GEOSYNTHETIC-ENCAPSULATED GRANULAR COLUMN BEHAVIOR

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Abstract: A series of triaxial compressive tests was carried out to investigate the response of granular columns encapsulated with flexible reinforcements. The tests consist of performing triaxial compressive testing on a sand column encapsulated by a sleeve fabricated from three different geotextiles. The increase in deviatoric stress, the reductions in volumetric and radial strains and the increase in confining pressure corresponding to the encapsulating reinforcement are measured and analyzed. The mobilized pseudo cohesion and friction angle corresponding to various strains are introduced to interpret the reinforcing effect. The response of the columns was analyzed using an analytical procedure. The experimental results reveal that: (1) Geotextile encapsulation enhances the axial strength and impedes the volumetric strain of the specimen. The reinforced specimen exhibits significant apparent cohesive strength. A marked increase in apparent cohesion is noted for sand specimens reinforced using high stiffness geosynthetics. (2) At greater axial strain, the volumetric strain is not sensitive to the chamber pressure. The magnitude of the confining stress provided by the encapsulating geotextile varies slightly for different confining pressures. (3) The mobilized friction angles for reinforced specimens increases gradually with the increase in axial strain, whereas the angle for pure sand reaches a constant angle. (4) The mobilized pseudo cohesion increases linearly with the increase in axial strain and the stronger geotextile has a pronounced effect on pseudo cohesion.

Keywords: encasement, geotextile, laboratory test, reinforcing effect, stone column, strength.

INTRODUCTION

Granular columns were introduced into engineering practice to improve the bearing capacity of a weak or soft soil, and to reduce settlement for foundations resting on weak soil (Bergado et al. 1991, 1992; Raithel et al. 2002; Kempfert 2003). This method is considered one of the most versatile and cost effective techniques for improving in-situ ground conditions. When vertical and corresponding lateral deformations in the granular column occur under a vertical load, the squeezed surrounding soil stratum responds to column expansion by providing additional confining stress to the column. However, most granular columns fail from bulging near the top due to insufficient lateral support (Hughes and Withers 1974; Madhav and Miura 1994). Therefore, granular column reinforcement was introduced to the column, especially to the top section of the column. Column encapsulation with a sleeve fabricated from flexible geotextiles is effective in providing lateral confinement to the column (Rao and Bhandari 1977; Alamgir 1989; Broms 1995; Nods 2002; Sharma et al. 2004; Ayadat and Hanna 2005). The reinforcing effects are verified through laboratory triaxial tests and theoretical analysis (Gray and Al-Refai 1986; Chandrasekaran et al. 1989; Al-Joulani 1995; Haeri et al. 2000; Raithel and Kempfert 2000; Sivakumar et al. 2004; Ayadat and Hanna 2005; Murugesan and Rajagopal 2006; Wu et al. 2008). Testing and analytical results demonstrated that reinforcement increases peak strength, axial strain at failure and reduces the post-peak strength loss.

An analytical procedure is proposed to analyze the response of granular columns encapsulated with flexible reinforcement. The proposed procedure can account for granular material subjected to continuous increase in lateral pressure. A series of laboratory experimental tests were carried out to investigate the response of reinforced granular columns. The test series consists of performing triaxial compressive tests on granular columns encapsulated by sleeves fabricated from three different geotextiles. The consequent reduction in radial strain and increase in confining pressure are analyzed and presented in this paper.

METHODS OF ANALYSIS

The reinforced granular column is composed of three constituent materials; the in-situ soil that surrounds the column, the granular material that fills the column and the reinforcing material that encapsulates the column. The mechanical characteristics of these materials and the in-situ environment govern the behaviour of the composite. Models used to describe the mechanical characteristics of these materials and analytical procedures are briefly presented in this section.

