Numerical procedure for predicting pile setup in clay

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ABSTRACT

This paper presents a numerical procedure based on the finite element method to predict pile set up in clay. The constitutive behaviour of clay has been modelled using the modified Drucker-Prager model with cap plasticity. The quality of the finite element mesh during pile installation has been preserved by adopting an Arbitrary Lagrangian-Eulerian (ALE) scheme. Applicability of the numerical procedure in simulating different life stages of a pile has been demonstrated by simulating pile installation, subsequent consolidation and loading. Increase in pile capacity following pile installation, which is known as pile setup in geotechnical practice, has been investigated by carrying out pile load tests at different degrees of consolidation following pile installation.

1 INTRODUCTION

Pile foundations are widely used to support structures over ground with inadequate bearing capacity. Although piles can be driven into the ground using a hammer system, due to the noise, in many onshore operations jacking of piles into the ground is the preferred method. In clay soils, pile capacity increases with time after installation and this is known as pile setup. Field observations have shown that pile setup is significant and in some cases, the capacity has increased up to twelve times the capacity at the end of installation. Normally pile foundations are expensive, therefore, consideration of pile setup during the design process will lead to significant cost reductions in projects involving pile foundations.

Technical literature gives many examples of pile setup observed in the field. Fellenius et al. (1989) showed that the capacity of steel pipe piles increased by 50% for the tests carried out after 21 days compared to load tests carried out one day after pile installation in sandy clay and silty sand. In sensitive overconsolidated Norwegian clay, Karlsruda and Haugen (1985) observed a 30% increase in pile capacity between 6 and 30 days. After jacking closed ended steel piles. Konrod and Roy (1987) observed a twelve fold increase in the capacity for piles installed in soft marine clay.

A major factor, which contributes to pile setup is dissipation of excess pore pressures generated during pile installation. With the dissipation of excess pore pressures, the effective stresses around the pile will increase and as a result, load carrying capacity of the pile increases with the progression of soil consolidation. Another factor that contributes to pile setup is thixotropy or increase in soil strength with time (Titi and Wathugala, 1999). However, in this paper, pile setup due to soil consolidation is discussed.

When designing pile foundations in clay, engineers use empirical correlations or their previous experience in practice. Currently available methods in predicting pile setup are based on field pile installation data and they are site specific and empirical. For example the guidelines provided in the American Petroleum Institute's API RP2A (Recommended Practice for Planning, Designing and Construction of Fixed Offshore Platforms), which has been widely used by offshore engineers are based on the field tests carried out at the Gulf of Mexico by Bogard and Matlock (1990) and Bogard (2001). Number of researchers used cavity expansion theory to simulate the pile installation (eg. Carter et al., 1979; Randolph and Wroth, 1979; Burns and Mayne, 1999). After establishing pore pressures and stress field around the pile, consolidation theory is applied to predict the decrease in pore pressure due to consolidation. However, these methods lack the ability in predicting increase in pile capacity during consolidation. To predict pile setup accurately, pile load tests should be carried out during consolidation subsequent to pile installation.

In this paper the finite element analyses are carried out to simulate different stages of the pile installation process. Large soil deformations are incorporated to the analysis using an adaptive finite element procedure in the commercial finite element program ABAQUS/Standard. First the pile is jacked into the ground. Next the soil is allowed to consolidate. At different stages of the consolidation, pile load tests are carried out to determine the pile setup due to consolidation of the soil. Results are presented for piles with diameters 0.25 m and 0.5 m, and a length of 8 m.

2 NUMERICAL PROCEDURE

A numerical procedure based on the coupled theory of nonlinear porous media has been used to investigate the increase in pile capacity over time for piles installed in saturated clay. Numerical simulations have been carried out during different life stages of piles: (i) pile installation, (ii) subsequent soil consolidation and (iii) pile load tests, using ABAQUS/Standard. Since pile installation and subsequent pile load tests incur large soil deformations, an Arbitrary Lagrangian-Eulerian (ALE) procedure has been used to preserve the quality of the mesh throughout the analysis. A volumetric mesh smoothing algorithm has been used to adjust the position of nodes after each increment during the analysis, which prevents excessive deformation of elements while preserving the initial topology of the mesh.

