Strength Characteristics of Reinforced Granular Soils in Three-Dimensional Space

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Abstract. A series of true triaxial numerical tests were carried out on reinforced granular soils based on DEM method, effects of reinforcement layers and intermediate principal stress ratio on the strength characteristic of granular soils were conducted. The simulation results show that the shear strength of reinforced soils can be improved with the increase of reinforcement layers. However, the improvement effect becomes weak with increase of the intermediate principal stress ratio. Furthermore, the shear strength of granular soils will decrease with the increase of intermediate principal stress ratio b under constant P stress path, the maximum shear strength can be obtained in case of b=0 test, while the minimum shear strength appears under b=1.0 condition.

Keywords. Reinforced granular soils, shear strength, reinforcement layers, intermediate principal stress ratio.

1. Introduction

Reinforced soils are widely used in civil engineering for their various applications like soil retention walls and road subgrades due to their cost economy and fast construction, quantities of lab tests have been carried out to study the strength characteristic of reinforced soils. In the early stage, reinforcements were mostly geotextiles, Gray and Al-Refaei [1] investigated the stress-strain response of a sand reinforced with fabric layers by a series of triaxial compression tests, results showed that the shear strength and deformation resistance can be improved by incorporating tensile inclusions into sand. Haeri [2] utilized geotextiles within beach sand to study the effect of reinforcement layers and types on the strength and deformation characteristics of soils. Noorzad [3] carried a series of unconsolidated and undrained tests to study the mechanic characteristics of geotextile-reinforced clay. At present, some natural reinforced materials like plant roots, agricultural wastes and fibers are used as reinforcements, Chen [4] studied the stress-deformation response of roots-reinforced clay. Hao [5] used wheat straw to reinforce clay and investigated the effect of wheat straw contents on the shear strength of soils.

It should be noted that the above tests are carried out under triaxial condition. However, the real soil is in 3-dimensional stress state, it is necessary to study the strength and deformation characteristics in 3-dimensional stress space. For this purpose, a series of numerical true triaxial tests were carried out on isotropically compressed assemblies with and without reinforcement layers.
2. Stress Path

In true triaxial test, the dimensionless parameter \( b \) refers intermediate principal stress ratio \([6]\), it is defined by three principal stress \( \sigma_1, \sigma_2 \) and \( \sigma_3 \): \( b = (\sigma_2 - \sigma_3) / (\sigma_1 - \sigma_3) \). In this paper, constant \( P \) and constant \( b \) test was used, \( P \) denotes the mean principal stress: \( P = (\sigma_1 + \sigma_2 + \sigma_3) / 3 \), thus, the following equation can be derived:

\[
\begin{align*}
    b &= \frac{\sigma_2 - \sigma_3}{\sigma_1 - \sigma_3} \\
    P &= \frac{\sigma_1 + \sigma_2 + \sigma_3}{3} \\
\end{align*}
\]

\[
\begin{align*}
    \sigma_1 &= \sigma_r + b\sigma_i \\
    \sigma_2 &= \frac{(3 - b)P + (2b - 1)\sigma_1}{2 - b} \\
    \sigma_3 &= \frac{3P - (1 + b)\sigma_1}{2 - b} \\
\end{align*}
\]

(1)

3. Testing Programme

A serious of true triaxial numerical tests with \( b \) varies from 0 to 1 under different reinforcement layers (0, 1, 2, and 3) were performed. Details of the test programme are shown in table 1.

| Reinforcement layers \( n \) | Confining pressure (kPa) | Intermediate principal stress ratio \( b \) |
|------------------------------|--------------------------|------------------------------------------|
| 0                            | 200                      | 0, 0.2, 0.4, 0.6, 0.8, 1.0               |
| 1                            | 200                      | 0, 0.2, 0.4, 0.6, 0.8, 1.0               |
| 2                            | 200                      | 0, 0.2, 0.4, 0.6, 0.8, 1.0               |
| 3                            | 200                      | 0, 0.2, 0.4, 0.6, 0.8, 1.0               |

4. Sample Preparation

A virtual cubic sample 70 cm by 70 cm by 140 cm with six rigid boundaries and around 12000 particles was generated. Reinforcement layers were modeled by assigning Rolling Resistance contact model between particles. Details of granular soils and reinforcement parameters are presented in table 2. The PFC\(^{3D}\) model of unreinforced (space pattern) and reinforced (plan view) samples are shown in figure 1.

| Assemble type | Contact model       | Young’s modulus (MPa) | Porosity | Particle size (mm) | Normal stiffness | Friction coefficient |
|---------------|---------------------|-----------------------|----------|--------------------|------------------|---------------------|
| Sample        | Linear              | 20                    | 0.4      | 1.8–2.5            | 1E8              | 0.4                 |
| Reinforcement layer | Rolling resistance | 50                    | 0.4      | 1.8–2.5            | 1E10             | 10                  |

**Figure 1**. PFC\(^{3D}\) model of unreinforced and reinforced samples.
5. Results and Discussions
In this section, the results of the true triaxial numerical tests are presented and discussed. The effects of intermediate principal stress ratio and reinforcement layers on deviatoric stress and friction angle are evaluated in detail.

5.1. Deviatoric Stress
In three-dimensional stress space, deviatoric stress is defined by three principal stresses $\sigma_1, \sigma_2$ and $\sigma_3$:

$$q = \left(\sqrt{2}/2\right)\left[(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2\right].$$

The simulated deviatoric stress versus axial strain curves for different $b$ values and reinforcement layers at a confining pressure of 200 kPa are shown in figure 2. It can be observed that the reinforcement layer has an insignificant effect on the strength characteristic of reinforced granular soils. The deviatoric stress increases with the increase of reinforcement layers for $b$ varies from 0 to 1, furthermore, when $b$ less than 0.8, the effect of reinforcement layers is more obvious, while the reinforcement effect decreases in cases of $b=0.8$ and $b=1.0$, especially for the 3-layer reinforced sample, the stress strain curve is approximately overlap with the 2-layer reinforced sample. This can be explained by the concept of stress path, $b=0$ corresponding to triaxial compression test, in this case, the shear-resistant capacity of reinforce materials can be fully used, however, $b=1$ is considered as triaxial tensile test, in this case, the major principal stress equals to the intermediate principal stress, the soil sample will fail under the effect of squeeze force, thus the shear-resistant capacity of the reinforce material can’t be fully used.

![Figure 2. Deviatoric stress-strain relationships for a confining pressure of 200 kPa.](image-url)
Figure 3 shows the relationship between peak deviatoric stress and intermediate principal stress ratio, it can be conducted that shear strength will decrease with the increase of intermediate principal stress ratio \( b \), the maximum shear strength can be obtained in case of \( b=0 \) test, while the minimum shear strength appears under \( b=1.0 \) condition, figure 4 presents the variation of peak deviatoric stress with reinforcement layers, as we can see, the peak deviatoric stress increases linearly with the increase of the reinforcement layer, which indicates that the shear strength of reinforced soil will increase with the increase of reinforcement layers, however, the improvement effect becomes weak with increase of the intermediate principal stress ratio.

5.2. Friction Angle
Friction angle is an important parameter to determine the strength of soils, it can be affected by many factors such as soil types, particle shapes and water contents etc., [7, 8], in this paper, we use \( \varphi \) to denote the friction angle, which can be calculated by the principal stresses [9] and the sine of \( \varphi \) is expressed as \( \sin \varphi = (\sigma_2 - \sigma_3) / (\sigma_1 + \sigma_3) \). As is shown in figure 5, with the increase of \( b \) value, the peak friction angle of reinforced sample increases first and then shows a slightly decreased trend when \( b \) is close to 1.0, and the 3-layer reinforced sample shows the maximum peak friction angle. Figure 6 shows the plot of friction angle versus reinforcement layers for \( b \) varies from 0 to 1, we can conduct that the friction angle and reinforcement layers present a linearly increased relationship, in addition, under a specific reinforcement layer, the maximum friction angle is observed in cases of \( b=0.6 \) and \( b=0.8 \).
5.3. Failure Surface
Figure 7 shows the failure pattern and contact force chain of four samples at 20% axial strain, it can be observed that the unreinforced soil sample appears a shear band which shows a degree of $45^\circ + \varphi/2$ with the horizontal direction under the shear stress, however, we can’t obtain obvious shear band for reinforced soil samples, which indicates that horizontal reinforcements will prevent the formation of failure surface, moreover, the reinforcement will uniformly reattribute the shear stress inside the soil sample, thus the shear strength of the soil is improved.

![Ball displacement and Contact force chain](image)

**Figure 7.** Failure patterns and contact forces chains.

6. Conclusion
By conducting numerical true triaxial tests on unreinforced and reinforced granular soils the following results were obtained:

1. The shear strength of granular soils decreases with the increase of intermediate principal stress ratio $b$ under constant $P$ stress path, moreover, the shear strength of reinforced soils can be improved with the increase of reinforcement layers, however, the improvement effect becomes weak with increase of the intermediate principal stress ratio.

2. The friction angle of granular soils increases as the reinforcement layers, under a specific reinforcement layer, the maximum friction angle is observed in cases of $b=0.6$ and $b=0.8$.

3. Reinforcements will prevent the formation of failure surface and uniformly reattribute the shear stress inside the soil sample, thus the shear strength of the soil is improved.

Acknowledgments
Authors wishing to acknowledge the financial support provided by the National Natural Science Foundation of China (NO.51879212,41630639), key research and development program of Shaanxi Province(2019KWZ-09), and Hubei Key Laboratory of Disaster Prevention and Mitigation (three gorges university) (2016KJZ02).

References
[1] Gray D H A T 1986 Behavior of Fabric-Versus Fiber-Reinforced Sand *Journal of Geotechnical Engineering* 8(112) 804-820
[2] Haeri S M et al. 2000 Effect of geotextile reinforcement on the mechanical behavior of sand *Geotextiles and Geomembranes* 18(6) 385-402
[3] Noorzad R and Mirmoradi S H 2010 Laboratory evaluation of the behavior of a geotextile reinforced clay *Geotextiles and Geomembranes* 28(4) 386-392
[4] Chen C F et al. 2007 Study on grassroots-reinforced soil by laboratory triaxial test *Rock and Soil Mechanics* 28(10) 2041-2045
[5] Hao J B et al. 2019 Strength characteristics and mesostructured of wheat straw reinforced soil
Journal of Tongji University 47(6) 764-768

[6] Abelev A V and Lade P V 2003 Effects of Cross Anisotropy on Three-Dimensional Behavior of Sand
Journal of Engineering Mechanics 129(2) 160-166

[7] Tsomokos A and Georgiannou V N 2010 Effect of grain shape and angularity on the undrained response of fine sands Canadian Geotechnical Journal 47(5) 539-551

[8] Sukumaran B and Ashmawy A K V N 2001 Quantitative characterisation of the geometry of discrete particles Geotechnique 51(7) 619-627

[9] Lade P V et al. 2008 Shear banding and cross-anisotropic behavior observed in laboratory sand tests with stress rotation Canadian Geotechnical Journal 45(1) 74-84