Particle finite element modelling of retrogressive slope failure in sensitive clays

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Abstract

In this paper, the slope failure in sensitive clays is studied numerically using the particle finite element method which is a novel approach capable of tackling extreme deformation problems. The sensitive clay is described using an elastoviscoplastic model with strain softening. This model can be considered a mixture of the Tresca model with strain softening and the classical Bingham model. The former is used to capture the solid-like behaviour, for example when the clay is intact, whereas the latter is to describe the semi-fluid behaviour of the fully remolded clay. The complete process of the collapse of a sensitive clay deposit is captured successfully and the widely-observed retrogressive failure mode is reproduced. The mechanism behind the failure is also discovered.

Keywords: Sensitive Clay, Retrogressive Failure, Landslide, Continuum Modelling, PFEM

1. Introduction

Sensitive clay is a kind of brittle soil whose shear strength decreases when experiencing plastic deformation. An intact sensitive clay behaves like a solid, whereas after it is fully remoulded it is more like a non-Newtonian fluid. Because of its complex behaviour, landslides in sensitive clays exhibit unique phenomena, with retrogressive failure being a remarkable one. More specifically, a series of collapse modes may occur following the initial failure, leading to an unexpectedly long retrogression distance. Such a retrogressive failure mode has been widely reported in Canada and Scandinavia [1].

The correct prediction of retrogressive slope failure is of great importance to minimize the degree of destruction. However, the modelling of landslides in sensitive clays has long been recognized as a challenge. Their complex behaviour requires models that can describe both their solid-like behaviour and rheological properties. In addition, the extreme deformation of the materials involved in the failure process requires a robust numerical approach. Although the classical finite element method has been widely used in engineering practice, it fails to produce the complete process of landslides in sensitive clays because of the severe mesh distortion.

In this paper, we introduce an elastoviscoplastic model with strain softening into the particle finite element method for simulating the retrogressive slope failure in sensitive clay. The complete failure process is captured successfully and the mechanism behind it is revealed.

2. Constitutive model

An appropriate constitutive model is essential for capturing the slope failure in sensitive clays accurately. The adopted model should be capable of describing both the solid-like behaviour of the intact sensitive clay and the semi-fluid behaviour when it is fully remoulded. In this work, an elastoviscoplastic model with strain softening is chosen to achieve this goal. The model, in a one-dimensional case, is illustrated in Fig. 1. For the general case, assume that $\dot{\boldsymbol{\varepsilon}}$ is the total strain rate that can be split into

$$\dot{\boldsymbol{\varepsilon}} = \dot{\boldsymbol{\varepsilon}}^{\mathrm{e}} + \dot{\boldsymbol{\varepsilon}}^{\mathrm{vp}} \tag{1}$$

in which $\dot{\boldsymbol{\varepsilon}}^{\mathrm{e}}$ is the elastic strain rate and $\dot{\boldsymbol{\varepsilon}}^{\mathrm{vp}}$ is a viscoplastic strain rate. The material is elastic and and $\dot{\boldsymbol{\varepsilon}}^{\mathrm{vp}} = 0$ when the stress state $\boldsymbol{\sigma}$ is inside a yield domain (defined, for example, by a yield function of the form $F(\boldsymbol{\sigma},\kappa)<0$ with κ being a set of hardening/softening variables). In this case, the elastic strain is related to the total stress through Hooke's law as

$$\dot{\boldsymbol{\varepsilon}}^{\mathrm{e}}=\mathbb{C}\dot{\boldsymbol{\sigma}} \tag{2}$$

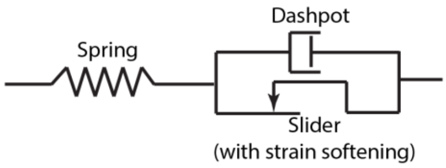

Fig. 1. A one-dimensional elastoviscoplastic model with strain softening

Herein, the Tresca yield criterion is employed which, for plane-strain cases, can be expressed as

$$F(\boldsymbol{\sigma},\kappa)=\sqrt{(\sigma_{xx}-\sigma_{yy})^2+4\sigma_{xy}^2}-2c_{\mathrm{u}}(\kappa) \tag{3}$$

where cohesion softening is adopted to capture the basic post-failure behaviour. The strain-softening is introduced by reducing the cohesion c_{u} with the accumulated equivalent deviatoric plastic strain $\kappa=\int\dot{\kappa}\mathrm{d}t$, where

$$\dot{\kappa}=\sqrt{0.5\dot{e}_{ij}^{\mathrm{vp}}\dot{e}_{ij}^{\mathrm{vp}}} \tag{4}$$

$\dot{e}_{ij}^{\mathrm{vp}}$ is the deviatoric viscoplastic strain rate tensor given by

$$\dot{e}_{ij}^{\mathrm{vp}}=\dot{\varepsilon}_{ij}^{\mathrm{vp}}-\frac{1}{3}\dot{\varepsilon}_{kk}^{\mathrm{vp}}\delta_{ij} \tag{5}$$

in which δ_{ij} is the Kronecker delta. As shown in Fig. 2, the cohesion is a bilinear function of κ which is in accord with the assumption in [2, 3].

