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Granular column collapse of wet sand

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Abstract

It is well known that partial saturation enhances the mechanical behaviour of sand. Whilst its influence at small deformation has been largely discussed in the literature, its influence at large deformation remains largely unexplored. This paper investigates the influence of partial saturation on the collapse and flow of dry and wet sand by simulating a well established experiment, called the *granular column collapse*, using the material point method. The results showed that partial saturation influences the primary failure surface at small deformation, which defined the volume of the mobilised mass, the post-failure flow behaviour and the run-out distance, though not as significantly as one could expect. This can be explained by small differences in the potential energy of the mobilised mass as it will be explained in this paper.

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Keywords: granular column collapse; material point method; unsaturated sand; stress-dilatancy.

1. Introduction

The granular column collapse is a well established experiment, which consists in releasing a column of granular material on a flat surface, and extensive data is available in the literature covering a variety of dry granular materials [1–5]. Whilst the the experiment offers simplicity in execution, its numerical simulation is complex [6] due to transition from solid-like to fluid-like behaviours and the low stresses. A variety of numerical methods have been used to simulate the column collapse such as MPM [7–11], DEM [8,12] or SPH [7,13]. However, most continuum methods relied on simple constitutive models to idealise the mechanical behaviour of the dry sand. Fern and Soga [14] showed that these simple constitutive models overestimated the run-out distance due to insufficient energy dissipation but more advanced models, such as Nor-Sand [15], could predict the correct run-out distance for dry sand. However, the failure and flow mechanism of wet sand have never been investigated. Yerro et al. [16] investigated the failure mechanism of progressive landslides in partially saturated conditions with MPM but did not investigate the flow behaviour. Bandara [7] investigated the collapse and flow of partially saturated levees but used a simple Mohr-Coulomb model. The ability of MPM to model slope failures has been discussed by Soga et al. [17].

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Fern et al. [18] showed that the enhancement by partial saturation of the mechanical behaviour of sand can be explained by a modification of the soil fabric and, hence, a modification of the dilatancy characteristics. This modelling approach offers an advantage when modelling the transition from small strain solid-like to large strain flow-like behaviours as the contribution of dilatancy towards strength is automatically mitigated as the material reaches the critical state [19].

The combination of the recent developments in MPM simulations along with the development in constitutive models allows investigating the failure and post-failure behaviours of wet sand. Following the same path as Fern and Soga [14], two columns collapses were simulated for dry and wet sand, respectively. Additionally, the potential energy of the mobilised mass, which can be used as a proxy for the run-out distance, was computed for two different densities and for two different degrees of saturation.

2. Constitutive modelling of wet sand - Unsaturated Nor-Sand

There is a consensus among the scientific community that constitutive models for unsaturated soils are extensions of saturated models. Therefore, the abilities and limitations of a given model to predict the mechanical behaviour of saturated soil will be inherited in the unsaturated extension. For instance, the Barcelona Basic Models [20] suffers from the same limitations, when modelling dense soils, as the Cam-Clay models [19,21] as discussed by Fern [22]. Moreover, constitutive models for large deformation simulations must be able to capture both the small strain and the large strain behaviours. This is paramount as the initial failure surface and, hence, the mobilised mass are defined at small strains whilst the flow takes place at large deformation. The critical state soil mechanics framework [23] offers an ideal framework in which such a model can be developed.

2.1. Critical State

Roscoe et al. [24] suggested that any soil sheared sufficiently would eventually reach a unique state called the *critical state* at which point the dilatancy rate and the changes in dilatancy rate are nil (Eq. 1).

$$D = \frac{\partial D}{\partial \varepsilon_d} := 0 \tag{1}$$

where $D = d\varepsilon_v/\partial\varepsilon_d$ is the dilatancy rate, ε_v the volumetric strain and ε_d the deviatoric strain.

Roscoe and Schofield [21] were driven by Taylor's idea [25] that the strength of soil was the consequence of inter-granular friction and interlocking. They expressed the stress-dilatancy theory in terms of stress and strains invariants. An additional dilatancy parameter N was later introduced [26] to offer better modelling possibilities (Eq. 2)

$$\eta' = M + (N-1)D \tag{2}$$

where $\eta' = q/p'$ is the effective stress ratio, p' the mean effective stress, q the deviatoric stress, M the critical state effective stress ratio and N the dilatancy parameter, which is nil in the Cam-Clay models ($N = 0$).

Therefore, the nil dilatancy condition of the critical state theory and the stress-dilatancy theory imply that a unique stress state exists at the critical state (Eq. 3).

$$D = 0 \rightarrow \eta'_{cs} = M \tag{3}$$

Bolton [27] suggested a state index called the *relative dilatancy index* as a proxy for dilatancy. Boulanger [28], followed by Mitchell and Soga [29], showed that the relative dilatancy index could be used to define the critical state density as the dilatancy rate is nil (Eq. 4).

$$I_R = I_D \cdot I_C - 1 = 0 \rightarrow e_{cs} = e_{max} - \frac{e_{max} - e_{min}}{\ln(Q/p')} \tag{4}$$

where I_R is the relative dilatancy index, $I_D = (e_{max} - e)/(e_{max} - e_{min})$ with e_{max} and e_{min} the maximum and minimum void ratios, $I_C = \ln(Q/p')$ the relative pressure index with the Q the crushing pressure, which is typically 10 MPa for silica sand, and e_{cs} the critical state void ratio.