The finite element discretisation of the soil domain is shown in Figure 1. In ABAQUS, only linear elements can be used with the Arbitrary Lagrangian-Eulerian procedure. Therefore, 2880 four-node rectangular elements with pore pressure degrees of freedom have been used to model the soil. Soil domain has been extended twenty pile diameters in radial direction away from the pile wall and twenty pile diameters away from the pile base in the vertical direction. This will avoid the influence of boundaries on numerical predictions. Deformation of the pile has been ignored in this analysis and assumed that it behaves as a rigid body.

Constitutive behaviour of the soil is modelled using a modified version of the Drucker-Prager model with cap plasticity. The yield surface of this model in p-t plane, where p is the mean stress and t is the deviatoric stress measure (ABAQUS Handbook, 2006), includes a shear failure surface, F_s, and a cap, F_c, which intersects the mean effective stress axis as shown in Figure 2. The transition surface, F_t, provides a smooth transition surface between shear and cap failure surfaces. This model uses an associated flow in the cap region and nonassociated flow in the shear and transition regions. Soil properties used for the cap plasticity model are given in Table 1, and in addition to those parameters, a hardening rule, which relates position of the cap as a function of inelastic volumetric strain, has been used.

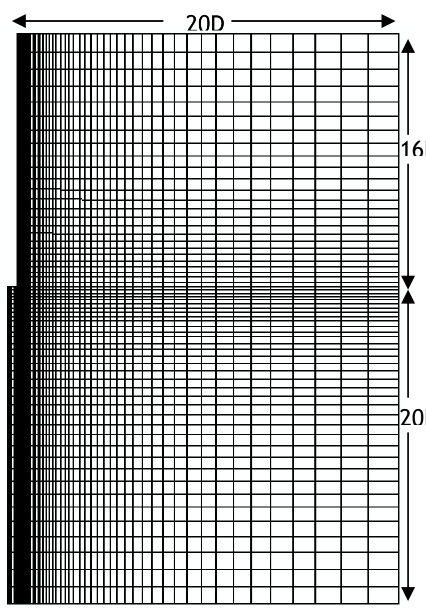

Figure 1. Finite element mesh for soil.

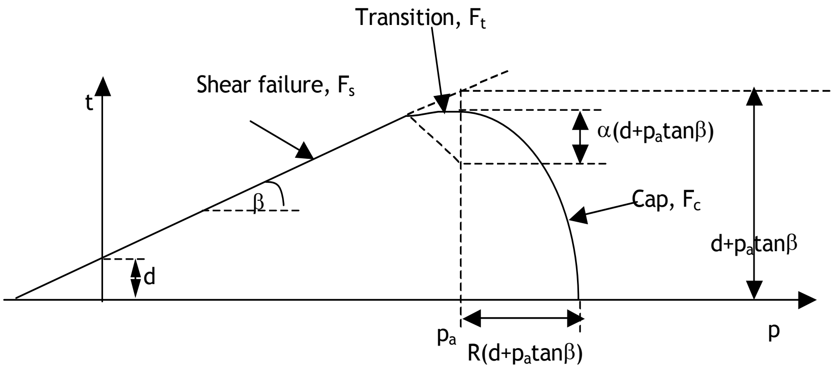

Figure 2. Modified Drucker-Prager model with cap plasticity.

Table 1. Soil properties used for the cap-plasticity model.

Property	value
Material cohesion, d (kN/m^2)	15
Angle of friction, β	25
Cap eccentricity, R	0.1
Initial yield surface position (plastic volumetric strain, $\varepsilon_{vol}^{pl}\big\|_0$)	0.002
Transition surface radius, α	0.01
Flow stress ratio, K	0.9
Young's modulus, E (kN/m^2)	$3\text{x}10^4$
Poisson's ratio, ν	0.42
Coefficient of earth pressure at rest, K_o	0.6
Density, ρ (kg/m^3)	1940
Void ratio, e	1.5
Permeability, k (m/s)	$2\text{x}10^{-10}$

3 RESULTS

Number of researchers (eg., Desai, 1978; Titi and Wathugala, 1999) used cavity expansion theory and strain path method to instate the stresses and pore pressures around the pile due to installation, before carrying out the consolidation analysis and pile load tests using the finite element method. In the present study, the numerical procedure described in Section 2 has been used to generate the stresses and pore pressures due to pile installation. During the first stage, pile is jacked into the ground. Analysis continued until the load carrying capacity of the pile is reached. During this phase 0.5 m diameter pile has penetrated 38 mm into the ground within 19 seconds.

