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Application of a finite deformation multiplicative plasticity model with non-local hardening to the simulation of CPTu tests in a structured soil

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Abstract

In this paper an isotropic hardening elastoplastic constitutive model for structured soils is applied to the simulation of a standard CPTu test in a saturated soft structured clay. To allow for the extreme deformations experienced by the soil during the penetration process, the model is formulated in a fully geometric non-linear setting, based on: i) the multiplicative decomposition of the deformation gradient into an elastic and a plastic part; and, ii) on the existence of a free energy function to define the elastic behaviour of the soil. The model is equipped with two bonding-related internal variables which provide a macroscopic description of the effects of clay structure. Suitable hardening laws are employed to describe the structure degradation associated to plastic deformations. The strain-softening associated to bond degradation usually leads to strain localization and consequent formation of shear bands, whose thickness is dependent on the characteristics of the microstructure (*e.g*, the average grain size). Standard local constitutive models are incapable of correctly capturing this phenomenon due to the lack of an internal length scale. To overcome this limitation, the model is framed using a non-local approach by adopting volume averaged values for the internal state variables. The size of the neighbourhood over which the averaging is performed (*characteristic length*) is a material constant related to the microstructure which controls the shear band thickness. This extension of the model has proven effective in regularizing the pathological mesh dependence of classical finite element solutions in the post-localization regime. The results of numerical simulations, conducted for different soil permeabilities and bond strengths, show that the model captures the development of plastic deformations induced by the advancement of the cone tip; the destructuration of the clay associated with such plastic deformations; the space and time evolution of pore water pressure as the cone tip advances. The possibility of modelling the CPTu tests in a rational and computationally efficient way opens a promising new perspective for their interpretation in geotechnical site investigations.

1. Introduction

The Cone Penetration Test with measurements of pore water pressure (CPTu) is a widely used investigation tool for the characterization of both coarse- and fine-grained soils, for its simplicity, reliability and its relatively low cost. The conventional interpretation of CPTu tests is typically based on empirical and semi-empirical correlations, the last based on very crude descriptions of soil behaviour, such as the adoption of a total stress approach to circumvent the difficulties associated to the evaluation of the excess pore water pressures induced by the total stress changes around the cone tip.
The aim of this work is to show that a more rational interpretation of the coupled deformation and flow processes occurring in the soil during a CPTu test is possible by resorting to the numerical solution of the relevant governing equations coupled with a realistic constitutive model for the soil. An effective strategy for modelling the penetration process during the execution of a CPTu test is the Particle Finite Element Method (PFEM, Onate et al., 2011), which is capable of dealing with the large displacements and deformations induced by the cone penetration process very efficiently. A key point in the PFEM computational strategy is the use of low-order elements (linear triangles or tetrahedra) with frequent *h*-adaptive remeshing, using algorithms based on extended Delaunay tessellation. The nodes of the spatial discretization are treated as material particles, the motion of which is tracked during the numerical simulation. Applications of PFEM to the modelling of CPTu tests have been reported, *e.g.*, by Monforte

et al. (2017) and Monforte et al. (2018). In these works, dealing with CPTu tests in clays, the soil has been modelled using the classical MCC model. Although this critical state model can capture the essential features of soft, lightly overconsolidated clays, it fails to reproduce the behaviour of natural structured clays, characterized by the presence of intergranular bonds of various origin. To investigate the effects of bonding on the soil response to the piezocone advancement, in this work the isotropic hardening elastoplastic model for structured soils proposed by Nova and co-workers (Tamagnini et al., 2002; Nova et al., 2003; Ciantia & Di Prisco, 2016) has been extended to finite deformations within the lines followed by Borja and Tamagnini (1998). The resulting finite deformation model – referred to as FD_MILAN model in the following – has been implemented in the geomechanics-oriented PFEM code G-PFEM (Monforte et al., 2017) and used to simulate a series of CPTu tests in a relatively soft, structured natural clay. An overview of the constitutive model for structured soils is briefly recalled in Sect. 2, while the details of the CPTu simulations program are given in Sect. 3. A selection of the results obtained is presented in Sect. 4, along with the main concluding remarks.

2. The FD_MILAN model

The FD_MILAN model is a three--invariants isotropic hardening model employing independent yield function f and plastic potential g. A representation of the yield surface in the Kirchhoff stress invariants space Q:P, for the set of constants of a typical clay, is given in Fig. 1.