Axial stress-lateral strain relationship of granular material

Using the hyperbola function for the stress-strain relation of granular material, the axial stress σ_1 corresponding to axial strain ε_1 can be expressed in an incremental form as (Duncan and Chang 1970).

$$\sigma_1=\sum_{i=0}^{m-1}\frac{\varepsilon_1}{m}\left[1-R_f\,\frac{(\sigma_{1,i}-\sigma_3)(1-\sin\phi)}{2c\cos\phi+2\sigma_3\sin\phi}\right]^2 KP_a\left(\frac{\sigma_3}{P_a}\right)^n \tag{1}$$

in which c and ϕ are the cohesion and frictional angle of the granular soil, σ_3 and σ_1 are the principal stresses and P_a is the reference atmospheric pressure, R_f, K and n are constants of the granular material to be determined experimentally, and m is the total number of strain increments.

As the reinforced granular column compresses vertically it also undergoes radial or lateral deformation. This lateral deformation stretches the reinforcing sleeve and squeezes the surrounding soil. The squeezed soil and the stretched sleeve in turn restrain column expansion and develop additional confining pressures acting on the column. Thus, the magnitude of the confining pressure increment depends on the lateral deformation of the column. Because Poisson's ratio and lateral deformation for granular soil subjected to continuously increasing confining pressure are difficult to measure, a normalized relation proposed by Wong (1990) is introduced to describe the relationship between the soil volumetric change and axial strain. The normalized parameter function is represented by a hyperbolic function of axial strain as

$$\kappa(\varepsilon_1) = \left(\frac{\sigma_1}{\sigma_3}\right)\left[1 - \chi\left(\frac{d\varepsilon_V}{d\varepsilon_1}\right)\right]^{-1} = R_0 + \frac{\varepsilon_1}{a' + b'\varepsilon_1} \tag{2}$$

where $\kappa(\varepsilon_1)$ is a function of the axial strain, χ is a normalized parameter, R_0, a' and b' are experimentally determined constants.

Volumetric strain under principal stresses σ_1 and σ_3 is obtained by summing the volumetric increments as

$$\varepsilon_V = \sum_{i=0}^{m-1}\left(\frac{\sigma_1-\sigma_3}{m}\right)\left\{\left[1 - R_f\frac{\frac{i}{m}(\sigma_1-\sigma_3)(1-\sin\phi)}{2c\cos\phi + 2\sigma_3\sin\phi}\right]^2 KP_a\left(\frac{\sigma_3}{P_a}\right)^n\right\}^{-1}\left(\frac{1}{\chi}\right)\left[1 - \frac{\left(\frac{\sigma_1}{\sigma_3}\right)_{,i}}{R_0 + \frac{\varepsilon_{1,i}}{a' + b'\varepsilon_{1,i}}}\right] \tag{3}$$

For a reference granular column length Δl_0, volumetric strain of a deformed column is

$$\varepsilon_{V,\Delta l_0} = \frac{-\Delta V}{V_0} = \frac{-\pi\Delta l_0\left[a_1^2\left(1-\varepsilon_{1,\Delta l_0}\right) - a_0^2\right]}{\pi\Delta l_0 a_0^2} = 1 - \left(1-\varepsilon_{1,\Delta l_0}\right)\left(\frac{a_1}{a_0}\right)^2 \tag{4}$$

where a_1 and a_0 are the radii of the deformed and initial column, respectively. Δl_0 is a reference length, V_0, ΔV and $\varepsilon_{1,\Delta l_0}$ are the initial granular column volume, volume change and axial strain for the reference length Δl_0.

The deformed radius a_1 can then be written as

$$a_1 = a_0\sqrt{\frac{1-\varepsilon_{V,\Delta l_0}}{1-\varepsilon_{1,\Delta l_0}}} \tag{5}$$

Since the confining pressures induced by the in-situ soil and the encapsulating sleeve depend on the extent of column expansion, determining the deformed column radius therefore enables evaluating the confining pressures from the surrounding soil as well as the circumferential strain of the encapsulating sleeve.

Cavity expansion of the in-situ clay

A granular column installed in soft soil is initially subjected to at-rest earth pressure. When an axial load is applied to the column the confining pressure increases as a reaction to the column expansion. The relation between the confining pressure and the lateral squeezing of the surrounding soil is developed from an analogy to a cylindrical cavity expansion in the soft soil.

