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Numerical study on stress states and fabric anisotropies in soilbags using the DEM

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25 **ABSTRACT**

26

27 Wrapping granular soils in geosynthetic containers, such as soilbags, results in a considerable
28 increase in the bearing capacity due to the effective restraint on the dilatancy of the soil. This paper
29 numerically investigates the stress states and fabric anisotropies in the wrapped soil using the discrete
30 element method, providing a novel perspective for new insights into the reinforcement mechanisms
31 and the development of constitutive relations for soilbags. The two most anticipated loading
32 conditions, namely, unconfined compression and simple shear, are considered, and numerical
33 predictions are compared to experimental results. During unconfined compression, both global and
34 local p - q stress paths evolve linearly, having the same slope until the global failure of the wrapping
35 geosynthetic. Under simple shear, the global stress path approaches the critical state line first and then
36 turns to the compression line of the wrapped soil. Some local loading-unloading stress paths are
37 observed, which may account for the high damping of soilbags during cyclic shear. The reduced fabric
38 anisotropies of the normal and tangential force chains suggest greater confinement from the lateral
39 sides of the geosynthetic container in either loading course. The performance and mechanisms of the
40 soilbag earth reinforcement method, i.e., confinement and interlocking, can be better understood based
41 on these new findings on the stress states and fabric anisotropies.

42

43 **Keywords:** soilbag reinforcement, geotextile, discrete element modeling, stress path, fabric
44 anisotropy

1. Introduction

The knowledge of geotechnical engineering has been tested and advanced by the increasing need for countermeasures against ground disasters, such as earthquakes, slope failures and landslides. However, the growing awareness of sustainable development demands cost-effective and environmentally friendly solutions to geotechnical engineering problems. In recent decades, geosynthetic-reinforced soil (GRS), e.g., geotextile sheets for soil nailing in retaining walls, has manifested itself as one of the most effective earth/subgrade reinforcement methods that addresses both concerns [1-5]. Instead of being layered by planar geosynthetic sheets, wrapping soils entirely in geosynthetic containers, such as soilbags [6,7], results in an astonishing rise in the bearing capacity by utilizing the straining geosynthetic fabrics that inhibit the dilatancy of the soil. Regardless of the different ways that geosynthetic is included in the ground, the GRS is fabricated to be more heterogeneous than natural soils. Due to the discontinuity and anisotropy at the interface between soil and geosynthetic, it is difficult to conduct a comprehensive evaluation on the performance and mechanisms of the GRS from the onset of the external load until the global/local failure of the geosynthetic reinforcement using a conventional continuum approach. The design of the GRS as a road/railway subgrade or foundation reinforcement is mainly based on laboratory testing and empirical assumptions. Hence, the development of efficient numerical tools to model GRS of various forms and types is still urgently needed.

In most cases, geosynthetic sheets are layered in soil to form planar reinforcement on road foundations or retaining structures. GRS does not typically come in the form of soilbags¹; nevertheless, its engineering values should not be underestimated. For economical, technical and sustainable reasons, soilbags have reclaimed their role as a permanent element in earth reinforcement [8]. Based on years of laboratory tests and engineering experience [6,7,9-11] soilbag earth reinforcement has

¹ The term “soilbag” refers to soils that are entirely wrapped by a geosynthetic container as a whole.

been found to be able to:

1. improve the bearing capacity of a soft foundation by five to ten times,
2. provide high compressive strength by wrapping soils of various types inside,
3. dissipate energy under traffic and seismic vibration, and
4. prevent frost heaving if broken stones are used to fill the inside.

Matsuoka and Liu [8] proposed a simplified analytical solution for the compressive strength of soilbags in plain strain conditions, introducing an apparent cohesion c based on the assumption of uniformly distributed tension and a frictionless soil-geosynthetic interface. The idea was later extended to predict compressive deformation by considering the evolution of tension and the principal stress ratio. Since then, several attempts have been made to model the soilbag using truss elements for modeling the tensile behavior of the geosynthetic fabric. The performance of soilbag reinforcement was examined at different scales, ranging from the soilbag-assembly level [12] to the soil-geotextile interface level [13].

Muramatsu et al. [12] validated the capability of a 2D finite element (FE) model to analyze the settlement of soilbag foundations under vertical load in a finite deformation scheme. To attain better control of the settlement, multiple small soilbags stacked together as an assembly are preferable to one large soilbag because the tension on the small bag is fully developed and the dilatancy of the soil is better constrained. This is part of the reason why small-sized soilbags are used in the present study. Tanton and Bauer [13] adopted the micro-polar hypoplastic model to discuss the effect of the soil-geotextile interface (slip/rotation) on stress and strain in the wrapped GRS in a soilbag. They based the interface modeling on a master/slave surface concept and discovered that a frictionless interface leads to slightly higher bearing capacity and lower stress concentration in the middle of the GRS. Ansari et al. [14] extended this concept in their 3D FE model, which allows for the consideration of large interfacial slips, surface separation and reclosures. Interfacial discontinuity and large deformation can be effectively addressed by implementing this concept in an FE scheme, but

they can be better handled with discrete approaches such as the discrete element method (DEM).

A large amount of numerical modeling of the GRS (e.g., geotextile, geogrid) has been conducted using the DEM [3,15,16] and proper coupling between the DEM and other methods [17,18]. Nevertheless, discrete element modeling of GRS wrapped in a geosynthetic soilbag is relatively new. In this work, a woven geotextile was considered as the wrapping material of the soilbag. Its fabric has peculiar mechanical behaviors because of the easy relative motion between the yarns, fabric abrasion and impregnation (with, e.g., resin matrix) [19]. The structural mechanics of this woven system can be accurately represented by orthogonal bars with nodes linked by diagonal and rotational springs. A simplified modeling of the wire mesh was initiated by Thoeni et al. [20,21] using the DEM, considering only the stretching springs with a stochastically distorted law that accounts for the initial distortion of the hexagonal steel wire meshes. The approach generally suits any woven system. In the present modeling, the discretization of the woven geotextile fabric was performed in a similar manner.

