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Numerical Simulation of CPT Cone Penetration in Sand

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Abstract. Numerical simulation of cone penetration in sand is performed by means of a computationally efficient critical state model implemented in an explicit-integration finite element code. Its main advantage, compared to other published studies employing simpler soil models such as the Drucker-Prager, is that sand compressibility can be described with a single set of model parameters, irrespective of the stress level and the sand relative density. Calibration of the constitutive model is based on back-analysis of published centrifuge tests results, and consequently the predictions of the numerical methodology are compared against independent tests. Additional analyses are performed for proposing a new simplified formula to correlate the cone penetration resistance with the *in situ* sand relative density.

Introduction

A number of studies on the numerical simulation of moving boundary problems in cohesionless soils have been published in the modern literature [e.g. 1, 2, 3, 4]. A common assumption among these studies is the simulation of the nonlinear response of sand with simple constitutive models, such as the Drucker-Prager. However, this has some limitations when it comes to the numerical simulation of the cone resistance developing during the CPT test, which depends chiefly on the effective stress level, but also the sand relative density and its compressibility characteristics [5, 6]. The latter two parameters related to sand response cannot be directly quantified with simple models such as the Drucker-Prager and the Mohr-Coulomb. Aiming at a more robust simulation of the cone penetration problem, by straightforwardly accounting the key parameters affecting resistance development, a new constitutive model was introduced by Kouretzis et al. [7]. This model balances between flexibility, simplicity and computational efficiency, with particular emphasis on the latter two aspects. The constitutive model has the same degree of flexibility when it comes to capturing different soil behavior as existing cone-cap [e.g. 8, 9] or bounding surface models [e.g. 10, 11], but with fewer material parameters and a better mathematical smoothness; rendering it ideal for the simulation of large-strain problems. Furthermore, its material parameters can be easily determined from conventional laboratory tests, such as triaxial tests. To simulate the boundary value problem of cone penetration, we implemented the constitutive model in ABAQUS/Explicit [12]. Here we present the calibration of the constitutive model parameters and the verification of the numerical methodology based on centrifuge test results in Fontainebleau sand of varying density, from five independent laboratories [6, 12]. The practical outcome of this study, resulting from a parametric analysis of sand density effect on the cone resistance, is the proposal of a new simple correlation between the sand relative density and the cone penetration resistance.

The Constitutive Model

The model to be introduced here is based on the versatile model of Yao et al. [14], which in turn has roots in the popular Modified Cam Clay model. The model has only 7 material parameters that are all associated with a clear (at least phenomenological) physical meaning, and thus can be directly determined from conventional laboratory tests:

1. The critical state friction angle of the soil, ϕ_{cr} (or the slope of the critical state line in p - q space under triaxial compression, M)
2. The maximum peak shear strength angle, ϕ_{max} (or the maximum possible shear strength under triaxial compression, M_{max})
3. The slope of critical state line in v - $\ln p$ space, λ_c
4. The slope of unloading line in v - $\ln p$ space, κ
5. The specific volume on the reference compression line when $p=1$ kPa, N_r
6. The specific volume on the densest compression line when $p=1$ kPa, N_d
7. The Poisson's ratio

The model employs two state parameters: χ_1 that depends on the current specific volume as well as the current stress state, and χ_2 that depends only on the initial specific volume. To preserve the limits of the presentation the model is not discussed herein, and the interested reader may refer to Kouretzis et al. [7]. Instead, we present model predictions in conceptual triaxial compression tests (Fig. 1), to exhibit the ability of the model to capture the density-dependent peak shear strength and volume change behaviour of sand.

Figure 1. (a) Drained and (b) Undrained conceptual triaxial compression tests on dense and loose sands.

Three dense sands with initial specific volumes $v_0=1.65\sim 1.86$ all exhibit a peak shear strength and volume dilation in the drained tests (Fig. 1a). Unlike the Modified Cam Clay model, the deviator stress q approaches the peak values smoothly, and the phase transition where volume contraction changes to volume dilation occurs before the peak deviator stresses. The magnitudes of the peak shear stresses and the volume dilation depend on the initial density of the sand. In the undrained triaxial tests (Fig. 1b), the shear strength again is density-dependent. A very loose sand with a specific volume of $v_0=2.05$ would exhibit a tendency to static liquefaction; with the deviator stress approaching zero as shearing continues. On the other hand, dense sands with $v_0=1.861\sim 1.762$ show a tendency of phase transition from contraction (positive excess pore pressure) to dilation (negative excess pore pressure). These generic results are in agreement with sand behaviour observed in triaxial tests, and although the model is significantly simpler, it compares well with the original model by Yao et al. [14], and the relevant laboratory test results.