As the surrounding soil is subjected to lateral pressure from the cavity, the relation between the radial pressure and cavity radius during the elastic stage is

$$a_1 = a_0\left[1 + (p_h - p_0)\left(\frac{1+\nu_c}{E_c}\right)\right] \tag{6}$$

Where p_h is the earth pressure acting on the cavity wall corresponding to the cavity radius of a_1, p_0 is the earth pressure acting on the cavity wall at the initial stage. E_c and ν_c are the modulus of elasticity and Poisson's ratio of the soil, respectively.

A plastic zone forms around the inner cavity wall as the cavity pressure further increases. The relation between the cavity pressure p_h and deformed radius a_1 for cohesive soils (frictional angle and the dilatant angle are zero) can be written as

$$p_h = (p_0 + c_u) + c_u \ln\left\{ \frac{E_c}{2c_u(1+\nu_c)} \left[1 - \left(\frac{a_0}{a_1} \right)^2 \right] \right\} \tag{7}$$

where c_u is the un-drained cohesion of the cohesive soil.

Confining stress induced by the circumferential strain of the expanded sleeve

The magnitude of the sleeve hoop stress depends on the circumferential strain and modulus of the sleeve. The sleeve hoop stress in turn exerts a radial confining stress on the encapsulated granular column, which mobilizes its compressive strength and resistance to further deformation in an interactive or synergistic manner. Figure 1 shows a reference length Δl_0 of the reinforced granular column and free-body of section A-A. The force equilibrium in the y-direction yields

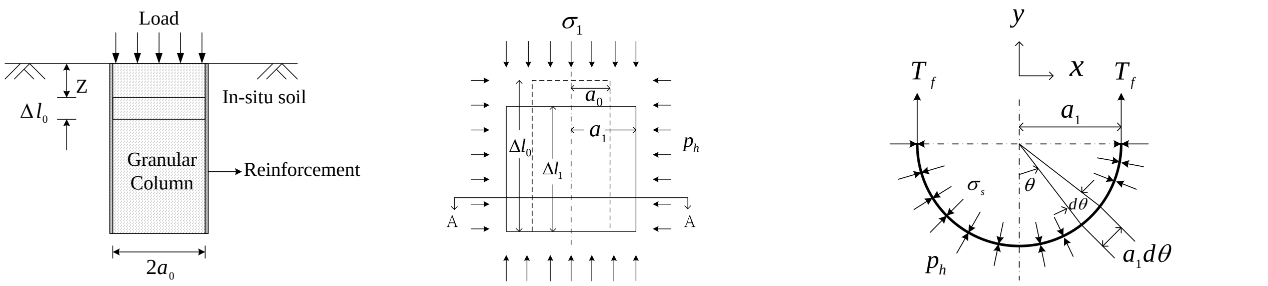
$$\int_{-\frac{\pi}{2}}^{\frac{\pi}{2}} (\sigma_s - p_h)\cos\theta (a_1 \Delta l_1 d\theta) - 2T_f \Delta l_1 = 0 \tag{8}$$

where σ_s is the total confining pressure acting on the periphery of the reinforced column. Δl_1 is the deformed reference column length, and T_f is the circumferential tensile force per unit length of reinforcing sleeve.

The lateral deformation of the column is consistent with the cavity expansion, hence the radial stress acting on the column σ_s is a combination of σ_f and p_h . The confining pressure induced by the expanded sleeve thus can be rewritten as

$$\sigma_f = \sigma_s - p_h = E_f \left(\frac{1}{a_0} - \frac{1}{a_1} \right) \tag{9}$$

When the reinforcing sleeve yields from column expansion, the relation between the sleeve-induced confining pressure and the column radius is



(a) Reinforcement encapsulating the granular column (b) Deformation of a reference column length (c) Free-body of cross section A-A

Figure 1. Schematic of a reinforced granular column

$$\sigma_{fp} = E_f \left(\frac{1}{a_0} - \frac{1}{a_{fp}} \right) \tag{10}$$

where σ_{fp} is the confining pressure for sleeve reaching tensile yield strength T_{fp}, and $a_{fp} = \left(\frac{T_{fp}}{E_f} + 1 \right) a_0$ is the corresponding deformed radius of the column.