In this paper, a novel approach for the discrete modeling of a soilbag is presented to investigate the stress state and fabric anisotropies of the wrapped GRS. The granular soil, which fills a polyethylene (PE) or polypropylene (PP) geotextile container, was fabricated by performing a 3D-assembling-filtering-expanding process using a well-calibrated DEM representative volume (RV) of Toyoura sand. The macroscopic behaviors of the materials and the interface were calibrated against laboratory test data. The proposed DEM soilbag model was validated with the responses of a soilbag in unconfined compression (UC) and simple shear (SS). Aside from reproducing the results of previous experimental [22, 23] and numerical [12-14] studies, the discrete approach is capable of capturing the macroscopic constitutive behavior of wrapped GRS due to discontinuous composition and interaction forces among particles while exploring the microscopic characteristics of this complex geomaterial. The evolution of local/global stress paths and fabric anisotropies of wrapped GRS was investigated to clarify the reinforcement mechanisms of a soilbag; i.e., confinement and interlocking. This numerical study is conducted using the open-source DEM framework YADE [24].

2. Discrete modeling of a soilbag

The numerical modeling of a soilbag must address both discontinuities in the particles and the anisotropy and heterogeneity caused by the inclusion of a geotextile. These problems can be solved by discretizing geotextile-wrapped GRS as an assembly of polydispersed particles that are capable of transferring loads and becoming mobilized by sliding and rolling/twisting. Generation of the assembly was handled with care to ensure that the macro response of Toyoura sand was sufficiently replicated in the simulation. In this model, the particles representing the woven fabric were positioned on the 3D orthogonal mesh of the container (Fig. 1a). These particles are connected with remote springs that sustain tension, and their rotational degrees of freedom are blocked. With accessible information about compressing, sliding, rolling/twisting and stretching among particles at the local scale, the discrete approach can provide a microscopic description of wrapped GRS, e.g., fabric anisotropy and coordination number.

The DEM simulates these local behaviors in a rigorous manner. At the local scale, inter-particle forces are computed from detected overlaps, i.e., the distance between neighboring particles, using simple force-displacement laws. After computing the resultant forces applied on each particle, the dynamic response of the granular system is solved numerically through Newton's second law in a time-marching scheme. As a result, the material is discretized with calculations of the kinematics of each constituent particle, which renders this approach very suitable to model discontinuous behaviors, such as the separation between soil and a geotextile, the breakage of yarn and the rupture of woven fabric. In YADE, this integration scheme is implemented using an explicit finite difference algorithm that considers constant velocities and accelerations at each time step. As detailed in [24], global non-viscous damping is usually adopted for reducing the fluctuation of a response. In this study, a damping coefficient of 0.2 is chosen and no gravitational field is employed, in order to maintain symmetry in the simulations.

2.1 Geometrical assumption of a woven geotextile container

In previous experimental studies [22, 23], each geotextile container was filled with 235.2 N Toyoura sand. The sand was thoroughly compacted until a target height of approximately 80 mm was attained for each soilbag. The container had a curved lateral surface and four slightly tightened corners (Fig. 1b). Given the initial cuboidal shape of the geotextile container (length and width of 400 mm, height of 100 mm) and the target height for the initial compaction, the cross-sections of the soilbag perpendicular to the short and long axes after the compaction were assumed, as shown in Fig. 1c. From these cross-sections, the 3D geometry of the container was constructed (Fig. 1a). On the 3D mesh, the geotextile was discretized using discrete nodes ($r = 2.5$ mm), with a regular mesh size of $5 \text{ mm} \times 5 \text{ mm}$. Note that the volume of the 3D geometry is 0.015 m^3 . If the inside soil weighs 235.2 N, the initial void ratio of the wrapped GRS after the initial compaction is approximately 0.68.

2.2 Determination of the local parameters

The materials modeled using the DEM are typically calibrated against their macroscopic responses in the laboratory. The results from triaxial or biaxial tests on soil specimens for different confining pressures are commonly compared with those produced by the DEM models. In this study, the contact constitutive behaviors of two interacting particles are described by the two nonlinear springs k_n and k_s , which relate the normal force F_n to the overlap u_n and the shear force increment dF_s to the relative shear velocity $\dot{u}_s$ and the time increment dt , respectively, as shown in Eq. (1). The stiffness values of the springs are defined using Eq. (2) from the simplified Hertz-Mindlin non-slip contact law [25]. The shear force is limited by the inter-particle friction angle φ according to the Mohr-Coulomb criterion (i.e., $F_s \leq \tan\varphi F_n$).

$$F_n = k_n u_n \text{ and } dF_s = k_s \dot{u}_s dt \quad (1)$$

$$k_n = 2E^* \sqrt{r^* u_n} \text{ and } k_s = 8G^* \sqrt{r^* u_n} \quad (2)$$

where $r^* = 1/(1/r_1 + 1/r_2)$ is the equivalent particle radius. The equivalent effective Young's modulus E^* and

the shear modulus G^* are respectively expressed as

$$E^* = \frac{1}{\frac{1-v_1^2}{E_1} + \frac{1-v_2^2}{E_2}} \quad \text{and} \quad G^* = \frac{1}{\frac{2-v_1}{G_1} + \frac{2-v_2}{G_2}} \quad (3)$$

where E_i and v_i ($i = 1, 2$) are the Young's moduli and the Poisson's ratios of the contacting particle i and the shear modulus G_i is related to E_i and v_i by $G_i = E_i/2(1+v_i)$.