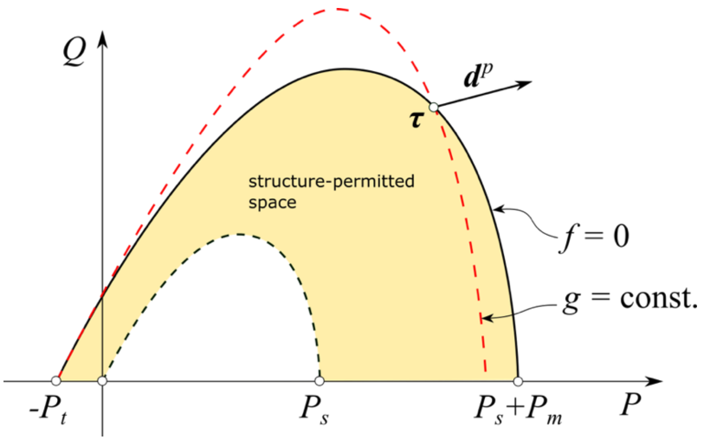

Figure 1: Yield surface and plastic potential of the FD_MILAN model.

Two independent scalar internal variables account for the hardening/softening effects due to volumetric and deviatoric plastic strains (*preconsolidation pressure* P_s) and for the effects of intergranular bonds (*bond strength* P_t). A positive value of P_t results in an expansion of the elastic domain in stress space, an increase of the isotropic yield stress in compression by a quantity $P_m = k\,P_t$, with k a material constant, and in the presence of a true cohesion and a non-negligible tensile strength. As a consequence of the microstructural rearrangement of the soil fabric – accompanied by macroscopic plastic deformations – the intergranular bonds are progressively destroyed and P_t reduces accordingly. When $P_t \to 0$, the structure-permitted space reduces to zero and the yield surface coincides with the intrinsic yield surface of the unstructured soil. The hardening laws for P_s and P_t are given by:

$$\dot{P}_s = \dot{\gamma}\,\rho_s P_s\big(\hat{V} + \xi_s \widehat{D}\big) \tag{1}$$

$$\dot{P}_t = -\dot{\gamma}\rho_t P_t\big(\big|\hat{V}\big| + \xi_t \widehat{D}\big) \tag{2}$$

where: $\hat{V} = \mathrm{tr}(\partial g/\,\partial\boldsymbol{\tau})$; $\widehat{D} = \sqrt{(2/3)}\|dev(\partial g/\partial\boldsymbol{\tau})\|$; $\dot{\gamma}$ is the plastic multiplier; $\boldsymbol{\tau}$ is the Kirchhoff stress, and ρ_s, ρ_t, ξ_s and ξ_t are material constants. For $\xi_s = 0$, eq. (1) reduces to the classical volumetric hardening law of critical state soil mechanics. Plastic volumetric compaction produces an increase of P_s while plastic dilation is accompanied by a reduction of P_s. As the RHS of eq. (2) is always negative, any plastic process will always cause a reduction of P_t (material destructuration).

To provide a characteristic length scale to the constitutive equation, regularizing the numerical solution in presence of strain localization phenomena, an integral non-local approach has been adopted. The internal variables P_s and P_t are treated as non-local, spatially averaged quantities over a neighbourhood Ω of the material point. The size of this neighbourhood is controlled by the *characteristic length* l_c, a material constant. Further details of the FD_MILAN model can be found in Oliynyk *et al*., (2021).

3. PFEM simulation program

A standard piezocone, with radius R = 1.78 cm and a cone tip angle of 60° is inserted in a calibration chamber with radius B = 0.45 m and height H = 1.05 m, filled with a fully saturated clay. The problem has been assumed as axisymmetric. The piezocone is wished-in-place at an initial depth z_0 = 0.25 m and then displaced downwards at a constant penetration speed of 2.0 cm/s, up to a depth $z = z_0 + 20R$. The piezocone tip and its lateral surface are modelled as rigid, impervious surfaces, and a smooth contact interface with the soil is employed to simulate the piezocone-soil interaction as the device penetrates the soil. Given the relatively small dimensions of the calibration chamber, the self-weights of the pore water and of the soil have been ignored. The initial pore water pressure has been assumed uniform and equal to zero. The material constants adopted for the clay soil are reported in Tab. 1. The flow rule has been assumed as associative, so the plastic potential coincides with the yield function.

Table 1 Set of material constants adopted in the CPTu test simulations.