EXPERIMENTAL PROGRAM

The experimental program consists of performing triaxial compression tests on 140 mm high x 70 mm diameter samples of dry sand encapsulated in three types of geotextile sleeves. The sand has a specific gravity of G_s = 2.65, maximum dry unit weight of γ_{max} = 16.5 kN/m^3, minimum dry unit weight of γ_{min} = 13.7 kN/m^3. The triaxial tests were carried out on sand compacted to 60% relative density. The triaxial test results for the pure sand specimens are depicted in Figure 2. The reinforcing sleeve was made by sewing a piece of geotextile sheet 140 mm x 240 mm into a cylinder 140 mm in height and 70 mm in diameter. The geotextile tensile tests were carried out on 200 mm wide 100 mm long specimens. The tensile force-strain relations of the three test geotextiles are presented in Figure 3.

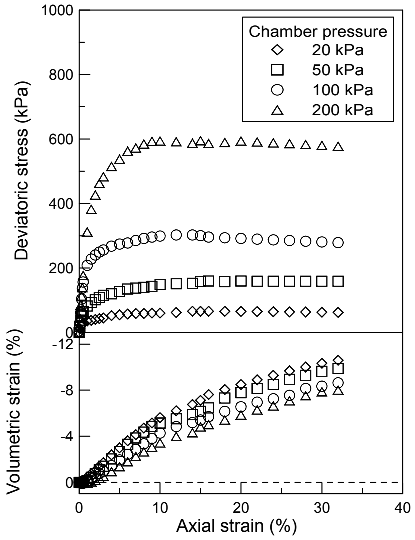


Figure 2. Triaxial test results for unreinforced sand specimen

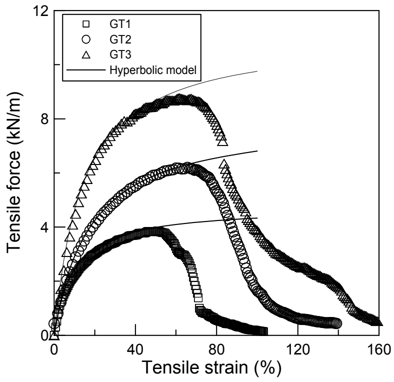


Figure 3: Tensile force-strain relations of reinforcements

EXPERIMENTAL RESULTS

The cylindrical sand specimen is strengthened by the external encapsulation. The axially loaded reinforced specimen expands laterally, which results in an increase in the circumferential strain on the sleeve. The circumferential stress in the sleeve together with the chamber pressure provides the confining pressure to the sand specimen. The experimental results for sand specimens wrapped using three different geotextiles are presented and discussed in this section.

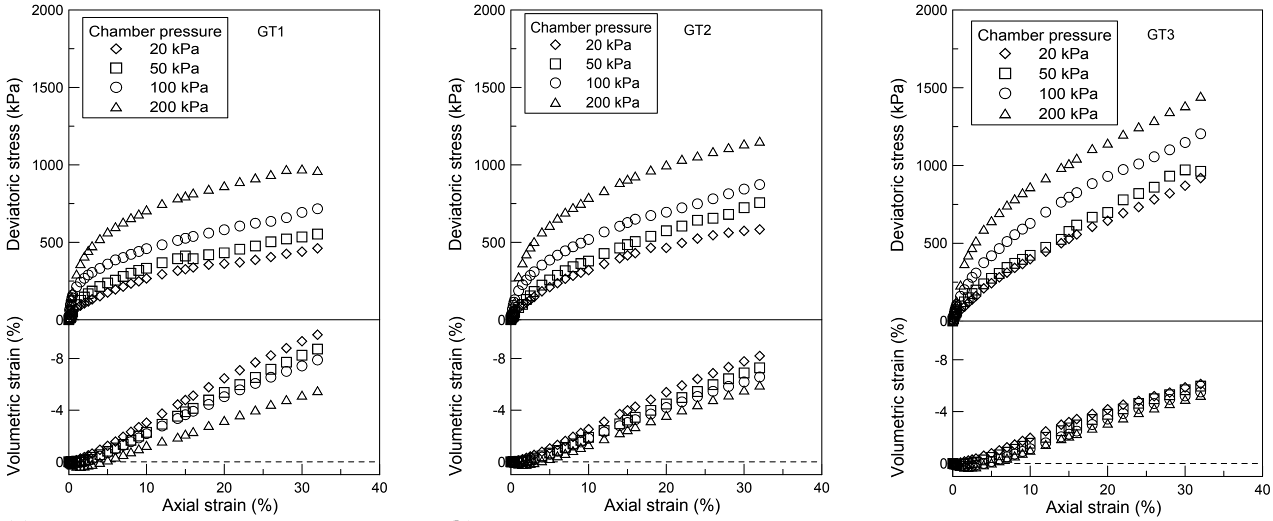
Reductions in volumetric strain and radial strain