Description of the Numerical Model and Validation Against Centrifuge Test Results

The finite element model was developed with the aim to replicate the 70g-centrifuge tests performed by Bolton and Gui [13] to study the effect of the relative density of the sand and of the container dimensions on the cone penetration resistance. Here we focus on the results of the experiments MWG5, MWG12, MWG9 and MWG10, which were back-analysed to calibrate the constitutive

model parameters. The former two were performed in very dense sand ($D_r=89\%$) and the latter two in medium sand ($D_r=54\%$), in containers of different sizes (Table 1). In addition to these four experiments, we present for comparison purposes two additional experiments; MWG8 performed in medium sand ($D_r=58\%$), and MWG11 performed in very dense sand ($D_r=76\%$).

Table 1. Selected experiments performed by Bolton and Gui [13] in Fontainebleau sand.

Test	MWG5	MWG8	MWG9	MWG10	MWG11	MWG12
Diameter of container (mm)	850	850	850	210	210	210
Height of specimen (mm)	350	351	348	358	351	342
Dry density, ρ (kg/m ³)	1663	1552	1538	1538	1613	1662
Void ratio, e	0.590	0.704	0.719	0.719	0.639	0.590
Relative density, D_r (%)	89	58	54	54	76	89
Rate of penetration (mm/sec)	3.6	3.5	3.5	3.7	3.7	3.7

The properties of the numerical model employed in the back-analyses resulted from an extensive study, investigating the effects on the results, run times, and numerical stability of several parameters, including: the extents of the simulated geometry; the finite element type, size and mesh grading; simulation of the penetration procedure and surface properties; constitutive model integration parameters; adaptive meshing technique and controls; etc. Details on this study are omitted. However, some aspects of the model are outlined here, as they may be of interest for readers engaged with the numerical simulation of similar problems.

Figure 2. (a) Finite element mesh employed in the analyses, and (b) Deformed finite element mesh for cone penetration 25D.

The problem of the cone penetration was simulated as axisymmetric, in model scale, considering a geometry of 0.14m x 0.35m (Fig. 2). Loukidis and Salgado [15] mentioned that in order to restrain the shear band thickness next to the cone shaft to realistic values, the minimum element size should be between 5 and $20D_{50}$, where D_{50} is the grain diameter at which 50% of the soil is finer. Grain diameter D_{50} for the Fontainebleau sand is $D_{50}=0.181\text{mm}$ [13] whereas the minimum element size adopted was 2mm ($11D_{50}$), thus within the acceptable range. The ratio of the model diameter, D , over the cone diameter, d , was $D/d=28$ and the total penetration of the cone tip reached 0.25m in model scale i.e. $25d$. A total of 8190 4-node reduced-integration elements (CAX4R) were used to simulate the sand inside the container. The "combined" viscous-stiffness hourglass control option was used, together

with the second-order accuracy formulation. The cone was modelled as a Lagrangian rigid surface with a diameter $d=10\text{mm}$ [13] and a standard apex angle 60° . Penetration of the cone into the sand is simulated by prescribing a vertical displacement on the reference node of the surface. The problem of cone penetration into the soil is, strictly speaking, not axisymmetric, as soil along the axis of symmetry will move laterally to accommodate the deformation imposed by cone penetration (Fig.1). Various techniques have been proposed to tackle this issue, such as slightly offsetting the left boundary from axis of symmetry or defining a pre-existing cavity at the area of the cone tip [e.g. 4]. A different, perhaps more robust technique was introduced in this study, resulting in enhanced mesh quality: A smooth rigid surface was defined, originating below the tip of the cone and extending below the bottom boundary of the model (Fig. 1b). This surface is moving together with the cone, thus preventing the leftmost soil elements to move inwards of the axis of symmetry but not outwards; allowing at the same time the cone surface to slide.

The problem was simulated dynamically in ABAQUS/Explicit, with a slow penetration rate of 0.25mm/sec , to avoid numerical issues related to the sudden application of high strain rates, and inertial effects. This rate was about 10 times slower than the actual rate of the tests (Table 3). The total penetration of the cone tip reached 0.25m in model scale i.e. 25 times the cone diameter. As the use of even small values of either fixed or variable mass scaling (a common technique to artificially increase the timestep in quasi-static analyses) proved to have an effect on the analysis results, another method was used to accelerate the solution to acceptable levels: Instead of applying a gravitational force, the geostatic stress field defined via a body force on the soil elements, taking advantage of the geometry of the problem. The applied body force magnitude was such as the geostatic effective stresses are equal to the centrifuge model, and horizontal stresses were calculated while considering the earth pressure coefficient at rest equal to $K_0=0.5$. Not including a gravitational force (geostatic stresses do not depend on density) allowed for increasing the soil density by 10,000 times, and a subsequent increase in the minimum time step, while keeping the penetration rate slow and avoiding the development of compressional waves that would be reflected on the fixed boundaries. Consequently, each full analysis required about 10^6 steps, and took 50~70hours to complete in a 4X6 core Xeon CPU's server.