The moment transfer law $\mathbf{M} = k_m \boldsymbol{\theta}$ for the relation between the moment and the rotation on the contact surface was considered to adequately reproduce the peak stress ratio of the soil. However, the plastic limit for the contact moment $\mathbf{M}$ in [26] was omitted due to its negligible effect when the value for k_m is set. Both contact laws are available in YADE [27].

The tensile constitutive laws of PE and PP geotextiles were acquired from the wide-width tensile tests and represented by nonlinear remote springs in the DEM [21]. For the interactions between soil particles and geotextile nodes, the simplified Hertz-Mindlin contact law and the moment transfer law were employed; however, the parameters in this case differed, based on the calibration against shear box test results.

2.2.1 Sand

The Toyoura sand inside the container is uniformly graded ($D_{60}/D_{10} = 1.3$) with the average radius $\tilde{r} = 0.1$ mm and the specific gravity $G_s = 2.65$. To calibrate the contact model such that the resulting macro response replicates the sand response in similar loading conditions, a cuboid DEM packing that consists of 1000 particles with a scaled particle-size distribution (PSD) ($\tilde{r} = 3$ mm, $G_s = 2.65$) was loaded during triaxial compression in the periodic boundary condition (PBC). To attain an initial void ratio ($e_0 = 0.68$) which is identical to the initial void ratios of the cylindrical triaxial specimens of Toyoura sand (diameter of 50 mm and height of 100 mm) [28], a cloud of particles that were randomly generated in a periodic cell (50 mm $\times$ 50 mm $\times$ 100 mm) underwent an isotropic pressure of 0.1 MPa followed by periodic reduction of the inter-particle friction angle when equilibrium was attained. The iterative process stopped when e_0 approximated 0.68. The contact

orientation diagram and the coordination number of the resulting cuboidal DEM particle assembly ($46.2 \text{ mm} \times 46.2 \text{ mm} \times 92.4 \text{ mm}$) prior to loading are shown in Fig. 2a.

Different levels of isotropic compression were applied before the triaxial loading. Fig. 2b compares the macro responses of the DEM packing and the Toyoura sand specimen in drained triaxial compression. The local behaviors of the DEM packing were defined using the parameters in Table 1. Because the initial void ratio of the wrapped GRS is 0.68 as well, the same initial DEM packing ($e_0 = 0.68$) which was used for contact model calibration can be employed as the RVs to generate the GRS packing inside the soilbag.

2.2.2 Geotextile

The discrete modeling of woven textiles considers the fabric as a collection of orthogonal yarns, giving insights into the contribution of yarn properties and fabric organization to the macro behavior [19]. The main objective of current geotextile modeling is to investigate the dry interactions of the woven fabric with soil. Therefore, the woven fabric was modeled at a scale at which the fabric's structural mechanics can be described. This was done by linking the physical nodes where the warp and the weft meet with remote springs, as shown in Fig. 3a. Because the fabric was constantly strained with great tensile force in both the warp and the weft in either a UC or SS condition. Sliding was not introduced at the intersections of the warp and the weft and the rotation of the particles that represent the geotextile was prohibited.

Although there is some degree of irregularity in yarn properties and fabric organization, the stochastically distorted shift of the force-displacement curve as formulated in Thoeni et al. [21] was not considered. Instead, the tensile behavior was solely governed by the curves in Fig. 3b, which were converted to the tensile stress-strain relations considering the fabric thickness and approximated by a piece-wise linear function. Based on the discretization in Fig. 1c, each remote spring is assumed to have the same cross-sectional area (PE: $5 \text{ mm} \times 0.25 \text{ mm}$ or PP: $5 \text{ mm} \times 0.39 \text{ mm}$).

2.2.3 Interface between sand and geotextile

The soil-geotextile interfacial behavior, which can be affected by apertures and undulations of the geotextile, is simply considered to be the roughness that is introduced from the topography of the linked geotextile nodes in the discrete model (blue spheres in Fig. 4a). The interface response was calibrated by shearing a sandbox ($100\text{ mm} \times 100\text{ mm} \times 6\text{ mm}$) on a fixed PE/PP geotextile sheet ($200\text{ mm} \times 200\text{ mm}$). Friction on the bottom and inner surfaces of the sidewalls were excluded by using double-layered Teflon films with silicone grease lubrication. The same boundary conditions were set for the soil spheres within the shear box in the simulations. Fig. 5a displays a schematic view of a shear box simulation. The best agreement with the experimental data was obtained with the interface friction angles 25° and 21° for the PE sheet and the PP sheet, respectively (Fig. 5b). A detailed explanation on the calibration process is provided in Cheng and Yamamoto [29,30].

2.3 Generation of geotextile-wrapped GRS

With the closed 3D boundary properly defined in Fig. 1c, the inside can be easily filled with a DEM packing, whose constituent particles expand in radii only when equilibrium is attained [31]. The primary concern is preserving a fabric structure that is equivalent to the fabric structure of the wrapped Toyoura sand after the initial compaction in the tests. The void ratio is the most conventional and convenient indicator to describe the fabric structure of an isotropic specimen and is also of great significance in classical soil models. Hence, a packing generation process in which a void-ratio-specified DEM packing gradually grows is applied. Thereby, the particles become geometrically compatible to the 3D boundary in the end without inducing excessive internal forces.

To generate a packing with a particular void ratio, the initial DEM packing ($e_0 = 0.68$) in section 3.1.1 was used as the RVs to construct the geotextile-wrapped GRS packing. Because of the compatible contacts along the boundaries of neighboring RVs, the assembled cuboid-shaped packing with 100 RV duplicates (10 along the x -axis and 10 along the y -axis) has intrinsic fabric consistency. Because the response of the DEM packing used as RVs was calibrated against a series of triaxial tests

(although not precisely for volumetric strain at high confining pressure), the cuboid-shaped packing is deemed capable of quantitatively reproducing the macro response of the wrapped Toyoura sand while shedding some light on the fabric characteristics of a soilbag in given loading courses.

The cuboid-shaped packing was filtered to satisfy the 3D soilbag boundary (red envelope in Fig. 4a). An illustration of the components representing the wrapped GRS on the central cross-sectional plane before the radius expansion is given in Fig. 4a. As shown in Fig. 4b, the contact orientation diagram and the contact number histogram of the wrapped GRS are similar to those for the RV in Fig. 3a. However, the coordination number in Fig. 3a is slightly bigger. This is because the RV has periodic boundary conditions, whereas the wrapped GRS has no contacts supporting its boundary, as shown in Fig. 4a.

To ensure that the filtered packing sufficiently conformed to the boundary, the sphere radii of the soil particles were multiplied by the small factor $\alpha = 1.0001$. A number of DEM calculation cycles were performed to dissipate the disturbances caused by the radius expansion in order to ensure that the packing regains equilibrium after the radius expansion (the ratio of the mean unbalanced force on the particles to the mean force on the interactions is less than 10^{-3}). To prevent the particles from significantly deviating from their original positions, the system was given a global damping coefficient of 0.9 during the DEM cycles. This process, which is classified as “dynamic specimen generation” [31], repeats until the total volume of the Delaunay cells of the packing (note that the triangles shown in Fig. 4a are actually tetrahedrons as this is done in 3D) ceased to grow and the mean overlap ratio decreased to 2.0×10^{-8} (the same level as for the RV in Section 2.2.1).

Fig. 4c shows the contact orientation diagram and coordination number after the radius expansion. The coordination number of the wrapped GRS packing is 2.979, which is slightly smaller than the coordination number of the RV (Fig. 3a) due to the convex boundary of identical spheres. Nevertheless, consistent shapes of the contact orientation diagrams and contact number histograms of these two particle systems can be observed. Fig. 3a, Fig. 4b and c demonstrate that the fabric

characteristics of the original RV were well preserved throughout this 3D-assembling-filtering-expanding process. The filtered packing that contains 68,883 spheres attained a void ratio of 0.675. Because the geotextile nodes (blue spheres in Fig. 4a) were fixed during this process, the produced packing is equally applicable for the PE and PP geotextile cases. It should be noted that the void ratio is computed using the volume of the periodic cell in the PBC. Analogously, the void ratio of the packing is based on the total volume of the Delaunay cells.

2.4 Particle-based stress tensor

The particle-based stress tensor can be computed as

$$\boldsymbol{\sigma}^c = \frac{1}{V} \sum_{N_c} \mathbf{d}^c \otimes \mathbf{f}^c \quad (4)$$

where $\boldsymbol{\sigma}^c$ is the bulk stress tensor, N_c denotes the number of considered contacts on a particle, V is the volume of the Voronoi cell occupied by the particle, and $\mathbf{d}^c$ and $\mathbf{f}^c$ are the corresponding branch vector and force vector, respectively, of the contacts [32]. By weighting the micro-tensors $\boldsymbol{\sigma}$ over the preferred domains, one can easily acquire the relevant global/local macro stress tensor as well as the mean stress p and deviatoric stress q . Because the soilbag packing is derived from the original RV packing in the PBC cells, it is reasonable to base the local p and q values on these RV duplicates. The stress components in Figs. 7, 8, 10, 11 and 16 are computed in such an averaged manner.

3. Results and discussion

3.1 Validation of the soilbag model

The macro responses obtained from the UC and SS tests were used to validate the proposed DEM soilbag model. In the UC test, a triple-layered PE soilbag stack was compressed vertically using a large steel loading plate at a constant compressive strain rate of 0.1%/s. In the SS tests, a single-layered PP soilbag was sheared at a shear strain rate of 0.02%/s with a constant vertical dead load P of 1.33, 1.87, and 2.41 kN. Abrasive paper (grit 40) was firmly attached to the bottom of the loading plate and the top of the test platform to minimize the interface sliding, but the effect became

marginal at high levels of vertical load during the UC test. This can be understood from the similar failure mode (fabric rupture in the middle) in the test (Fig. 6a) and the simulation with a 0° geotextile-plate interface friction angle (Fig. 6b). When a friction angle of 90° is used (an extreme case to eliminate sliding), the fabric ruptures along the seams (Fig. 6c), which differs significantly from the experimental observation (Fig. 6a). In addition, introducing friction on the geotextile-plate interface will affect the confinement of the wrapped GRS. Therefore, the UC simulations excluded such interfacial friction. In the SS simulation, however, the geotextile-plate contacts were given a 90° friction angle to eliminate the interfacial sliding. During initial vertical loading prior to shear, a friction angle of 0° was applied at the interface. To reduce the computational costs, only one soilbag was modeled in the simulations and density scaling [33] was applied according to Table 1. The initial compaction process was left out because its effect on the initial void ratio has been considered by the assumed 3D geometry, as stated in section 3.1.1. The strain rate was held constant at 10%/s in both the UC and SS simulations. Although not shown here, a consistent macro response was obtained with strain rates of 1%/s and 0.1%/s. The UC simulations were incrementally conducted; each step was completed when the soilbag attained a quasi-static state with a negligible level of unbalanced force ratio (10^{-3} as in Ng [34]) and kinetic energy (lower than 10^{-5} J). The setups of the simulations and the tests for model validation are given in Table 2.