The deviatoric stress-axial strain-volumetric strain relations for the reinforced sand specimens are depicted in Figure 4. The encapsulating sleeve increases the strength and impedes the volumetric strain of the sand specimen. The effects are more pronounced for stronger geotextiles. A volumetric strain reduction percentage $(\Delta\varepsilon_V)/(\varepsilon_V)_U$ defined as the ratio between the reduction in volume strain and the volumetric strain of the pure specimen is introduced to evaluate the effectiveness of volumetric reduction by the reinforcing sleeve. The volumetric strain reduction percentage shown in Figure 5 reveals that stronger geotextile has greater suppression in specimen's volumetric strain. Reinforcing sleeve has greater effect in reducing volumetric strain for reinforced specimen subjecting to greater chamber pressure. But the variation in volumetric suppression with different chamber pressures is trivial at greater axial strain.

A radial strain reduction percentage $(\Delta\varepsilon_r)/(\varepsilon_r)_U$ is defined as the ratio between the reduction in radial strain and the radial strain on the pure specimen. Figures 6 and 7 exhibit the radial strain and the radial strain reduction percentage for the reinforced specimens. The results reveal that the stronger geotextile provides more confinement to the sand specimen. The radial strain reduction subsides at greater axial strain. The geotextile has greater degree of lateral strain suppression for a reinforced specimen subjected to lower chamber pressure.

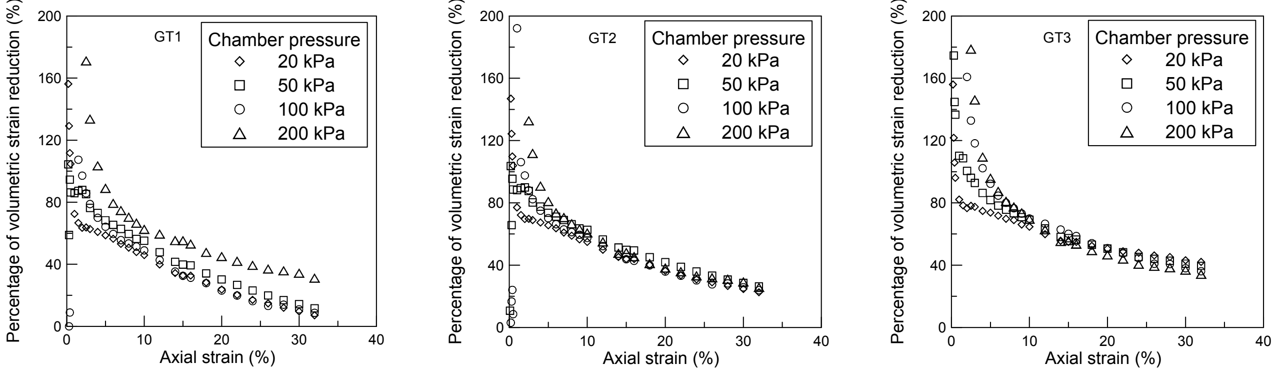
Increases in deviatoric stress and confining pressure

In response to the column expansion, the stretched sleeve creates additional confinement to the column. Therefore, the granular material is subjected to monotonically increasing confining pressure during axial loading, which makes the column stiffer during the subsequent loading. The results in Figure 4 display the increases in deviatoric stress due to reinforcing sleeve application. The deviatoric stress ratio $(\Delta\sigma_d)_R/(\Delta\sigma_d)_U$, defined as the ratio of deviatoric stresses