To minimize element distortion during penetration, the Arbitrary Eulerian-Lagrangian (ALE) adaptive mesh option was employed. Maintaining the quality of the grid and avoiding excessive element distortion required regeneration of the mesh after each single analysis step (Fig. 1b). In addition, the curvature refinement parameter a_c [20] was set equal to zero, to maintain the desired element size and shape along the interface of the sand with the concave surface representing the penetrating cone. Friction at the interface between the cone and the sand was simulated with the classical Coulomb friction law. The friction coefficient at the cone-sand interface was assumed constant, and was determined from the back-analysis of centrifuge test results equal to $\mu=0.5$. Calibration of the constitutive model parameters (Table 2) was too based on the back-analysis of centrifuge test results, via two benchmark analyses for medium sand (relative density $D_r=54\%$, initial specific volume $v_0=1.72$) and very dense sand (relative density $D_r=89\%$, initial specific volume $v_0=1.59$).

Table 2. Constitutive model parameters used in the numerical analyses.

λ_c	$\square$	N_r	N_d	ϕ_{cr}	ϕ_{max}	Poisson's ratio
0.0085	0.006	1.89	1.58	30°	50°	0.333

Results of the centrifuge are presented in terms of cone resistance q_c versus the corrected prototype penetration depth, z_{pc} [14]. The corrected prototype depth is defined as $z_{pc}=z_p[1+z_m/(2R)]$, where z_p is the prototype penetration depth $z_p=N \cdot z_m$, z_m is the model penetration depth, $R=0.3755\text{m}$ is the average radius to the surface of the sand specimen measured from the central axis of the centrifuge rig [6], and N is test acceleration level in g's (here $N=70$). Results of the two benchmark numerical analyses mentioned earlier for medium sand and very dense sand are depicted in Fig. 3a, together with the results from the centrifuge tests mentioned in Table 1. A fair agreement with the centrifuge test measurements is observed, irrespective of the sand density and penetration depth.

Figure 3. Comparison of numerical results with (a) the tests results by Bolton and Gui [13], and (b) the test results from five independent laboratories presented by Bolton et al. [6].

Note that the numerical cone resistance values are plotted against the prototype penetration depth, z_p . Numerical cone resistance curves inevitably include some noise, due to the frequent remeshing. Further evidence on the validity of the presented numerical methodology is drawn from the comparison with similar centrifuge test results from five independent laboratories, reported by Bolton et al. [6]. These tests were performed again in Fontainebleau sand of average relative density $D_r=84\%$, but using different preparation methods, container sizes and cone diameters. Results of an additional analysis considering dense sand (relative density $D_r=84\%$, initial specific volume $v_0=1.61$) are compared with the independent test results in Fig. 3b. The comparison is made in terms of normalized cone resistance, $Q=(q_c-\sigma'_v)/\sigma'_v$, to be in line with Bolton et al. [6] presentation, where $\sigma'_v=\sigma_v=\rho \cdot g \cdot z_p [1+z_m/(2R)]$ for the centrifuge tests and $\sigma'_v=\sigma_v=\rho \cdot g \cdot z_p$ for the numerical analyses. Observe that numerical results are consistent with the reference centrifuge experiments; whereas the shape of the normalized cone resistance curve suggests that the model is able to effectively capture cone-sand interaction effects both above and below the "critical depth", where the transition from shallow foundation to deep foundation mechanism takes place [13].

Practical Application: Quantification of Sand Density Effects on the Cone Resistance

One of the main goals of Bolton and Gui [13] experimental study was to correlate the sand relative density with the cone tip resistance. Bolton and Gui suggested that the relation between the relative density and the maximum normalized cone resistance Q could be approached with a curve similar to the dashed one in Fig. 4, which was "not determinable" due to the lack of additional test data, and proposed a linear expression instead (Fig. 4).

Figure 4. Correlation of the relative density of the sand with the normalized cone resistance.

During this study, 15 additional parametric analyses were executed to fill in the gap in Bolton and Gui's measurements, considering sand relative densities within the range $D_r=20\%$ and $D_r=90\%$, commonly met in practice. If we plot the maximum normalized tip resistance derived from these analyses against the relative density (Fig. 4), we can define the non-determinable curve mentioned by Bolton and Gui. Perhaps not surprisingly, the curve resulting from the parametric numerical analyses matches well with the hypothetical curve drawn by Bolton and Gui. We can further proceed with the

proposal of a curve fitting expression (Fig. 4) to provide the in situ relative density of the sand as a function of the normalized cone tip resistance.

Summary and Concluding Remarks

The presented numerical methodology is based on a new constitutive model for the description of density-dependent peak shear strength and compressibility of sands, in order to account for the key parameters controlling tip resistance development during cone penetration: relative density, stress level and sand compressibility. Employing this model to estimate state-related *in situ* sand properties via parametric analyses does not aim to replace existing studies, which draw on large sets of experimental data from various sand types. Rather, it is shown that the methodology can effectively simulate moving boundary problems of practical interest, when properly calibrated to specific sand properties.

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