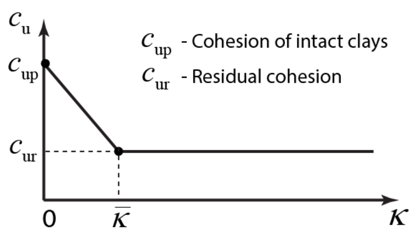

Fig. 2. Variation of the cohesion c_u which is a bilinear function of the equivalent deviatoric plastic strain κ

If stress states satisfy $F(\boldsymbol{\sigma})\geq 0$, then a viscoplastic strain rate is induced and the classical Bingham model is adopted. The total stress is now rewritten as

$$\boldsymbol{\sigma}=\boldsymbol{\tau}+\eta\dot{\boldsymbol{\varepsilon}}^{\mathrm{vp}} \tag{6}$$

where η is the viscosity coefficient, $\boldsymbol{\tau}$ is the stress lying on the boundary of F so that $F(\boldsymbol{\tau},\kappa)=0$, and the quantity $\boldsymbol{\sigma}-\boldsymbol{\tau}$ is called the overstress. The viscoplastic strain rate can be determined by

$$\dot{\boldsymbol{\varepsilon}}^{\mathrm{vp}}=\dot{\lambda}\nabla_{\boldsymbol{\tau}}F(\boldsymbol{\tau},\kappa) \tag{7}$$

since $\dot{\boldsymbol{\varepsilon}}^{\mathrm{vp}}$ is normal to the yield surface at $\boldsymbol{\tau}$. In the above equation $\dot{\lambda}$ is the rate of the non-negative plastic multiplier and $\nabla_{\boldsymbol{\tau}}(*)$ denotes the derivative of $(*)$ with respect to $\boldsymbol{\tau}$. Obviously, the classical elastoplastic model is recovered in the case of $\eta=0$.

The above equations for the constitutive model, together with the momentum conservation equations, the kinematic equations and the boundary conditions, comprise the governing equations that need to be solved for the study of slope failure in sensitive clays.

3. Particle finite element method

The retrogressive failure of a slope in sensitive clay inevitably involves extreme deformation of the material as well as free-surface evolution. These features may lead to issues when using the traditional Finite Element Method (FEM), since it cannot capture easily the emergence of new free surfaces owing to its fixed mesh topology. Furthermore, severe mesh distortion occurs when the slope material undergoes ultra large strains. To tackle these issues, the so-called Particle Finite Element Method (PFEM) is used, which is a mixture of the particle method and the finite element method.

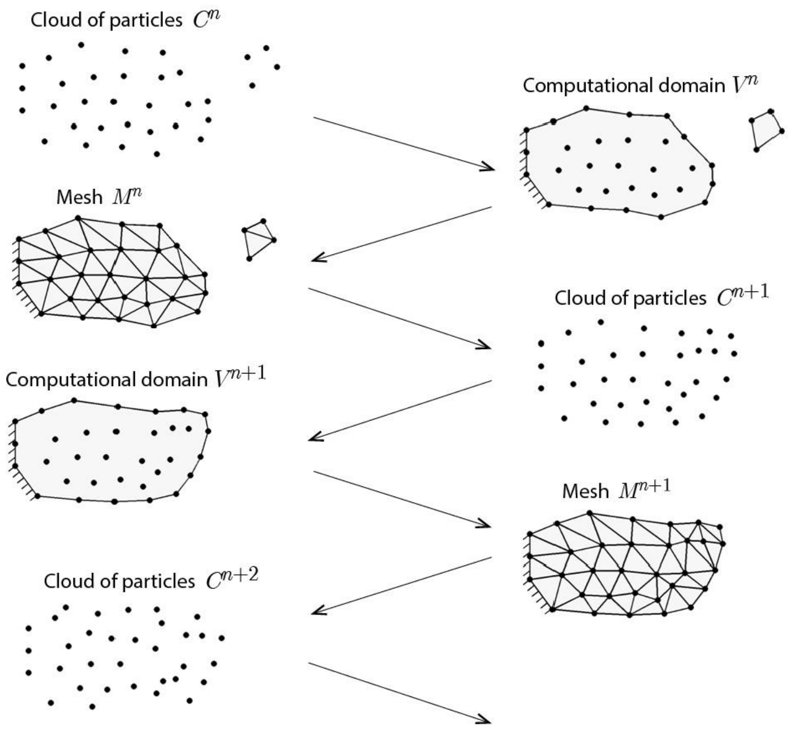

Fig. 3. Basic steps of the PFEM [4]

In the PFEM, the computational domain is discretized using standard meshes, for which the governing equations are solved through standard finite element analysis. The mesh topology after each step, however, is erased and the computational domain is re-identified on the basis of mesh nodes, considered as free particles at this stage, using the α-shape method [5]. The domain is then discretized again through remeshing techniques and state variables such as velocities, stresses, and strains are then mapped from the old mesh to the generated new mesh. Thereafter, another incremental FE analysis proceeds. In such a way, the PFEM is capable of not only tackling mesh distortion issues resulting from extreme deformation, but also of capturing severe evolution of the free-surface. Furthermore, it inherits the solid mathematical foundation of the classical finite element method.

The basic steps of the PFEM are illustrated in Fig. 3 with more details given in [4]. The PFEM has so far been applied successfully for modelling a number of challenging problems in geomechanics including granular flow [6, 7], fluid-soil-structure interaction [8], soil-pipeline interaction [4, 9], ground excavation[10], and landslides [11-13].

4. Results and discussions

The problem considered is a sensitive clay deposit shown in Fig. 4, where the geometry is illustrated. The collapse is triggered by releasing the pressure initially imposed on the slope surface. The material parameters of the sensitive clay are as follows: Young's modulus $E = 5\times10^6$ Pa, Poisson's ratio $\upsilon = 0.42$, unit weight of the clay $\gamma = 2.0\times10^4$ N/m^3, viscosity coefficient

$\eta = 500$ Pa·s, undisturbed shear strength $c_{up} = 15$ kPa, remoulded shear strength $c_{ur} = 1.0$ kPa, and $\bar{\kappa} = 25\%$. The surface of the ground is assumed to be rough and the gravitational acceleration is $g = -9.8$ m/s^2. The time step utilized is $\Delta t = 0.05$ s and the simulation proceeds until the final deposit is obtained.

