NUMERICAL MODELLING OF PIEZOCONE PENETRATION IN CLAY

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Abstract

This paper presents results from finite elements analyses of cone penetration in clay. The software used is Abaqus/Standard (v. 6.13-2). The analyses are carried out adopting the Updated Lagrangian scheme (UL), which is necessary to account for large deformation and finite sliding effects for coupled consolidation analyses in Abaqus. The Updated Lagrangian scheme allows the cone tip to be advanced several diameters in order to achieve the steady-state condition. The undrained penetration in clay is modelled in two different ways: for the first model the soil is treated as a mono-phase material obeying a Tresca yield criterion, while the second is a coupled-consolidation analysis with pore fluid/effective stress elements.

1. Introduction

The Piezocone Penetration testing has become a useful tool for in situ investigation and geotechnical design. Cone Penetration Test (CPTu test) consists of pushing an instrumented cone tip into the ground at a constant rate. During the test the pressure required for tip penetration ($q_c$), the adhesion force between the sleeve and the soil ($f_s$) and the pore water pressure are measured. Thanks to measurement repetitivity, possibility to investigate a soil volume greater than that of a laboratory sample and possibility of getting continuous records, these tests are ideal to identify lithological variations and to provide the stratigraphic profile with excellent resolution.

The use of Piezocone Test to estimate the strength profile in soils requires reliable correlations between test results and soil engineering properties. Piezocone soundings are widely used in geotechnical engineering practice, using a standardized device geometry with a 60° conical tip and base radius $R = 1.78\,cm$, jacked into the ground at a constant rate of 2 cm/sec. At this rate and for this diameter, tests in clay soils can be considered fully undrained, tests in sands can be considered fully drained, while in the case of intermediate soils the test may be partially drained depending on the coefficient of consolidation $c_v$ (Randolph and Hope, 2004).

Numerical analyses of the cone penetration process has been conducted using the Strain Path Method (Baligh, 1985; Teh and Houlsby, 1991) and Cavity Expansion Analysis (Vesic, 1977; Salgado et al. 1997; Yu, 2004), as well as finite element analysis and finite difference analysis. The Strain Path Method proposed by Baligh models the mechanics of deep penetration in clays. It simulates both vertical and radial displacements of soil caused by the penetration process. The analysis assumes that the deformations caused by the penetration are obtained only as a function of the penetrometer geometry and independent of the shearing.
resistance of the soil. Stresses are then obtained using a soil constitute model and pore water pressures are computed from equilibrium. The theoretical analysis predicts that the tip resistance is a function of the undrained shear strength and the yield strain of the soil. The expression of the cone tip resistance is given by:

\[ q_c = N_{sp} \cdot s_u + \sigma_0 \]

where \( N_{sp} \) is the cone factor, \( s_u \) is the undrained shear strength obtained from triaxial tests and \( \sigma_0 \) the in situ octahedral stress. Typical values of cone factor \( N_{sp} \) can range between 8 and 14.

Conventional small strain finite element analyses are not suitable to determine the cone factor value since large displacements are necessary to reach a steady state stress field.

This paper presents results from two numerical studies performed to estimate the value of the cone factor for a normally consolidated clay. Updated Lagrangian finite-element analyses were performed using the software Abaqus/Standard (version 6.13-2) which utilizes an implicit solution scheme.

2. Numerical modelling aspects

2.1 Model characteristics

The model adopted is shown in Figure 1 together with the initial mesh. The radius of the penetrometer is \( R=18 \text{ mm} \), the width of the model is equal to 66\( R \) and the height is equal to 88\( R \). The cone penetrometer is treated as a rigid body (2D analytical surface). The tip is slightly rounded to avoid numerical instabilities due to the sharp edge between the cone tip and the shaft. As shown in Figure 1 the initial position of the penetrometer is above the soil surface. During the analysis the tip is pushed down in to the soil at a constant rate of 20 \( \text{mm/s} \). The final displacement reached in both cases is 210 mm.

As suggested by Mahutka et al. (2006) the left boundary of the model is offset 0.05 \( R \) from the axis of symmetry. The soil-tip interaction is modelled with a surface-base contact algorithm (finite sliding). In this analysis the contact interaction is assumed frictionless.

