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Research Paper

Three-dimensional discrete element analysis of triaxial tests and wetting tests on unsaturated compacted silt

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ABSTRACT

Three-dimensional DEM (discrete element method) simulations of triaxial tests and wetting tests on unsaturated silt were carried out in order to analyse the macro and microscopic behaviour under triaxial stress conditions. Interparticle adhesive forces were incorporated into a rolling and twisting resistance model to represent the van der Waals and capillary forces. A loose DEM sample was first produced considering rolling and twisting resistance and van der Waals forces to sustain an open structure. A series of pseudo-constant water content triaxial tests were simulated by controlling adhesive forces at contacts using the matric suction and a group of representative soil-water characteristic curves. The wetting tests were performed by quickly and gradually wetting (QW and GW) the unsaturated sample at different deviator stress levels. By introducing adhesive forces at contacts, the simulation reproduces the main mechanical behaviour. In the triaxial tests, Z_m (mechanical coordination number) for the unsaturated sample increases rapidly during shear until becoming approximately constant. The induced structural anisotropy (under deviatoric loading) reduces with decrease in water content due to the higher interparticle adhesive forces. In the gradual wetting tests, the deviatoric fabric increases gradually during wetting. The deviator strains induced by QW and GW (loading-wetting sequence) are much larger than for the saturated sample at the same stress state in triaxial tests (wetting-loading sequence).

1. Introduction

Geo-researchers have made great efforts to establish a framework for unsaturated soil mechanics using in situ and laboratory tests [1–3], and theoretical and numerical analyses [4–8]; in order to comprehend unsaturated soil mechanics problems such as seepage, shear strength and volume change [9]. Although the macro-mechanical behaviour has been obtained by laboratory tests and facilitated theoretical formulation and modelling, it is difficult and expensive to obtain insight into some microscopic information that controls the macroscopic response. With progress in computational techniques, the discrete element method (DEM) [10] is an alternative to carry out multi-scale investigations of soil behaviour. This numerical method can facilitate sample reproducibility and monitor the detailed internal information (e.g. average coordination number and fabric tensor) in a non-destructive way.

In DEM simulations of unsaturated soils, the theoretical approximation of Fisher [11] is commonly adopted to calculate the capillary force based on the volume and shape of the water menisci between particles. However, simulations [12] using this method are restricted to the pendular regime (the degree of saturation is generally less than 20%). To simulate unsaturated soil over a large range of degree of saturation, Gili and Alonso [8] defined general expressions for the effect of water menisci between particles and transfer laws between pores and menisci. Jiang et al. [5] proposed a method to consider the disappearance of menisci and air bubbles. Unfortunately, these methods can only simulate the capillary force qualitatively because of the ideal capillary theory and the sphere particle shape. It was concluded by Gili and Alonso [8] that the water menisci contribute to the soil shearing strength and also resist tension by interparticle capillary forces. Hence, the essential feature of unsaturated soil at the particle scale is the liquid bridge forces at contacts resulting from the contact water menisci. A simple empirical method to simulate unsaturated soils is to directly install adhesive forces at contacts to represent capillary forces, as reported by Liu and Sun [13] and Kim et al. [14].

Silt-sized soil is relevant to investigate/validate unsaturated soil properties and theories. The void ratio limits of a compacted soil are influenced by the dimensionless particle size distribution, particle

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shape and particle size [15]. The structure becomes looser and looser as particle size decreases because when the particle size is small, the interparticle weak attractive force (collective outcome of van der Waals and ion electrostatic forces etc.) becomes more important and the gravity force is no longer dominant [15]. In the authors' opinion, the weak force at contacts is an essential characteristic of fine grained soils and results in weak cohesion and loose structure. For loess, a kind of silt-sized aeolian soil, the ion electrostatic force is negligible, the van der Waals force is dominant. To create DEM samples of fine grained soils, it is necessary to consider the van der Waals force at contacts as well as the rolling and twisting resistance ascribed to particle shape and surface roughness.

Unsaturated soils may undergo volume-change (large decrease in volume that is termed collapse) under particular stress states due to wetting because the decrease in suction causes a decrease in soil shear strength and bearing capacity. Compacted soil engineering with unsaturated filling materials (e.g. embankments, slopes and subgrades) may experience deformation and instability ascribed to rain and irrigation. Many experiments have been reported on both compacted and naturally deposited unsaturated soils to study wetting-induced volume change under one-dimensional (K_0) compression [16–18], isotropic compression [19–22] and triaxial compression [18,20,22,23] and have been reviewed to summarize the collapse triggering mechanisms [24]. Besides experimental investigations [17,18,25], two-dimensional DEM simulations have also been carried out to investigate collapse behaviour of cemented soil under one-dimensional compression and biaxial stress states [26,27].

The objective of this study is to numerically simulate the mechanical behaviour of unsaturated silt under triaxial test conditions and wetting tests and to investigate the associated microscopic information. A loose DEM sample was first produced considering rolling and twisting resistance and van der Waals forces to sustain an open structure. The constant water content condition is applicable in engineering practice since the time taken is significantly reduced in laboratory tests [3] and the air phase is drained and the water phase is undrained in many field situations [28].

A series of pseudo-constant water content triaxial tests were simulated by controlling adhesive forces at contacts using the matric suction, which is determined by the water content and void ratio using a group of representative soil-water characteristic curves. A sample with an initial water content of 7.1% was quickly and gradually wetted under different deviatoric stress levels and the stress-strain relationship, volume change, coordination number and fabric of contact normals were analysed.

