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A depth-integrated viscoplastic model for dilatant saturated cohesive-frictional fluidized mixtures: Application to fast catastrophic landslides

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ABSTRACT

Fast catastrophic landslides, flowslides, avalanches, lahars and debris flows involve fluidized mixtures of soil and water. To improve safety of human settlements endangered by them it is necessary to predict their occurrence, triggering conditions, path, velocity and depth, and runout distance. Predictions are based on models (mathematical, rheological and numerical). In this paper (i) we introduce a hierarchical set of mathematical models describing solid-pore fluid coupling, (ii) we introduce the concept of dynamical critical state line (DCSL), depending on shear strain velocity, which generalizes the critical state line (CSL) used in Geomechanics to describe residual conditions at failure, (iii) we propose a dilatancy term depending on the distance to the DCSL, (iv) we introduce a viscoplastic law for dilatant cohesive-frictional fluids based on a deviatoric and a volumetric part, and (v) we describe the depth-integrated version of the proposed rheological model, providing all necessary items to be implemented in a numerical model.
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1. Introduction

Fast landslides, avalanches, lahars and debris flows are geophysical, gravity driven flows causing extensive loss of human live and economic damage throughout the world every year. They involve phase changes from solid to fluid at the triggering stage and from fluid to solid when the motion stops. The interaction between solid grains and pore water is of paramount importance. Indeed, the mechanical behaviour of soil–water mixtures does not depend on total stresses, but on effective stresses, which are obtained by subtracting the pore pressure hydrostatic tensor from the total stress tensor.

Slope failures can be triggered by different causes: (i) changes of effective stresses induced by changes of load (earthquakes), pore pressures (rain), or changes of geometry (erosion), and (ii) material degradation due to weathering. There exist several alternative classifications of landslides according to their shape, morphology, velocity of propagation, etc. Two interesting types are slides and flows. In the former, deformation is concentrated in a narrow zone, the failure surface. Kinematics can be approximately described as the movement of a rigid mass along the failure surface. This mechanism of failure is associated to concepts such as material softening, strain localization and shear bands. Flows are usually produced in loose metastable deposits, where failure is of diffuse type [14,43]. The tendency of the soil to compact results on the generation of high pore pressures and a decrease of the effective confining pressure which causes in some cases liquefaction. As the fluidized material propagates, phenomena of dissipation of pore pressures and dilatancy take place. Modelling of these phenomena is complex because of the many difficulties encountered in mathematical, rheological and numerical modelling [1,24].

Concerning mathematical modelling, there is still much work to be done to describe coupling between different phases, and phenomena such as segregation and comminution. Coupling between solid particles and pore fluids has been investigated in Geomechanics since the early work of Biot [5], and considerable progress has been achieved [6,8,13,35,50–52]. Concerning the propagation phase, the study of coupling has been developed at a later stage [22,25–27,43].

Rheological models have been developed since the work of Bingham [4]. In the case of cohesive fluids, exhibiting a yield stress, it is worth mentioning the contributions of Hohenemser and Prager [19] and Oldroyd [40] who generalized Bingham model for general

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stress conditions (see also [37]), and those of references [9,10,11,15] and [36].

The case of geophysical granular flows involves the additional complexity of the grain sizes, which can range from fractions of mm to m. Rheological tests are also complex to perform, and in many occasions, the researchers use small scale flumes [39]. After the publication of Bagnold experiments [3], many different models have been proposed [2,7,30–32,46,49].

In this paper (i) we introduce a hierarchical set of mathematical models of different complexity describing solid-pore fluid coupling, (ii) we introduce the concept of dynamical critical state line (DCSL), depending on shear strain velocity, which generalizes the critical state line (CSL) used in Geomechanics to describe residual conditions at failure, (iii) we propose a dilatancy term depending on the distance to the DCSL, (iv) we introduce a viscoplastic law for dilatant cohesive-frictional fluids based on a deviatoric and a volumetric part, and (v) we describe the depth-integrated version of the proposed rheological model, providing all necessary items to be implemented in a numerical model.

2. Mathematical modelling framework

Fluidized geomaterials are mixtures of soil and water with a mechanical behaviour governed by coupling between solid and fluid phases. Indeed, intergranular stresses depend on pore pressure. In the limit case of liquefaction, the intergranular stresses are close to zero, and the soil behaves as a viscous fluid.