Good agreement was observed between the responses in the tests and the simulations with the parameters given in Table 1 (see Fig. 7 for the UC case and Fig. 8 for the SS case). Fig. 7 provides the evolutions of the external axial stress on the soilbag σ_a and the z component of the average stress tensor σ over the GRS domain σ_z using Eq. (3). Because the soilbag was continuously flattened, σ_a' calculated as P over the reference top surface area of the soilbag ($A = 0.16 \text{ m}^2$) is not accurate. The true value was computed in a manner similar to σ_z , except that the averaging took place over the whole soilbag domain. Nevertheless, the stress $\sigma_a' = P/A$ is plotted for direct comparison between test and simulation responses. σ_z constantly exceeds σ_a due to the additional confining stress on the wrapped

GRS from the sustained tension on the geotextile. After the soilbag attained global failure in UC at the axial strain $\varepsilon_a \approx 13\%$, both σ_z and σ_a began to decrease while the level of the lateral stress σ_x and σ_y remained stationary due to the interlocking force between the soil particles and geotextile the nodes. In the test, however, the PE soilbag did not fail in the ε_a range considered. Such difference in bearing capacity can be attributed to the relatively large radii of the geotextile nodes in the simulation ($r_{bag} = 2.5$ mm, 10 times the PE geotextile thickness). When the fabric ruptured as illustrated in Fig. 6b, large interconnecting openings are formed with soil particles consistently being shoved into them. As a result, the GRS volume expanded with a gradual decline in stress level (see Fig. 10c). The first local constituent yarn broke at $\varepsilon_a = 8\%$ in the simulation. An additional 5% was required for the breakages to develop into full openings and the stress in the GRS to reach the peak. In the test, given the thin PE geotextile, more strain would be required for global failure to occur. The initial toe region of the stress-strain relationship in Fig. 7 was another difference between the simulation and test, possibly due to lack of initial compaction. As suggested by O'Sullivan et al. [35], cyclic loading (initial compaction) rearranges the strong force chains to be more anisotropic, which causes hardening of the granular material. Previous experiments have also found that the initial stiffness of soilbags improved with thorough initial compaction.

Furthermore, because of the absence of compaction in the numerical model, the maximum normal strain ε_n was reproduced higher in the SS simulations than in the tests, but the evolution of the stress ratio τ/σ_a (τ is computed as shear force Q over A) matches the test data well (see Fig. 8). Fig. 8 also provides the evolution of the shear-to-tangential stress ratio τ_{zx}/σ_z on the x - y plane of the wrapped GRS using Eq. (3). τ_{zx}/σ_z was found to fall progressively behind τ/σ_a due to the increasing σ_z in the GRS (see Fig. 16) because of the enhancing confinement from the wrapping geotextile. As would be expected, both the stress ratios and axial strain decrease accordingly with greater vertical load.

3.2 Stress path analysis

The particle-based stress tensors were averaged in five consecutive GRS volumes, #1 – 5, aligned

along the x axis. Each consists of four neighboring RVs as shown in Fig. 9. #0 represents averaging over the overall Delaunay volume of the soilbag from which the void ratio is computed.

Fig. 10 shows the stress paths of the selected GRS volumes in the PE and PP soilbags under the UC loading course. In both cases, linear global and local stress paths were observed regardless of the difference in the tensile strength of the two geotextiles. The paths turned slightly nonlinear when partial breakages on the geotextile fabric started to form. After the global failure, the PE and PP soilbags underwent stress paths similar to the unloading of drained triaxial compression. The sustained lateral stress σ_x and σ_y , as shown in Fig. 7, ensured a gentle decrease in the compressive strength of the soilbag. A summary view of the internal force chains and the distribution of mean stress p in the PE geotextile-wrapped GRS at the peak stress state and the final state is shown in Fig. 12. T represents the tensile force in the geotextile yarns. The colors of the force chains indicate the magnitudes of the tangential forces f_t^c , whereas the radii of the force chains' cylinders are proportional to the norms of the normal forces f_n^c . It was confirmed again, as in Tantonio and Bauer [13], that a soilbag under compression has concentrated stress in the middle (#2, #3 and #4 in Fig. 12a). This non-uniformity does not exist initially. It grows with sustained compression until the global failure and then drops gradually as the fabric rupture progresses in the geotextile. At the final state ($\varepsilon_a = 20\%$) the geotextile wears out in the middle part, which corresponds to the failure mode of the soilbag in the test (see Figs. 6a and 12b). The development of large interconnecting openings as shown in Fig. 12b leads to a great loss of strong normal force chains in the vicinity. As a result, all local stress paths collapse to the global one, returning to a uniform stress state.

The stress paths of the PP soilbag under SS are plotted in Fig. 11. Four cases ($P = 2.41, 12.05, 24.1$ and 36.15 kN) are presented here. In each case, the global stress path initially deviates from the stress path under UC, approaching the critical state line (CSL) of the wrapped GRS. Because shearing only dominates in the middle part of the GRS, the global path eventually heads away from the CSL. As the vertical dead load rises, the slope of the CSL of the wrapped GRS decreases gradually and is always

lower than that of the Toyoura sand. As a result, less distortion is produced at the critical state in the wrapped GRS than in the Toyoura sand. The dilatancy of the sand is better contained by wrapping the soil completely in a soilbag.

