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Numerical simulation of cone penetration testing using a new critical state constitutive model for sand

George P. Kouretzis [a,*], Daichao Sheng [a], Dong Wang [b]

[a] ARC Centre of Excellence for Geotechnical Science and Engineering, Faculty of Engineering and Built Environment, The University of Newcastle, Australia
[b] Centre for Offshore Foundation Systems, The University of Western Australia, Australia

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ABSTRACT

A new perspective on the numerical simulation of cone penetration in sand is presented, based on an enhanced critical state model implemented in an explicit-integration finite element code. Its main advantage, compared to similar studies employing simpler soil models, 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 is based on back-analysis of published centrifuge experiments, while results of the methodology are also compared against independent tests. Additional analyses are performed to investigate sand state effects on cone penetration resistance, in comparison with empirical expressions from the literature.

1. Introduction

A number of studies have been published in the modern literature on the numerical simulation of moving boundary problems in cohesionless soils [e.g. 1–5]. A common assumption among these studies is the simulation of the nonlinear response of sand with simple constitutive models, such as the Drucker–Prager model. However, this approach has some limitations with regard to the numerical simulation of the cone resistance developing during the cone penetration test (CPT), which depends chiefly on the stress level, the sand relative density, and the sand compressibility [6,7]. The latter two parameters related to sand response cannot be directly quantified with simple models such as the Drucker–Prager and the Mohr–Coulomb models. On the other hand, the use of a very complex constitutive model to realistically simulate cone penetration in sand can be extremely challenging, keeping in mind that the problem involves high mesh distortion and frictional contact in finite element analysis.

Aiming at a more robust simulation of large deformation problems in sand, by straightforwardly accounting for the key parameters affecting resistance development, a new constitutive model is developed here. The model balances between flexibility, simplicity and computational efficiency, with particular emphasis on the latter two aspects. The constitutive model is expected to have about the same level of flexibility in capturing different soil behaviors as existing cone-cap models [e.g. 8–13] or bounding surface models [e.g. 14–20], but with fewer material parameters and a better mathematical smoothness, in order to render 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. The typical features of soil behavior considered here include those observed for sands with different densities, but also clays with different overconsolidation ratios. In this context, we exclude complicating features in soil behavior such as partial saturation, initial structure, anisotropy, rate dependency and hysteresis under cyclic loading. While these features are unquestionably very important, they are not involved in quasi-static moving boundary problems as the one we focus herein, and they can be considered separately without causing major changes to the model framework.

To simulate the boundary value problem, we implemented the constitutive model developed in the explicit module of the commercial finite element code ABAQUS [21], as described in the following paragraphs. Calibration of the constitutive model and verification of the results of the developed numerical methodology in based on centrifuge test results in Fontainebleau sand of varying density, from five independent laboratories [7,22].

A series of parametric analyses regarding sand state effects on the cone resistance is accordingly presented, and the results are compared against well-established empirical relations proposed for the estimation of the sand relative density, D_r and the state parameter ψ [23] from CPT measurements. The comparison

* Corresponding author. Address: EA Building, University of Newcastle, Callaghan, NSW 2308, Australia. Tel.: +61 2 4921 6449.

E-mail address: Georgios.Kouretzis@newcastle.edu.au (G.P. Kouretzis).

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suggests that the proposed methodology is able to capture the effects of sand density on large displacement problems, if properly calibrated to account for sand-specific parameters. Although the centrifuge experiments, as well as the numerical analyses, refer to dry sand only, the effect of the degree of saturation on the developing cone resistance has been found to be trivial [24–26], thus it can be argued that the outcome of this work applies to saturated sands too.

Apart from the above, a relatively detailed description of the numerical methodology is presented, as it could be of interest for researchers and practicing engineers working in the field of simulation of (very) large displacement problems in cohesionless soils.

2. The constitutive model

The sand model is based on the versatile model of Yao et al. [27] which in turn has roots in the popular Modified Cam Clay (MCC) model. The model of Yao et al. [27] uses two state parameters similar but not identical to the state parameter concept of Been and Jefferies [23] to capture the behavior of sands and overconsolidated clays, and it reduces to its base MCC model for normally consolidated clays. It also employs a complex hardening law to capture the density-dependent contraction/dilation behavior of normally- and overconsolidated sands. In this paper, the model of Yao et al. [27] is further simplified in terms of the hardening law, the compressibility and the three-dimensional generalization, without losing its versatility. The enhanced 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.