Much effort has been devoted in the past to understand coupling between soil skeleton and pore water. The first model was proposed by Biot [5,6] for linear elastic materials. Further development was produced at Swansea University, where Zienkiewicz and co-workers [50–52] extended the theory to non-linear materials and large deformation problems. It is worth mentioning the contributions to this field of Lewis and Schrefler [35], Coussy [8], and de Boer [13]. In the case of fluidized geomaterials, this theoretical framework has not been applied until recently. We can mention here the work of Hutchinson and Prager [19] who proposed a sliding consolidation model to predict run out of landslides, Iverson and Denlinger [27], Pastor et al. [42–45] and Quecedo et al. [47]. This section is devoted to derive a coupled depth-integrated model from the velocity–pressure model of Zienkiewicz.

2.1. Velocity–pressure model

We will assume that the fluidized soil consists of a solid skeleton and a fluid phase, water, which fills completely the voids. The skeleton is made of particles of density ρ_s having a porosity n (volume percent of voids in the mixture) and a voids ratio e (volume of voids per unit volume of solid fraction). Both are related by:

$$n = \frac{e}{1+e}$$

Movement of the fluid can be considered as composed of two parts, the movement of soil skeleton and motion of the pore water relative to it:

$$v_w = \frac{v+w}{n}$$

where v_w is the velocity of pore water and w the averaged velocity of water relative to the soil.

The total stress tensor σ acting on the mixture can be decomposed into a hydrostatic pore pressure term $p_w I$ and an effective stress tensor σ' acting on soil skeleton as:

$$\sigma = \sigma' - p_w I \quad (1)$$

where I is the second order identity tensor.

The balance of momentum equation for the mixture can be written as:

$$\operatorname{div}(\sigma' - p_w I) + \rho b = \rho \frac{dv}{dt} \quad (2)$$

where we have neglected the term $\rho_m n (d/dt)(w/n)$ which characterizes the acceleration of water relative to soil grains. In above, ρ is the mixture density, and b the vector of body forces.

Concerning the balance of mass of the pore water, we will consider the following volume changes in the mixture:

(a) Soil skeleton

$$d\theta_1 = \operatorname{tr}(d\varepsilon) = d\varepsilon_v$$

where $d\varepsilon$ is the increment of the strain tensor, tr denotes the trace operator and $d\varepsilon_v$ is the increment of volumetric strain. In the case there exists an additional mechanism causing volumetric strain, we will add a term $d\varepsilon_{v0}$ to account for it. This will be described later on.

(b) Deformation of soil grains caused by pore pressure

$$d\theta_2 = \frac{1-n}{K_s} dp_w$$

where K_s is the volumetric stiffness of soil particles.

(c) Deformation of pore water caused by pore pressure

$$d\theta_3 = \frac{n}{K_w} dp_w$$

where K_w is the volumetric stiffness of pore water.

From here, the mass conservation for the pore fluid can be written as:

$$\operatorname{div} w + \left(\frac{1-n}{K_s} + \frac{n}{K_w}\right)\frac{dp_w}{dt} + \operatorname{tr}\left(\frac{d\varepsilon}{dt}\right) + \frac{d\varepsilon_{v0}}{dt} = 0 \quad (3a)$$

or

$$\operatorname{div} w + \frac{1}{Q}\frac{dp_w}{dt} + \operatorname{div} v + d_{v0} = 0 \quad (3b)$$

where

$$d_{v0} = \frac{d\varepsilon_{v0}}{dt}$$

In above, we have introduced the mixed stiffness of water and solid particles Q:

$$\frac{1}{Q} = \left(\frac{1-n}{K_s} + \frac{n}{K_w}\right) \quad (3c)$$

The balance of momentum of the pore fluid is:

$$-\operatorname{grad} p_w + \rho_w b - \frac{1}{k_w} w = \rho_w \frac{dv}{dt} \quad (4)$$

where ρ_w is the density of the water, and k_w the permeability. We will assume that Darcy law can be used to describe the interaction between pore water and soil skeleton, although other alternatives can be chosen. In above, we have neglected the accelerations of the pore water relative to soil skeleton.

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