At the local scale, interesting loading-unloading local stress paths exist in the middle (#2-4) of the GRS. After passing the phase transformation stage (PTS), which coincides with ε_a changing from compressive to dilative, further shear deformation results in a persistent drop in both p and q levels along the CSL. While the GRS in the middle keeps dilating, the particles neighboring the lateral boundaries (#1, #5) are constantly compressed along the compression line. In turn, the global stress state evolves toward a more uniform state and the dilatancy is further restrained. High damping in the soilbag can be attributed to hysteresis energy dissipation induced by the partial loading-unloading stress paths in the middle of the soilbag. These absorbing local stress paths result from the redistribution of internal forces. The transfer of concentrated force chains into the domain near the lateral boundaries during shear is shown in Fig. 13 ($P = 12.05$ kN). Upon the wrapped GRS reaching its shear strength during shear, tension distributes in a pattern opposite to that in the UC condition: the tensile force on the lateral faces is higher than on the top and bottom surface of the geotextile bag. In both loading courses, tension in the yarns was found to be greater along the hoop direction than in the axial direction.

3.3 Volumetric change in compression

The relationships between the void ratio e and mean stress p of the PE and PP geotextile-wrapped GRS under UC are compared in Fig. 10c. The dashed line shows persistent volumetric expansion in the PE soilbag after initial contraction, similar to the volumetric change of dense sand in triaxial compression. However, applying stress paths like those in Fig. 10a and b to the sand should result in volumetric contraction instead. Such a difference can be attributed to the contribution of the confinement mechanism in the soilbag. Unlike compression at all principal directions on the sand specimen in triaxial tests, the confining stress on the soilbag is gained from growing tension in the

geotextile, which is essentially induced by axial compression and lateral extension. In the case of the weaker geotextile (PE), relatively less lateral confinement (larger σ_z/σ_x , or q/p) is produced. When the lateral extension surpasses the axial compression, the wrapped GRS volume expands, as illustrated by the dashed line in Fig. 10c. The stress path in the PE case has a slope ($q/p = 1.65$) close to that of the critical state line ($M_c = 1.76$) of the Toyoura sand at 90% relative density in the triaxial compression condition. In contrast, the deviatoric strain distribution in the x - z cross-sectional plane (see Fig. 14) indicates strain localization starting at a small strain level. The compression stress paths of the GRS wrapped by the weak geotextile (PE) follows the CSL of the Toyoura sand. Because the confinement effect provides the wrapped GRS with additional lateral stress, the stress state of the wrapped GRS evolves along the CSL until the global failure of the geotextile. In this case, it is the progressing granular failure that eventually drives the failure of the soilbag.

The solid line in Fig. 10c shows a completely different failure mechanism. The volume of the PP soilbag was contracted persistently until the geotextile could no longer bear any greater tensile force. Prior to the first fabric rupture, the void ratio remained nearly constant. With the ruptures growing on the top and bottom surfaces of the container, the wrapped GRS gradually expanded. After a sufficient number of large openings were formed, the mean stress p started to drop gradually with more significant volumetric expansion in the wrapped GRS. In this case, it is the sustained flattening of the geometrical shape of the soilbag that caused the failure. The wrapped GRS did not fail during the whole loading course and the stress state was heading away from the CSL. This finding explains how effective the reinforcement is in making use of and constraining the dilatancy of the soil to improve the bearing capacity of the wrapped soil.

3.4 Interlocking in compression and confinement in shear

One of the motivations of this study is to clarify the roles of confinement and interlocking in a soilbag in symmetrical (UC) and asymmetrical (SS) boundary conditions. The following equation can be applied to examine the interfacial interlocking in the DEM soilbag during compression.

$$\Phi_i = 180/\pi \tan^{-1}(\sqrt{\tau_{ij}^2 + \tau_{ik}^2}/\sigma_i) \quad (5)$$

where τ_{ij} and τ_{ik} are a pair of shear stresses on the interface at which the normal stress σ_i is applied, and Φ_i represents the friction angle at the interface perpendicular to axis i . Interfacial stresses defined in this manner are shown in Fig. 15a. Because local shear stress should be symmetrically distributed during UC, averaging over the entire GRS domain would produce shear stress caused by strain localization. Thus, the averaging was carried out over a quarter of the GRS, as shown in Fig. 15a, to calculate the interfacial friction angles.

In the case of the PE soilbag, the interfacial friction angle was considerably smaller than the maximum strength (Fig. 15b and Fig. 5b), whereas the wrapped soil attained its frictional strength at a small strain level and exploited it thereafter with increasing confinement from the wrapping material (Fig. 10a). As shown in Fig. 14, the deviatoric strain at the soil-geotextile interface was not as mobilized as the deviatoric strain in the shear bands of the wrapped GRS. On the x - z or y - z planes, both the soil and the geotextile compatibly deformed at the interface and produced no shear stress.

This finding is reasonable; if the symmetry holds for the GRS and the boundary condition for UC on the wrapped soil is symmetrical, shear stress will never be exerted at the interface. In the simulation, the interlocking force should be negligible when the symmetry remains intact. Fig. 15b shows that only the interfacial friction on the x - y plane developed with axial compression of the soilbag. Starting at a mild level, the friction abruptly increased after passing the failure and then became stagnant. It suggests that the symmetry on the x - y plane does not perfectly hold even at a small strain level, which causes some initial interlocking along the hoop direction. When the geotextile begins to rupture, which triggers the symmetry to break, the interlocking will be significantly enhanced and will work as a secondary mechanism for constraining the swelling of the soil and maintaining effective lateral confinement.

The interface interlocking serves a dominant role in the soilbag in shear. The confinement effect in shear should also be investigated to analyze the roles of the two mechanisms. Fig. 16 shows that the

lateral normal stress σ_x and σ_y nearly doubled when the wrapped GRS approached its shear strength. However, because σ_z was rising with sustained shear, there should be neither a peak nor softening in the τ_{zx} response. Upon reaching the critical condition, the stress state of the wrapped GRS can be approximated as an isotropic stress coupled with a pair of shear stresses in the shear (x - z) plane.

