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Numerical modeling of cone penetration test in slightly overconsolidated clay with arbitrary Lagrangian-Eulerian formulation

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Abstract

In this paper the results of the cone penetration test (CPT) modeling with the arbitrary Lagrangian-Eulerian (ALE) formulation provided by Abaqus software package have been presented. The study compares the cone resistance and sleeve friction obtained in numerical analysis with values measured in soundings performed in the uniform layer of clayey soil in the Koszalin area. The clay layer was found to be slightly overconsolidated with OCR ranging from 3.5 to 4.5. The subsoil parameters used in the numerical model are based on laboratory tests data and the complementary CPT estimation. Evaluation of the most important factors influencing the numerical solution such as friction on the probe-soil interface and the undrained shear strength of the clay have been discussed. The possibilities of ALE method for soil parameters calibration are introduced and future challenges in large deformation problems modeling due to penetration issues are described.

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Keywords: Abaqus; arbitrary Lagrangian-Eulerian; ALE; CPT simulation; undrained analysis.

1. Introduction

The cone penetration testing (CPT) is a common geotechnical site investigation method to estimate soil strength parameters. In recent years many numerical studies, especially large deformation finite element calculations, have been performed to better understand the mechanisms which influence penetration resistance. Cone penetration into cohesive soil has been analyzed by Van den Berg [1], Abu-Farsakh et al. [2], Wei et al. [3], Sheng et al. [4] and

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others. However, in major part of the research only the cone resistance has been taken into consideration and the cone factor was investigated (e.g., [4,5]). In addition, the numerical model validation has been usually presented in comparison to the laboratory tests (e.g., [3]) and the verification with the field tests was not widely used. The main aim of this paper is to present the numerical simulation of the CPT with the influence of the friction between soil and probe in comparison with the field measurement. The secondary objective is the identification of CPT simulation sensibility to laboratory test data. The research performed is intended to shed more light on the cone-soil interaction and the possibility of CPT predictions based on laboratory test data.

The CPT numerical model is developed using Arbitrary Lagrangian-Eulerian (ALE) formulation with accordance to the total stress approach. The ALE formulation is widely used for calculating the cone penetration problems in homogeneous soil layer (e.g., [5]) and it provides better calculation efficiency due to the possibility of application of axisymmetric models in comparison to another numerical methods such as Coupled Eulerian-Lagrangian formulation where fully three-dimensional model is required (e.g., [6]). The geotechnical parameters of soil have been based on laboratory tests, but CPT correlations need to be used for initial lateral stresses estimation. Two friction coefficients for probe-soil interaction have been tested and their influence on the results has been presented. The numerical results are compared with field measurements including the cone resistance and sleeve friction. The interface friction influence is discussed and the possibility of geotechnical parameters validation with numerical CPT probing is also pointed out.

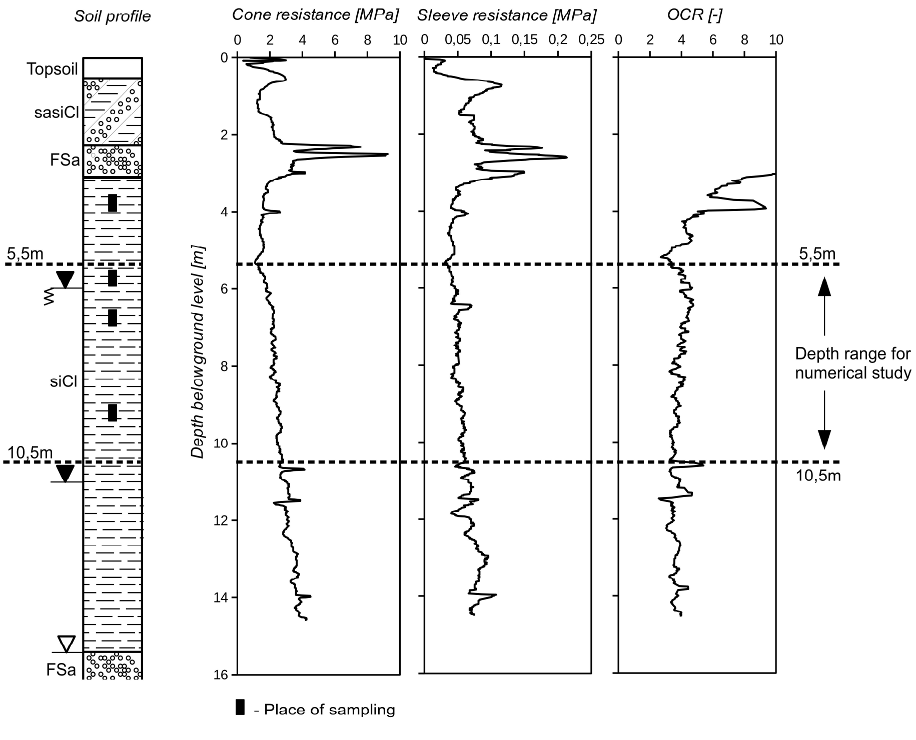

Fig. 1. Soil and CPT profiles for the WD-102 site.

2. Site geotechnical investigation

The WD-102 structure in the Koszalin area of the S6 highway currently under construction is selected as the reference localization. The geotechnical field investigation consists of one electrical CPT test done in clayey soil, one borehole drilled next to CPT point and two boreholes drilled in a distance of approximately 15 meters from the probing location. The laboratory tests include basic physical and mechanical soil parameters determination such as grain size analysis, consistency limits tests, oedometer tests, direct shear tests, pocket shear vane tests and pocket penetrometer tests. Geotechnical investigation have been commissioned by General Directorate for National Roads and Motorways in Poland and forwarded to authors for the purpose of this research. The CPT probing graph with borehole log and soil sample locations is presented in Fig. 1. The soil profile is standardized from all three boreholes situated close to CPT location and the depth range taken into consideration for numerical studies is chosen from 5,5 to 10,5 meters below ground level. The considered soil, a grey, stiff clay layer ($I_L \approx 0{,}1$) is slightly overconsolidated with OCR ranging with 3,5 to 4,5. The water table is stabilized 11,0 m depth below ground level, but the water seepage into the borehole was recognized at the depth of 6,0 m.

3. Geotechnical parameters estimation

The one-phase, total stress analysis performed with ALE is conducted with assumption of isotropic, elastic-perfectly plastic material and fully saturated soil. The first assumption is related to the linear elasticity and Tresca criterion, which are chosen for soil constitutive modeling. The laboratory study has shown, that soil saturation ratio for the considered depths is ranging between 93% and 97%, so the assumption of fully saturated soil can be successfully applied [7]. The soil physical and strength parameters are summarized in Table 1. The bulk soil density is equal to 2,23 g/cm^3 as determined directly from laboratory tests. The undrained shear strength estimates based on pocket instruments and the CPT correlations for two boundary cone factors are presented in Fig. 2. The calibration procedure with numerical direct shear and oedometer tests has been undertaken to find appropriate value of the undrained shear strength. Laboratory tests impose the effective stress approach where Modified Cam Clay (MCC) model have been used. The numerical oedometer and direct shear tests results have been compared with the laboratory ones and the satisfactory agreement within the error of 5% was reached. Consequently, the undrained shear strength c_u of 130 kPa was calculated from the MCC parameters (e.g. [8,9]):

$$\frac{c_u}{p_0'} = \frac{M}{2}\left(\frac{p_c'}{2p_0'}\right)^{(\lambda-\kappa)/\lambda} \tag{1}$$

where: c_u – undrained shear strength, p_0'- initial mean stress, M-stress ratio, p_c'- pre-consolidation mean stress, λ - logarithmic plastic modulus, κ – logarithmic elastic modulus.

The successful undrained pile installation modeling in terms of total and effective stress approach using correlation described with Eq. (1) has been previously shown by Konkol and Bałachowski [9]. The undrained elastic modulus estimation is based on assumption of volumetric and shear effects decomposition [10]:

$$E_u = 2G(1+\nu_u) \tag{2}$$

where: : E_u – undrained elastic modulus, G – effective shear modulus, ν_u – undrained Poisson's ratio.

The undrained Poisson's ratio fixed to 0,49 provides the soil incompressibility. The shear modulus G of 11280 kPa is based on calibration procedure with direct shear and oedometer tests. Finally, the undrained elastic modulus is calculated as E_u= 33600 kPa using Eq. (2).

The initial ratio between total horizontal and vertical stresses has been adopted after lateral earth pressure at rest coefficient K_0. K_0 is estimated as 1,0 from CPT measurements, which corresponds to slightly overconsolidated conditions (OCR ≈ 3,5 ÷ 4,5). The ratio between total horizontal and vertical stress (σ_{h0}/σ_{v0}) is calculated including pore water pressures. As lateral earth pressure at rest coefficient is equal to 1,0, the σ_{h0}/σ_{v0} ratio is also 1,0.

The shear behaviour in cone-soil interface with total stress analysis is usually expressed as a function of undrained shear strength [11]. However, this kind of contact is not available in Abaqus software. Consequently, the penalty contact algorithm and tangential behavior with prescribed coefficient of friction have been applied. As a practical workaround in this study two numerical runs have been performed with two coefficients of friction equal to

0,129 and 0,264, corresponding to the angle of interface friction δ as δ=⅓Φ' and δ=⅔Φ', respectively. As it will be shown in later part of this paper, the friction on probe-soil interface strongly influences the numerical solution.

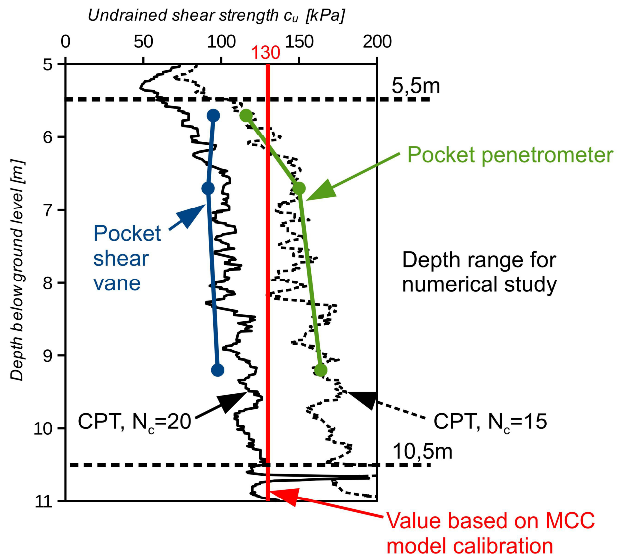

Fig. 2. Undrained shear strength estimates.

Table 1. Material parameters for CPT model (linear elasticity with Tresca plasticity).

Parameter	ρ (t/m^3)	E_u (kPa)	υ_u (-)	c_u (kPa)	σ_{h0}/σ_{v0} (-)
Value	2,23	33600	0,49	130	1,0

4. The Arbitrary Lagrangian-Eulerian model

The model geometry used in this study is presented in Fig. 3(a). The standard cone with diameter of 35,68 mm is modeled as a discrete rigid body and it is pre-installed at the depth of 6 cm in the soil to avoid initial mesh distortions [12]. Cone was pressed into the subsoil with so-called zipper-type technique developed by Mabsout and Tassoulas [13]. The rounding between cone and sleeve was designed to provide better soil flow around the probe and to minimize the sudden stress change around the vertex, which was observed when sharp chamfer is used [14]. The axisymmetric soil domain is 18 m height and 3 m wide and it is discretized with 102822, quadratic, 4 nodded, linear elements with reduced integration (CAX4R) with minimum size of 4 x 4 mm in the jacking area. The dimensions of soil domain are sufficiently large to minimize the boundary effects. The beginning of jacking was assumed to be 5,5 m below ground level, so overburden pressure was applied on the top of the soil domain, see Fig. 3(a). The probe was jacked to the depth of 10,5 m with a standard rate of 2 cm/s.

5. Numerical investigation results

The results of numerical simulation with ALE formulation are presented in Fig. 3(b). The test with δ=⅓Φ' was interrupted with technical problems shortly after the cone passed 9,5 m depth and this test could not be restarted due to restart file corruption. However, the results are still valid and can be used for the interpretation. As can be seen, a relatively good agreement in cone resistance between field measurement and numerical study is achieved and the influence of friction angle δ on the cone resistance is not significant. The numerical cone resistance curve is almost constant with depth and this observation is conformed with other numerical studies in homogeneous soil (e.g., [4,5]). The better coincidence of the results with field measurement is possible and it could be done by application of undrained shear strength increasing with depth. However, due to limited scope of laboratory tests (lack of triaxial tests in particular), more precise estimation of the undrained shear strength distribution is hampered.