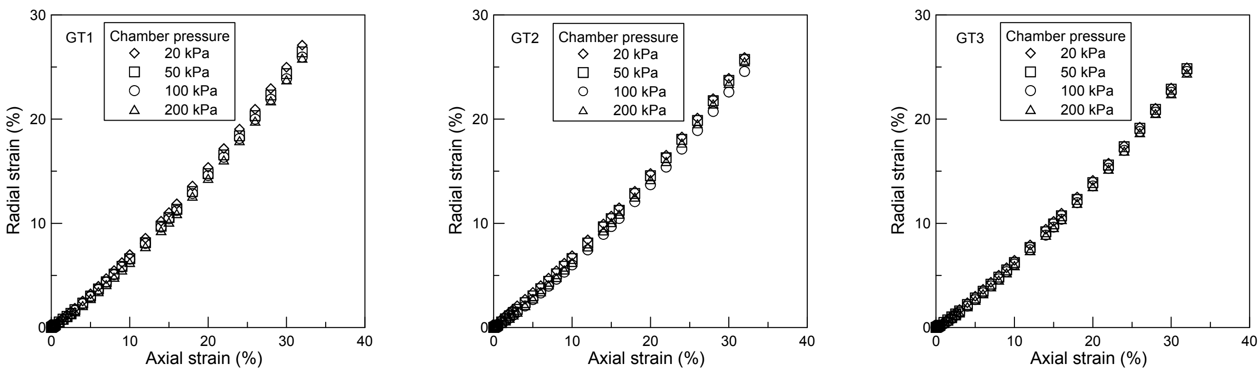
(a) GT1 (b) GT2 (c) GT3

Figure 4. The deviatoric stress-axial strain-volumetric strain for reinforced specimens