Figure 3 shows the excess pore pressure field developed during pile installation at the vicinity of pile toe. It shows that significant pore pressures will develop due to pile installation, up to a region of about ten pile diameters away from the pile. Behind the pile toe, due to the shearing of soil, a negative pore pressure region is developed.

The second stage is the soil consolidation subsequent to pile installation. Consolidation analysis is continued until the steady state condition has been reached and excess pore pressures generated during pile installation have been fully dissipated. Figure 4 shows the pore pressure dissipation during the soil consolidation. The dissipation curves are similar to those obtained in the field by Bullock et al. (2005) and Pestana et al. (2002). The amount of excess pore pressure generation during pile installation increases with pile diameter. The pile with 0.25 m diameter reaches steady state conditions earlier than the 0.5 m diameter pile. With the dissipation of excess pore pressures generated during pile installation, effective stresses in the soil will increase as shown in Figure 5. Both vertical and radial effective stresses will increase significantly during the consolidation. Both stress components show more than a two-fold increase during the 1000 hour period analysed as shown in Figure 5. Due to the higher excess pore pressures generated for 0.5 m diameter pile during installation than for 0.25 m diameter pile, effective stresses around 0.5 m diameter pile is smaller than those around 0.25 m diameter pile.

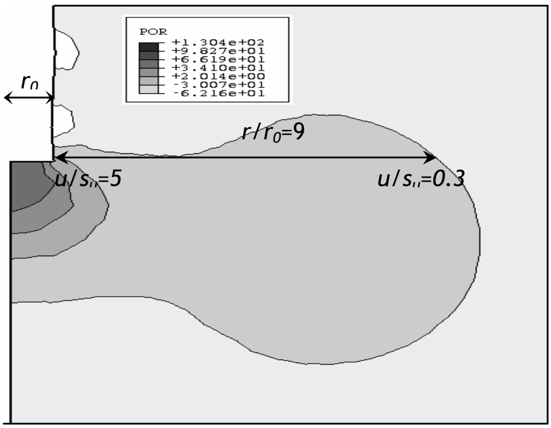

Figure 3. Pore pressure generation around pile.

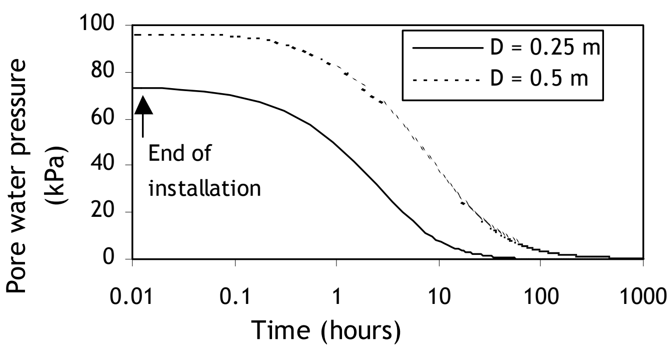

Figure 4. Variation of predicted pore pressure closer to pile toe with time after installation.

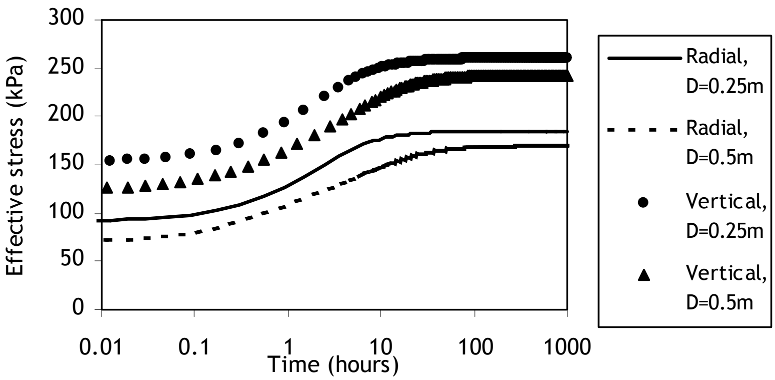

Figure 5. Effective stresses closer to pile toe during soil consolidation.