3.5 Fabric anisotropies of the wrapped GRS

The characteristics of the fabric anisotropies in the geotextile-wrapped GRS are investigated based on the studies of Oda [36] and Guo and Zhao [33], which should explain the soilbag reinforcement mechanisms from a microscopic perspective. With the information about the configuration of particles and internal force chains, Eqs. (6)–(8) provide the corresponding fabric tensors for the spatial distribution of the contact normals and the normal and tangential force chains, namely, $\boldsymbol{\varphi}^c$, $\boldsymbol{\chi}^n$, and $\boldsymbol{\chi}^t$. As an analogy to q of the stress tensor, the deviatoric parts of these tensors are used to compute the anisotropies.

$$\boldsymbol{\varphi}^c = \frac{1}{N_c} \sum_{N_c} \mathbf{n}^c \otimes \mathbf{n}^c \quad (6)$$

$$\boldsymbol{\chi}^n = \frac{1}{N_c} \sum_{N_c} \frac{f_n^c \mathbf{n}^c \otimes \mathbf{n}^c}{1 + \mathbf{a}^c : (\mathbf{n}^c \otimes \mathbf{n}^c)} \quad (7)$$

$$\boldsymbol{\chi}^t = \frac{1}{N_c} \sum_{N_c} \frac{f_t^c \mathbf{t}^c \otimes \mathbf{n}^c}{1 + \mathbf{a}^c : (\mathbf{n}^c \otimes \mathbf{n}^c)} \quad (8)$$

where $\mathbf{n}^c$ and $\mathbf{t}^c$ correspond to the unit vectors normal and tangential to the contact plane, respectively; N_c is the total number of contacts that fall into the categories of (a) soil-to-soil, (b) soil-to-geotextile, (c) soil-to-plate (only definable after fabric rupture); and f_n^c and f_t^c are the length of the inter-particle contact forces along the normal and tangential directions, respectively. $\mathbf{a}^c = 15/2 \text{dev}(\boldsymbol{\varphi}^c)$ is the fabric anisotropy tensor due to geometrical change. Analogously, using the averaged contact normal force $\bar{f}^0 = \text{tr}(\boldsymbol{\chi}^n)$, the other two mechanical anisotropy tensors can be defined as $\mathbf{a}^n = 15/2 \text{dev}(\boldsymbol{\chi}^n)/\bar{f}^0$ and $\mathbf{a}^t = 15/3 \text{dev}(\boldsymbol{\chi}^t)/\bar{f}^0$ for the normal and tangential part of the force chains, respectively. Their second invariants can be employed to measure the intensity of anisotropy as defined in Eq. (9).

$$a_* = \text{sign}(S_r) \sqrt{\frac{3}{2} \mathbf{a}^* : \mathbf{a}^*} \quad (9)$$

where $*$ denotes c , n or t and S_r is computed as the inclination between a specific anisotropy tensor and the stress deviator $\text{dev}(\boldsymbol{\sigma})$. For the wrapped GRS, S_r is always positive in either the UC or SS loading condition. The following paragraphs will discuss the relation of these fabric anisotropies to the confinement effect, the compressive and the shear strength of soilbag.

Fig. 17 shows the evolution of the fabric anisotropies a_c , a_n and a_t with deviatoric strain ε_d under UC. For both PE and PP soilbags, a_n and a_t drop gradually with increasing ε_d , indicating an increase of normal and tangential forces in the lateral directions. Macroscopically, this results in rising lateral confining stresses σ_x and σ_y . Fabric anisotropy due to the geometrical configuration in the soilbag has a tendency similar to that in conventional triaxial loading conditions. In both cases, a_c becomes stagnant when some yarn starts to break partially, as illustrated in Fig. 12a, and drops down mildly after some interconnecting ruptures start to develop, as shown in Fig. 12b. Although similar tendencies can be found regardless of the tensile strength in use, the magnitude of each fabric anisotropy is lower in the case of the stronger PP geotextile. With a strong geotextile, relatively more and stronger lateral force chains (lower a_c and a_n) exist on the wrapped GRS. At the macro scale, this results in greater confining stress and eventually reduces the slope of the stress path under the UC condition.

Fig. 18 shows the evolution of the fabric anisotropies a_n and a_c with shear strain γ under SS. For all considered SS simulations, a_n dropped gradually with sustained γ after passing the PTS even though a constant vertical load was maintained in these cases. The geometrical fabric anisotropy a_c has a similar tendency to that of a soil specimen in triaxial compression DEM simulations [32]. In both cases, a_c becomes stagnant when the wrapped soil reaches its shear strength. The critical value of a_c in granular material is closely related to the material's shear strength. For geotextile-wrapped GRS, the critical a_c should be determined from both the geotextile's tensile stiffness and the soil property. Fig. 18 also gives a group of fabric anisotropy responses under a variety of vertical dead loads. In addition

to the variation of a_c with shear deformation, the critical a_c was found to be reduced with higher levels of compressive load, suggesting that the critical a_c is a state parameter related to the stress history as well. A similar pattern exists among the responses of the mechanical fabric anisotropy a_n under different vertical loads. The confinement of the geotextile-wrapped GRS develops in not only compression but also shear. Greater compression prior to shear will enable more isotropic confinement to be attained on the wrapped GRS during shear.