The undrained shear strength, \( s_u \), is equal to 20 kPa. The initial stress is considered uniform throughout the soil model. The initial vertical and horizontal stress is 35 kPa. The shear modulus \( G \) is 960 kPa.

The first model (model A) considers the soil as a homogeneous elastic-perfectly plastic material obeying a Tresca yield criterion with Poisson’s ratio equal to 0.49. The second is a coupled-consolidation analysis (model B), with the soil modelled using “hybrid” elements (pore fluid/effective stress elements), Poisson’s ratio equal to 0.25 and Tresca yield criterion.

For the second study drainage is permitted at the right and bottom boundary and the value of the initial pore pressure in the soil is equal to zero. The value of the hydraulic conductivity is \( k = 10^{-8} \text{m/s} \).

2.2 Adaptivity technique

In this study the Updated Lagrangian formulation, available in Abaqus/Standard, is used. Preliminary studies have been carried out using the Arbitrary Lagrangian-Eulerian scheme (ALE). Many strategies in terms of controlling parameters and mesh characteristics have been adopted, but it is necessary to point out that the ALE adaptive meshing technique available in Abaqus Standard doesn’t produce significant improvements in the mesh distortion.
The “mesh to mesh solution mapping technique” available in Abaqus/Standard consists in mapping a solution from a deformed mesh to another mesh of better quality. The new mesh is created using the mesh generation capability in Abaqus. The results from nodes in the old mesh are interpolated to the points in the new mesh. Subsequently the analysis continues as a new problem. In this study the total displacement of 210 mm is the result of 23 different analyses, the tip displacement before re-meshing is required, varies from 5 to 10 mm. Four-nodes elements, with displacement degrees of freedom for the first model and with displacement and pore pressure degrees of freedom in the second case, are used for the soil. Previous trials showed that these elements are more stable than higher order elements for coupled-consolidation analysis.

3. Analyses results

Figure 1 shows the model adopted for both the analyses, together with the shape of the initial mesh. The left boundary of the model coincides with the axis of symmetry. Figure 2 shows the shape of the mesh at the depth of 200 mm for both the analyses. The octahedral stress
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Contour at the final position of the tip is showed in Figure 3. Figure 4 shows the pore water pressure contour at the end of the last step of the analyses carried out for the model B.

The graph in Figure 5 shows the evolution with depth of the cone resistance, $q_c$, obtained for the analysis A, analogous results are available for the second model. The cone resistance is calculated as the ratio between the force required to push the penetrometer at the constant rate of 2 cm/s, and the tip section area. The normalized vertical displacement $d/R$ is the ratio between the vertical displacement, $d$, and the tip radius $R$. As shown in the graph, the steady state condition is reached after a vertical displacement of about $11R$.

Figure 6 shows the evolution of the cone factor for both models. The cone factor $N$, is obtained from the following equation:

$$q_c = N \cdot s_u + \sigma_0$$

where $s_u$ is the assumed undrained shear strength and $\sigma_0$ the value of the initial mean stress. The final value reached for both the analyses is $N = 10.6$.

Figure 2: shape of the mesh at the depth of 200 mm for the analysis A (left) and the analysis B (right).

Figure 3: Octahedral stress contour at the end of analysis A, tip displacement of 210 mm (the legend values are expressed in kPa).
Figure 4: Pore water pressure contour at the end of analysis B, tip displacement of 210 mm (the legend values are expressed in MPa).

Figure 5: Variation of cone resistance with penetration depth for Model A.
Figure 6: Variation of calculated cone factor with penetration depth.

4. Conclusions

The results obtained from the two models, static stress and coupled pore fluid/stress analysis, are in good agreement assessing a common value for the cone tip resistance and the cone factor.

Most of the numerical analyses available in literature assume either fully drained or undrained conditions, but the effect of consolidation is still not well studied. This study represents the first step to develop a methodology aimed at simulating the large strain penetration process under different drained conditions. For this purpose the Updated Lagrangian technique will be used for coupled-consolidation analysis, adopting more complex constitutive models for the soil.

References