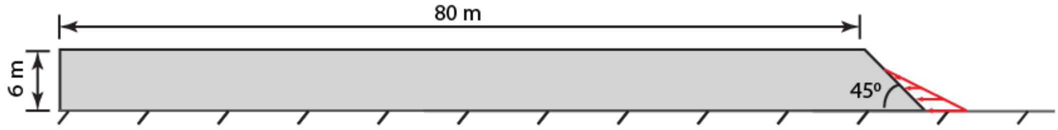

Fig. 4. The geometry of a sensitive clay deposit

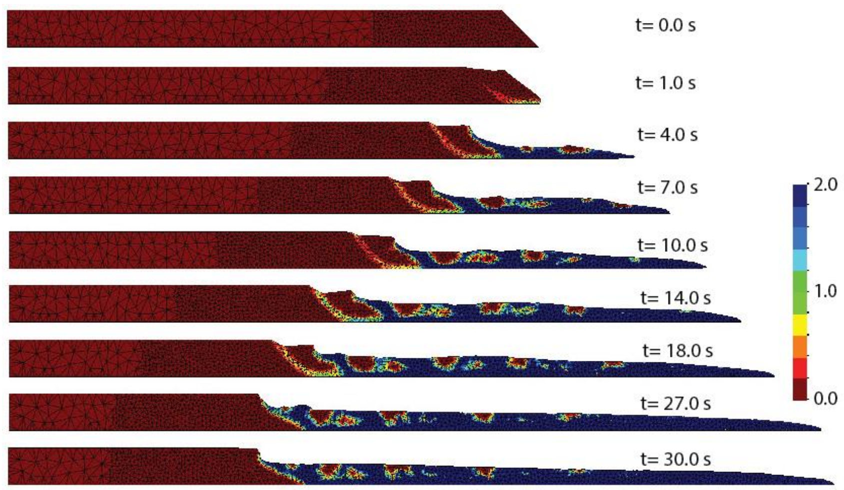

Fig. 5. Distribution of the equivalent deviatoric plastic strain

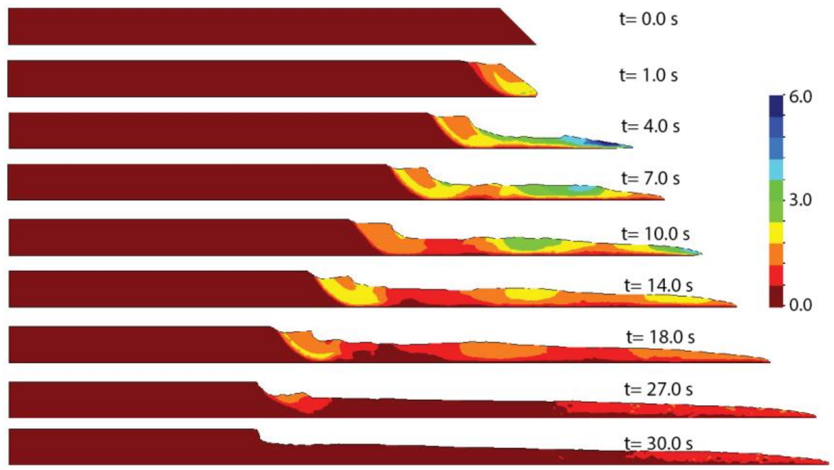

Fig. 6. Distribution of the velocity (unit: m/s)

The distribution of the accumulated equivalent deviatoric plastic strain is illustrated in Fig. 5. As shown, the slope fails the moment the pressure is released. A shear band originating from the toe of the slope propagates along the basal surface (t = 1.0 s) and then towards the upper surface of the sensitive clay deposit. At t = 4.0 s, the mass involved in the first failure is far away from the new slope surface generated due to the first slope failure; a new shear band, similar to the first one, appears leading to the collapse of the newly-generated slope. Such collapse continues until a sufficient amount of the mass is stored at front of the new slope surface. For this problem, there are a total of six collapse states leading to a retrogressive distance of around 40 meters.

Fig. 6 shows the distribution of the velocity. This plot indicates that the sliding front first increases to its maximum speed before starting to decrease due to effects of plastic dissipation. When new collapse occurs, a part of the potential energy is transferred into the kinematic energy with the rest being dissipated. It is worth noting that the horizontal velocity dominates throughout the collapse process as shown in Fig. 7; nevertheless, the vertical velocity is also signficant at the point where each new slope failure occurs due to the rotation of the involved mass.

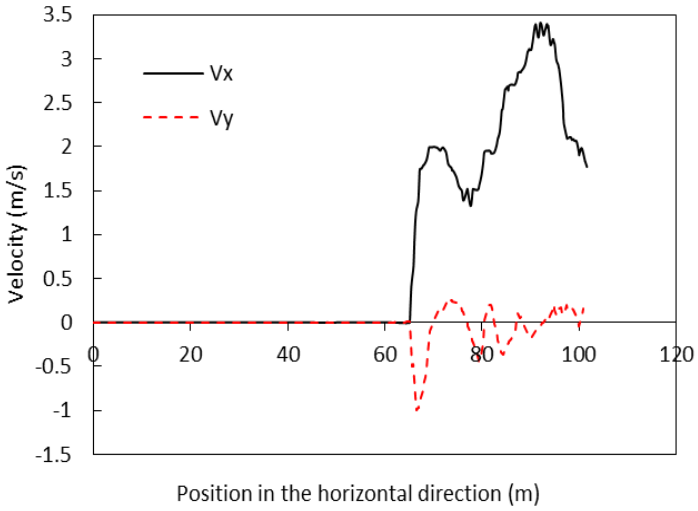

Fig. 7. Variation of the velocities along the plane y = 1 m at t = 7.0 s

5. Conclusions

The collapse of a sensitive clay deposit is studied numerically using a continuum approach – the particle finite element method (PFEM). The solid-like and semi-fluid like behaviour of the sensitive clay is described using an elastoviscoplastic model with strain softening. It is shown that the PFEM, along with the utilized model, succeeds in capturing the retrogressive failure of the slope. The shear band triggers from the toe of the slope, then propagates first along the basal surface and finally upwards to the top surface of the deposit. A sequential failure occurs if the mass in front of the newly generated slope surface moves away. The distribution of the material velocity indicates that the horizontal velocity dominates throughout the failure process. However, when a new failure occurs, the vertical velocity may increase as well because of the rotated failure mode.

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