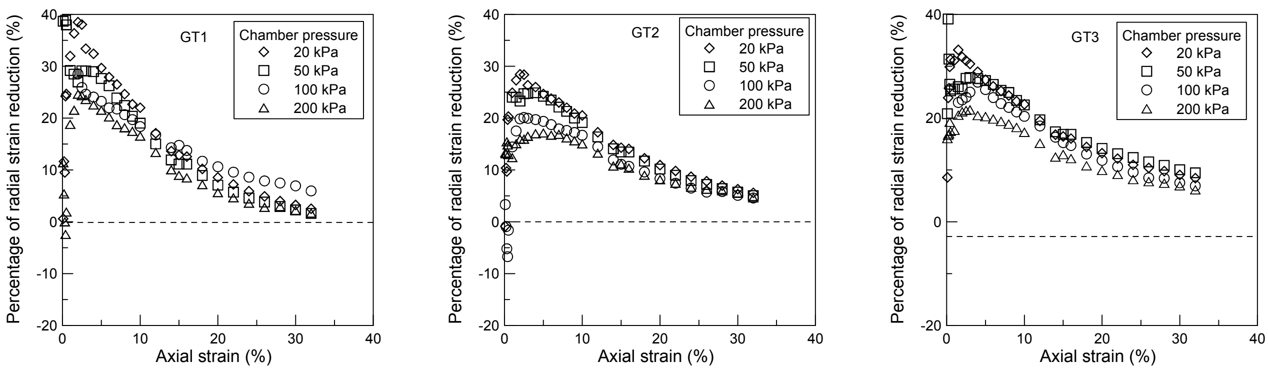
(a) GT1 (b) GT2 (c) GT3

Figure 5. Reduction in volumetric strain by reinforcing sleeves



(a) GT1 (b) GT2 (c) GT3

Figure 6. Radial strain of reinforced specimen



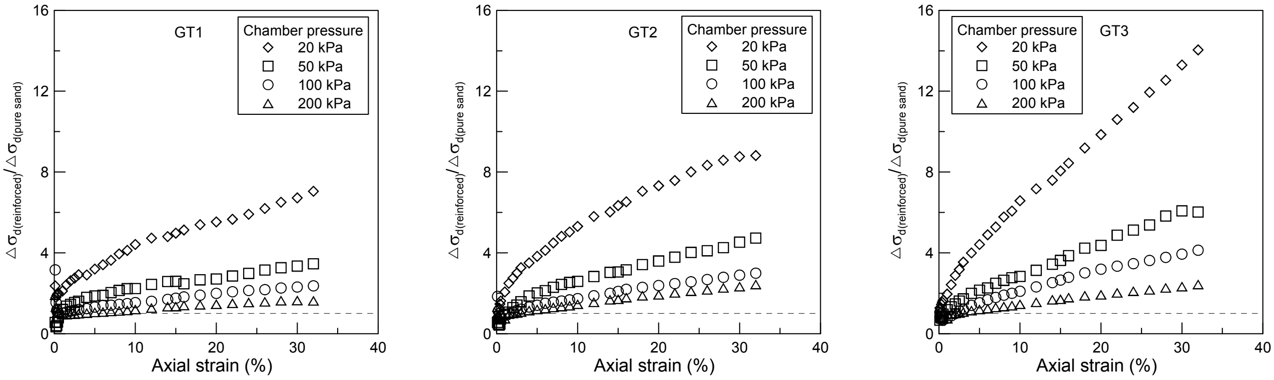
(a) GT1 (b) GT2 (c) GT3

Figure 7. Radial strain reduction percentage for reinforced specimens

between the reinforced and unreinforced specimens is introduced to evaluate the reinforcing effectiveness by the sleeve. Test results depicted in Figure 8 show marked increase in deviatoric stress. The highest deviatoric stress ratio value is 13.3 for the specimen subjected to 20 kPa chamber pressure while the specimen is reinforced by GT3 geotextile. Because volume expansion is not sensitive to chamber pressure, the magnitude of additional confining pressure varies insignificantly with chamber pressure. Accordingly, the deviatoric stress ratio decreases with the increase in chamber pressure. For specimens reinforced by GT3, the deviatoric stress ratio is 2.3 at 30% axial strain while the specimen is subjected to 200 kPa chamber pressure.

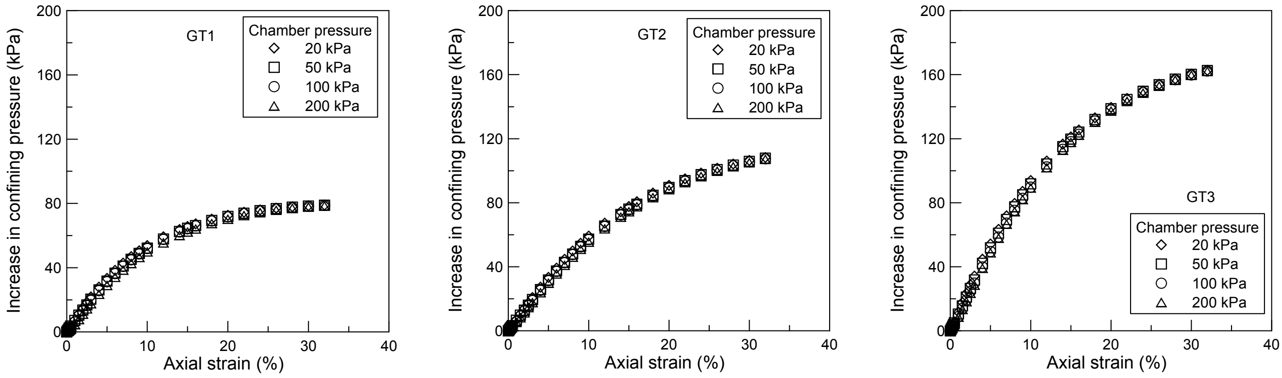
The increases in confining pressure counteracting specimen expansion are presented in Figure 9. The influence of chamber pressure on the increase in confining pressure is indistinct. This result can be attributed to the insensitivity of volume expansion to chamber pressure. A confining pressure ratio, ρ defined as the ratio of confining pressures between the reinforced and unreinforced specimens is introduced to delineate the effectiveness of the reinforcement. The relations between the confining pressure ratio and the axial strain are presented in Figure 10. The decrease in confining pressure ratio with the increase in chamber pressure occurs for the same reason as the variation in deviatoric stress ratio with respect to chamber pressure.