$\hat{k}$ (-)	G_0 (MPa)	α (-)	P_{ref} (kPa)	$M_{f,c}$ (-)	α_f (-)	μ_f (-)	$M_{g,c}$ (-)
1.82	3.0	0.0	5.0	1.1	0.75	1.5	1.1
α_g (-)	μ_g (-)	ρ_s (-)	ρ_t (-)	ξ_s (-)	ξ_t (-)	k (-)	l_c (mm)
0.75	1.5	8.33	15.0	0.0	3.0	5.0	5.0

All simulations have been performed as fully coupled hydromechanical problems, adopting the mixed $\mathbf{u}$-Θ-p_w formulation of Monforte et al. (2017). The bottom and lateral surfaces of the calibration chamber have been assumed as rigid, impervious and perfectly rough boundaries. At the top surface of the soil body a uniform normal pressure q_0 = 100 kPa and a constant pore pressure p_w = 0 have been imposed. Consistently with these boundary conditions, the initial Cauchy effective stress in the soil mass has been assumed axisymmetric, with components σ_z = 100 kPa and $\sigma_r = K_0\sigma_z$ and K_0 = 0.5.

4. Selected PFEM results

Six different simulations have been performed considering different values of the hydraulic permeability, k_h, in the range between 1.0e-6 m/s and 1.0e-9 m/s, and initial values of the bond strength P_t in the range between 0 and 60 kPa. Fig. 2 reports some selected results from the simulations performed with k_h = 1.0e-9 and the two extreme values of the bond strength. For such a low permeability value, the penetration process occurs in almost undrained conditions.

From the contours of accumulated deviatoric plastic strains E_s^p it can be observed that in the unbonded soil (top row) the plastic zone around the piezocone extends by about 3R around the shaft and the cone, with contour lines following the shape of the penetrating device. In the bonded soil (bottom row), on the other hand, the observed pattern of plastic shear deformations is much more irregular and show the presence of localized shear zones which originate at about 3R below the cone tip and bend upwards as the cone advances. The presence of localized shear zones is a consequence of the softening mechanism associated to soil destructuration, clearly visible in the contour maps of P_t, bottom row. In the unbonded soil, the preconsolidation pressure P_s is only slightly affected by the penetration process. On the contrary, in the structured soil P_s experiences a significant decrease in a zone of soil located at the boundary of the plastic region, where plastic dilatancy is occurring. Peak excess pore water pressures of about 420 kPa develop at the cone mid-height in both cases, but overall, the predicted values of Δp_w around the piezocone tend to be larger for the bonded soil.

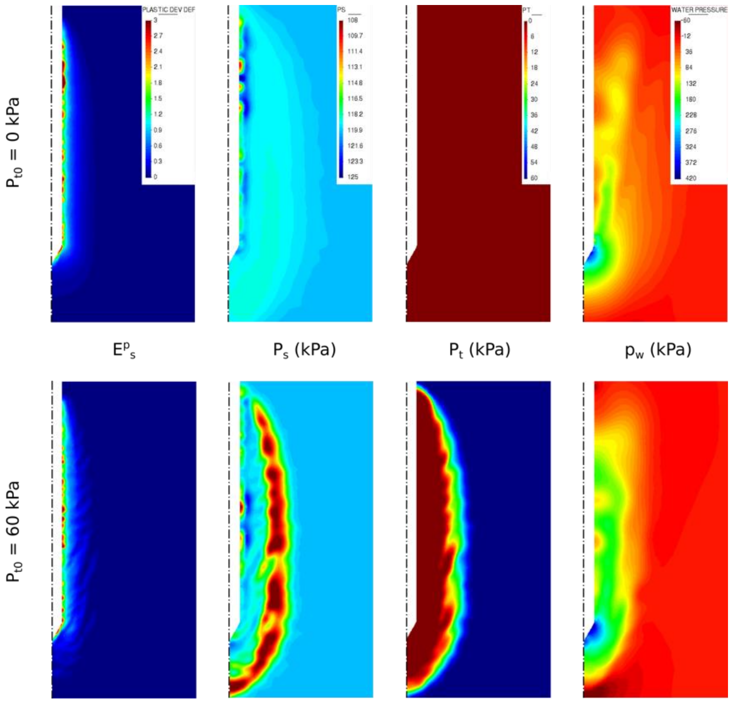

Figure 2: Contour maps of CPTu test results at the maximum penetration depth.

5. Conclusions

The results of this study appear consistent with the observed CPTu data in similar soils and demonstrate that the complex soil response to the penetration of the piezocone in CPTu tests can actually be modelled quantitatively in a rational and computationally efficient way. This opens a promising new perspective for a more realistic interpretation of this kind of in-situ tests for geotechnical site investigations.

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