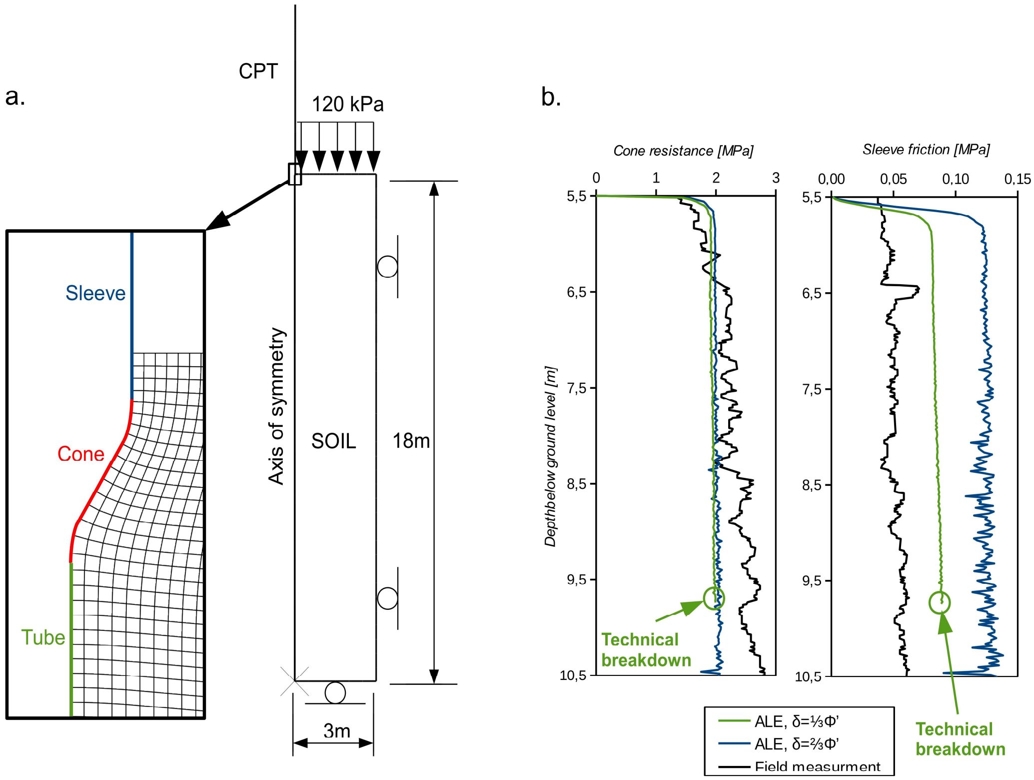

Fig. 3. (a) Axisymmetric CPT model with ALE formulation; (b) numerical results versus field measurement.

There is important difference between the numerical sleeve friction and the measured one. The reasons for this phenomenon can be twofold; the applied coefficients of friction are too high for the appropriate modeling of probe-soil interface behavior and there is a maximum shear stress on pile-soil interface, which can be determined from laboratory interface testing. As 0,129 is relatively low coefficient of friction in comparison to databases (e.g., [15]), presumably the lack of maximum shear stresses specification in numerical model is the significant reason for the sleeve resistance differences.

Table 2. Average friction ratios for cohesive soils in Koszalin area.

Soil	siCl[1]	Cl	sasiCl	ALE, δ=⅓Φ'	ALE, δ=⅔Φ'
R_f [%]	1,5÷2,5	4÷6	2÷4	~4,4	~6,2

The simulations performed have shown that contact behavior is essential for the proper modeling of shaft friction problem and, as a proof, the friction ratios from different CPT soundings in cohesive soils in Koszalin area are compared with the numerical study, see Table 2. The measured value of sleeve friction is overestimated 2 or 3 times

[1] Soil modeled in numerical study

in numerical analysis. As one can notice, the correct choice of the contact parameters on soil-structure interface is crucial and should be always treated individually.

6. Conclusions

The numerical modeling of CPT testing has shown that ALE formulation is a satisfying tool for large deformation problems calculations. The penetration of CPT was numerically modeled over 5 m depth which is equivalent to 140 cone diameters. The increasing numerical noise was observed for higher friction coefficient, especially in sleeve resistance. The calculated cone resistance is close to the field measurement regardless the interface friction, while sleeve friction strongly depends on probe-soil interface parameters. Relatively good performance of cone modeling in this research is consistence with other studies conducted in laboratory (e.g., [2]). Herein, the application of ALE method to the modeling of in-situ penetration test with geotechnical parameters estimated in majority in laboratory has been demonstrated. Overestimation of the sleeve resistance in numerical model suggests that the proper estimation of the soil interface behavior is crucial for the successful modeling and it should be based on laboratory investigation rather than literature database.

The numerical modeling of CPT probing is a promising research technique. It can be used as a verification tool in calibration tests where it can provide more detailed insight into the soil-structure behavior. However, the advantages of ALE technique and its efficiency can be limited by hardware capabilities and computational time. Finally, numerical CPT study may be extended to effective stress approach and pore water pressure measurement with dissipation test as well. This may help the modeling techniques to be improved and verified by the application of large deformation methods to the calibration of field tests.

Acknowledgements

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