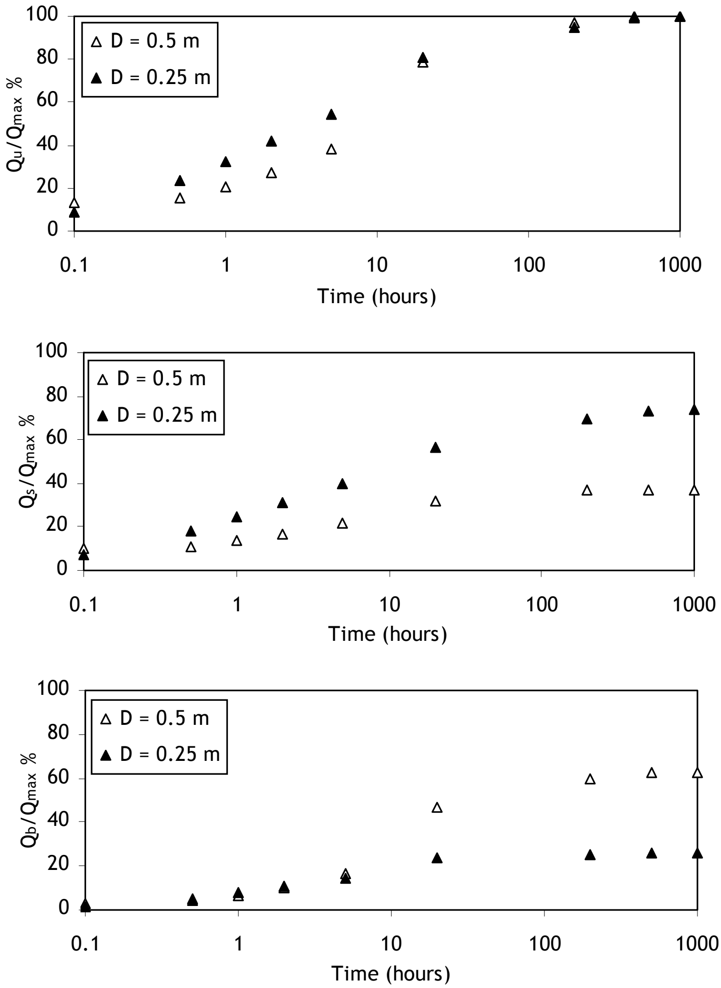

Figure 6. Increase in different components of pile capacity over time: (i) Total capacity, (ii) Shaft capacity and (iii) End-bearing capacity.

In order to investigate pile set up, static load tests were numerically simulated for two 8 m long piles with diameters 0.25 m and 0.5 m, during different stages of soil consolidation. Loading during the static load tests are simulated by applying a linear displacement with time at the head of the pile. Figure 6 shows the increase in total pile capacity, Q_u, shaft capacity, Q_s, and end bearing capacity, Q_b, over a 1000 hour period. Q_{max} is the load carrying capacity of the pile, 1000 hours after installation. At this time, soil consolidation is complete. Irrespective of pile diameter, both piles show a similar trend when pile setup is computed based on total load carrying capacity of the pile. During consolidation, load carrying capacity has increased ten times compared to the load carrying capacity at the end of installation. The curves of variation of pile capacity with time have an elongated "S" shape and this shape has been observed by other researchers during field studies (Titi and Wathugala, 1999).

For the 0.25 m diameter pile, the major increase in pile capacity is associated with the shaft capacity. However, for the 0.5 m pile, the major increase in pile capacity is associated with the end-bearing capacity. Therefore, it can be concluded that whether the pile setup is associated with shaft capacity or end-bearing capacity will depend on the length to diameter ratio of the pile.

The numerical examples demonstrated in this paper clearly shows that the proposed numerical procedure is successful in predicting pile setup. Therefore, this procedure has the potential to be used to develop design charts, after verification with field tests or centrifuge tests on full-scale piles.

4 CONCLUSION

A numerical procedure based on the coupled theory of nonlinear porous media has been outlined to predict pile setup subsequent to pile installation. An ALE procedure has been used to preserve the quality of the finite element mesh during the analysis. Soil behaviour is simulated using a modified version of the Drucker-Prager model with cap plasticity. To predict pile setup, nine pile load tests are simulated during different stages of the consolidation process. Results show that this procedure can be used to predict pile setup that occurs in full-scale piles. After validating with field or centrifuge tests, proposed procedure can be used to develop design charts to predict pile setup, where pile setup will be a function of soil properties and geometry of the pile. These design charts will be a useful tool for practicing geotechnical engineers.

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