2. Contact model with adhesive force

A three-dimensional (3D) rolling and twisting resistance model was proposed by Jiang et al. [29] to consider particle shape effects in sphere-based DEM simulations. In this study, an unified adhesive force is incorporated into the rolling resistance model so that the influence of capillary force and weak attraction forces including van der Waals force can be considered. Besides the normal and tangential forces, it is assumed that particles can transfer rolling moment and twisting torque through a circular flat contact area. The model includes elastic behaviour and limiting strength in the four directions of interaction.

Fig. 1(a) and (b) show the schematic diagram of the contact model. Although force/moment–displacement/rotation relationships between real soil particles are nonlinear, linear force/moment–displacement/rotation laws are applied in the normal, shearing, rolling and twisting directions for simplicity. In the normal direction, the normal force is expressed as:

$$F_n = \begin{cases} k_n u_n - F_a & u_n \geqslant 0 \\ 0 & u_n < 0 \end{cases} \tag{1}$$

where k_n is the normal stiffness, u_n is the overlap of a particle pair with a negative value denoting separation and F_a is the adhesive force incorporated in this study. When a particle pair separates, the adhesive force is regarded as zero. Once the two spheres come into contact, an adhesive force acts between them immediately.

The tangential mechanical response is elastic-perfectly plastic and the tangential force is incrementally calculated by:

$$\mathbf{F}_s \leftarrow \mathbf{F}_s - k_s \Delta\boldsymbol{\delta}_s \tag{2}$$

where k_s is the tangential stiffness and $\Delta\delta_s$ is the relative shear displacement increment. Tangential sliding occurs when the tangential force reaches the limiting value, i.e. $\|\mathbf{F}_s\| \leqslant \mu(F_n + F_a)$, where μ is the friction coefficient.

As assumed, two spheres microscopically interact at a contact over a circular flat contact area with a radius of R_c. The contact radius is calculated by:

$$R_c = \beta R \quad \text{where,} R = 2R_1R_2/(R_1 + R_2) \tag{3}$$

where β is the contact radius coefficient (or shape parameter [29]) used to link the contact area radius and particle size and evaluate the rolling and twisting resistance, R is the common radius of the particle pair and R_1 and R_2 are the respective radii. It is assumed that the circular contact area is continuously distributed with an infinite number of normal spring –divider elements and tangential spring –slider elements, which transfer normal stress and shear stress respectively as shown in Fig. 1(c) and (d).

Fig. 1(c) shows the distribution of the normal contact stress without separate spring –divider element in rolling interaction. By integration, the linear distribution of the normal stress leads to $M_r = 0.25k_nR_c^2\theta_r$, where, θ_r is the relative rotation. At a critical rolling rotation $\theta_{r0} = (F_n + F_a)/k_nR_c$, the normal spring –divider element at point 2 is about to separate as no tension is permitted, and $M_{r0} = 0.25R_c(F_n + F_a)$ denotes the critical rolling moment. Once θ_r exceeds θ_{r0}, the separation of normal spring –divider elements evolves continuously towards the left as θ_r increases. After the critical state, M_r is formulated and plotted in Fig. 1(e) (solid curve, in detail in [29]). To simplify M_r, a local crushing parameter $\zeta_c = 2.1$ related to the hardness of the particle mineral material is introduced to describe the effects of local asperity crushing. Hence, the limiting value of the rolling moment is $M_r \leqslant \zeta_c M_{r0}$.

In the DEM implementation, the rolling moment is incrementally computed by:

$$\mathbf{M}_r \leftarrow \mathbf{M}_r - k_r \Delta\boldsymbol{\theta}_r \tag{4}$$

where $k_r = 0.25k_nR_c^2$ is the rolling stiffness and $\Delta\theta_r$ is the relative rotation increment. Rolling occurs when the rolling moment reaches the limiting value, i.e. $\|\mathbf{M}_r\| \leqslant 0.25\zeta_c R_c(F_n + F_a)$.

Fig. 1(d) shows the distribution of the contact shear stress without sliding spring –slider element in twisting interaction. Through integration, the radially linear distribution of the shear stress leads to $M_t = 0.5k_sR_c^2\theta_t$, where, θ_t is the relative twist. As θ_t constantly increases, the sliders of tangential spring –slider elements at the periphery of the circular area will begin sliding once the shear stresses reach the limiting value $\mu(F_n + F_a)/\pi R_c^2$. Then, the critical twist $\theta_{t0} = \mu(F_n + F_a)/k_sR_c$ and the critical twisting torque $M_{t0} = 0.5\mu R_c(F_n + F_a)$ are derived. After the critical state, M_t is formulated and plotted in Fig. 1(f) (solid curve, in detail in [29]). For simplicity, a linear elastic –perfectly plastic curve (dashed) is applied, and the limiting value of the twisting torque is $M_t \leqslant 1.3M_{t0}$.

In the DEM implementation, the twisting torque is also updated incrementally as:

$$M_t \leftarrow M_t - k_t \Delta\theta_t \tag{5}$$

where $k_t = 0.5k_sR_c^2$ is the twisting stiffness and $\Delta\theta_t$ is the relative twist increment. Twisting occurs when the torque reaches the limiting value, i.e. $M_t \leqslant 0.65\mu R_c(F_n + F_a)$.

The stiffnesses can be computed by [30,31]: