engineering practice. This process is usually called “soil setup”. The main finding here is that as the OCR of the clayey soil increases, both the effective radial and vertical stresses increase more significantly during the consolidation process after the pile driving. This could end up in higher shaft resistance after the so-called soil setup phenomenon for soils with higher OCR. More studies are required to further validate this observation, in particular because of the scale effects between the min-cone and prototype pile.

5 CONCLUSIONS

Pile installation in saturated clay is modeled using finite element method ABAQUS/Standard. Reasonable agreement between the numerical analysis results and laboratory test is indication of reliability of the numerical prediction. In addition, effects of OCR on soil stress state are evaluated. The most important findings of the study are summarized below:

1. The results show that the axisymmetric FEM numerical model has successfully captured the difficult task of the cone penetration and in particular the simulation of PWP variations and subsequent consolidation of the clayey soil.

2. Considerably higher PWP is generated for NC clay compared to other OCR values. With increase in OCR, however, EPWP keeps reducing during the cone penetration.

3. The rate of dissipation of EPWP is higher in soils with greater OCR.

4. Increase in radial and vertical effective stresses due to installation and subsequent dissipation of EPWP, is considerably higher in higher OCRs.

5. The increase in coefficient of lateral earth pressure during cone penetration has shown to be more significant in soils with greater OCR. Similar trend is observed during dissipation of EPWP.

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