4. Conclusions

This paper presents a novel approach for the discrete modeling of soilbags. New knowledge is provided for clarifying the reinforcement mechanisms of the geosynthetic-wrapped soil from the perspectives of stress state, volumetric change and anisotropies. Two types of woven geotextile (PE and PP) soilbags were modeled using the DEM. The parameters of the contact laws for the soil, the geotextile and their interface were carefully calibrated against triaxial, wide-width tensile and shear box test results. With a properly assumed 3D geometry for the geotextile container, the wrapped soil was generated in a 3D-assembling-filtering-expanding process, using the same RV as for the calibration of the contact law. The responses of the soilbag in the DEM simulations and the tests in UC and SS were consistent. Using this new approach of soilbag modeling, the evolution of fabric rupture in the geotextile can be observed and a close relationship between the fabric anisotropies and the reinforcement effect can be attained. The main findings regarding the stress states and the fabric anisotropies of a soilbag are as follows:

- Under unconfined compression, both global and local stress paths are linear at the pre-failure state with greater p and q values in the middle of the wrapped soil. At the post-failure stage, the lateral confining stress remains nearly constant, which leads to a gentle decline in the bearing capacity. The soil-geotextile interfacial shear stresses are found to be higher on the lateral sides than on the top and bottom sides, but each of them is greatly below the maximum for mobilizing the interface.
- The global stress path in simple shear approaches the critical state line of the wrapped soil first and

then heads away with lower q and greater p . Partial loading-reloading paths are experienced in the middle of the wrapped soil, which explains the high damping of the soilbag. As the vertical dead load rises, the slope of the critical state line decreases, indicating a better control on the dilatancy of the soil.

- The soilbag constrains dilatancy differently depending on the tensile stiffness of the geotextile. For the soil wrapped by the strong geotextile (PP), the numerical predictions show that the critical state is never reached during compression and the volume does not dilate until the geotextile starts to fail. For the weak geotextile (PE), the soil reaches the critical state with sufficiently developed shear bands and dilatancy. However, the stress path progresses along the critical state line due to the sustained confinement on the volume of the wrapped soil.
- The decreasing mechanical fabric anisotropies in either loading course suggest that confinement is the primary mechanism of the soilbag reinforcement method. A stronger geotextile enables more isotropic confinement. The results on interfacial shear indicate that the interlocking effect takes a secondary role in prolonging the effective lateral confinement, particularly during the post-failure stage of compression. During simple shear, the confinement of the wrapped soil grows increasingly isotropic with greater shear deformation and a higher level of vertical dead load.

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Tables

Table 1 Micro-parameters for Hertz-Mindlin contact model.

	Young's modulus (GPa)	Poisson's ratio	Rolling/twisting stiffness (N·m/rad)	Scaled density (10 ³ kg/m ³)	Inter-particle friction angle (°)
Soil	4	0.33	0.13	2650	29
Geotextile	8	0.33	0.13	444	PP: 21 PE: 25
Plate	200	0.33	0.13	7850	UC: 0 SS: 90

Table 2 Setup of simulations and tests for model validation.

		Strain rate	Num. of soilbags	Initial compaction	Geotextile-plate interface	Geotextile type
Test	UC	0.1%/s	Three	With	Sandpaper (grit 40)	PE
	SS	0.02%/s	One	With	Sandpaper (grit 40)	PP
Simulation	UC	10%/s + Stress increment	One	without	Smooth (0°)	PE
	SS	10%/s	One	With	Fixed (90°)	PP

Figures

(a)

(b)

(c)

Fig. 1 (a) Discretization of the woven geotextile container, (b) representative laboratory soilbag after initial compaction, and (c) assumed cross-sections with dimensions of mm.

Fig. 2 (a) Contact orientation diagram and coordination number of the particle assembly prior to loading, and (b) response in drained triaxial compression compared with test data of Toyoura sand.

Fig. 3 Characteristics of PE and PP woven geotextiles: (a) woven fabric structure and (b) tensile behavior.

Fig. 4 (a) Components that represent the wrapped GRS in the central x - z plane, and corresponding contact orientation diagram and coordination number (b) before and (c) after radius expansion.

Fig. 5 (a) Schematic of the shear box simulation and (b) interfacial response in experimental tests and simulations with different soil-geotextile interface friction angles.

Fig. 6 Failure modes in (a) test and simulations with (b) 0° and (c) 90° interface friction angle for geotextile-plate contacts.

Fig. 7 Stress-strain relationship of a PE soilbag in UC test (partially cyclic) and simulation.

Fig. 8 Response of (a) the tangential-normal stress ratio and (b) the normal strain of a PP soilbag in SS tests and simulations.

Fig. 9 Illustration of the five selected volumes for probing local stress paths.

Fig. 10 Global and local stress paths of the soil wrapped by (a) PE and (b) PP geotextile, and (c) the e - $\log(p)$ relations of both types of GRS in UC simulations.

Fig. 11 Global and local stress paths of the soil wrapped by PP geotextile in SS with a vertical dead load P corresponding to: (a) 2.41 kN, (b) 12.05 kN, (c) 24.1 kN, and (d) 36.15 kN.

Fig. 12 Force chain, p distribution in the wrapped soil and T distribution in the geotextile of a PE soilbag for (a) peak stress state, (b) final state in UC.

(a)

(b)

Fig. 13 Force chain, p distribution in the wrapped soil and T distribution in the geotextile of a PP soilbag for (a) initial state, (b) final state in SS ($P = 12.05$ kN).

Fig. 14 Deviatoric strain distribution on the central x - z cross-section plane of a PE soilbag in UC.

Fig. 15 (a) A quarter of the GRS domain and corresponding stress components, (b) evolution of friction angles at the soil-geotextile interfaces of a PE soilbag in UC.

Fig. 16 Average stress components over the entire GRS domain of a PP soilbag with a vertical dead load P that corresponds to (a) 1.33 kN (b) 1.87 kN, and (c) 2.41 kN.

Fig. 17 Evolution of fabric anisotropies a_n , a_t and a_c of PE and PP soilbag in UC.

Fig. 18 Evolution of fabric anisotropies a_n and a_c of a PP soilbag in SS.