2.1. Density-dependent peak shear strength and compressibility

One of main features of the model is that the peak shear strength and the compressibility of a soil depend on its initial void ratio e_0. To characterize this dependency, we first define a number of straight lines in the v–$\ln p$ space, where v is the specific volume $v = 1 + e$ and p is the mean effective stress (Fig. 1). As in all critical state models, we assume there is a straight line representing the ultimate state where the volume of the soil remains constant with increasing shear strain. This line is the so-called critical state line (CSL) and its slope is assumed to be λ_c. We also assume that the unloading line (with decreasing mean effective stress) has a slope κ, irrespective of the initial void ratio or the initial stress. Another straight line representing the isotropic compression of the densest possible state of the soil is referred to as the densest compression line (DCL) and is assumed to have the same slope as the unloading line. We further define a reference compression line (RCL) that lies above the CSL, separated by a vertical distance of $(\lambda_c - \kappa)\ln 2$. This line is recognized as the normal consolidation line (NCL) in the MCC model, and is not the locus of points of maximum void ratio. We will demonstrate later that a soil with an initial void ratio on or above the RCL will exhibit only volume contraction, but below the RCL, both volume contraction and dilation.

A state parameter χ_1 is introduced to characterize the relative distance between the current specific volume and the corresponding specific volume on RCL:

$$\chi_1 = \max\left(\frac{v_\eta - v}{v_r - v_d}, 0\right) \tag{1}$$

where v is the current specific volume, v_r is the specific volume on the RCL under the same mean effective stress (Fig. 2), v_d is the specific volume on the DCL, and v_η is the specific volume on a line which is equivalent to the RCL but with a stress ratio of $\eta = q/p$, where q is the current deviator stress. The state parameter χ_1 becomes zero if the specific volume of the soil is on or above the RCL, but becomes equal to $\chi_1 = 1$ if v is on the DCL.

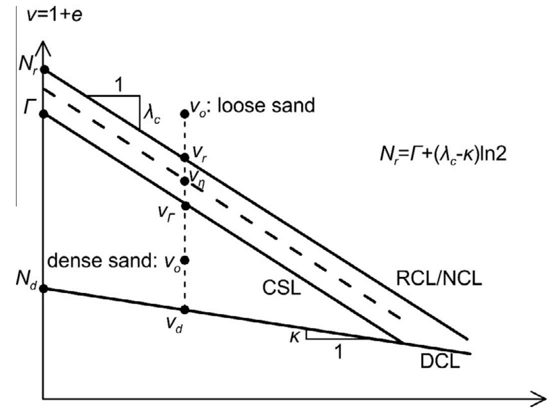

Fig. 1. Dilatancy of sand with different initial densities in the v–$\ln p$ plane [27].

The peak shear strength of a soil depends on its relative location between the RCL and DCL. If its initial state is on or above the RCL, its peak shear strength slope is the same as the ultimate critical state slope (M). If its initial state is on the DCL (the densest possible state), the peak shear strength slope reaches a maximum ($M_{\max}$). For any states between the RCL and DCL, the peak shear strength slope (M_f), varies between the slope M of the CSL and the maximum possible slope $M_{\max}$ for the densest possible state. Here we assume the following interpolation holds:

$$M_f = (M_{\max} - M)\sqrt{\chi_1} + M \tag{2}$$

Eq. (2) suggests that M_f becomes equal to M when $\chi_1 = 0$ for an initial state on or above the RCL and $M_{\max}$ when $\chi_1 = 1$ for an initial state on the DCL.

In the MCC model, a soil with an initial specific volume below the NCL (or RCL) behaves purely elastically until the mean stress reaches the preconsolidation pressure. However, the model discussed here does not feature a purely elastic region. Instead, a soil with an initial state below the RCL will also undergo elastoplastic deformation under loading, with its compressibility depending on its current specific volume. In this context, we further assume that the compressibility of a soil under isotropic compression varies between the maximum slope λ_c of the RCL and the minimum slope κ of the DCL, as:

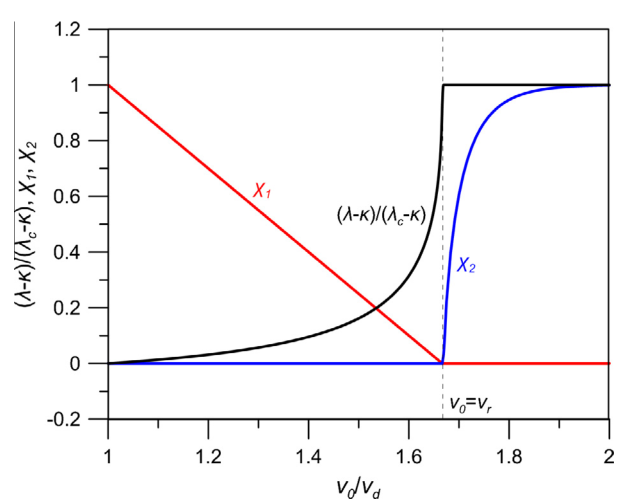

Fig. 2. Variation of the state parameters and compressibility with initial specific volume, v_0.