The mobilized pseudo cohesion and friction angle



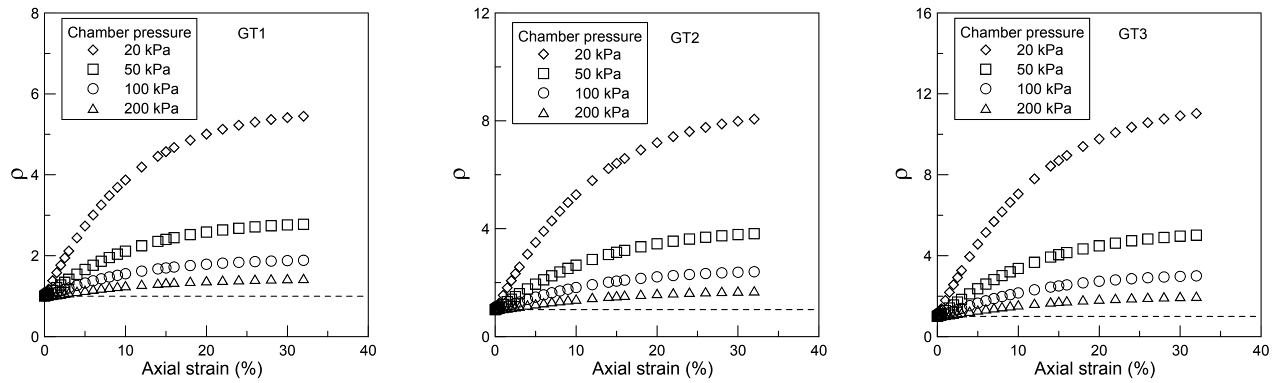
(a) GT1 (b) GT2 (c) GT3

Figure 8. Deviatoric stress ratios for reinforced specimens



(a) GT1 (b) GT2 (c) GT3

Figure 9. Confining pressure increase from encapsulating sleeves



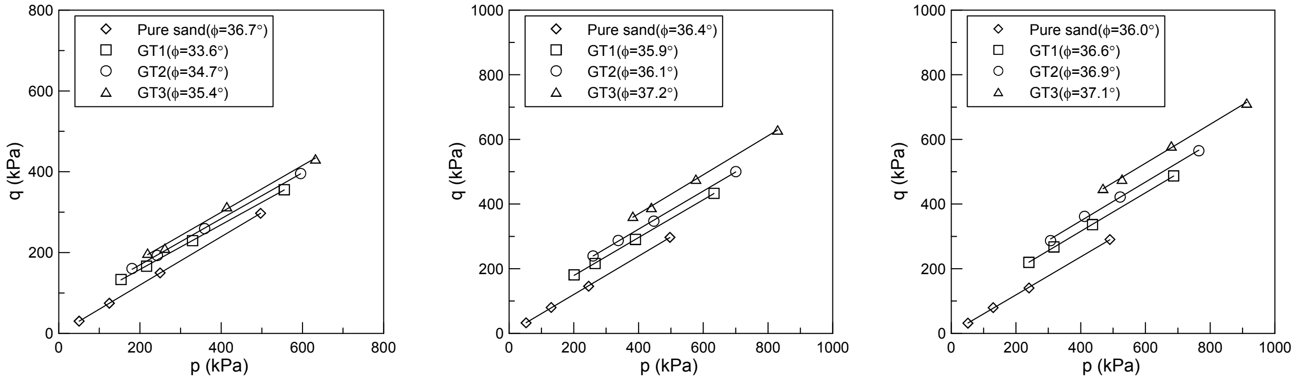
(a) GT1 (b) GT2 (c) GT3

Figure 10. Confining pressure ratios for reinforced specimens

The cohesion and friction angle of soil are evaluated using ultimate specimen strengths under various chamber pressures. However, the compressive strengths of the reinforced granular columns were not reached in these tests. Therefore, cohesion and frictional angle corresponding to different strains are evaluated and referred to as mobilized pseudo cohesion c_m and mobilized frictional angle ϕ_m . The p-q plots corresponding to different strain levels are presented in Figure 11. The intercept of the p-axis and the angle of the p-q line are used to calculate the mobilized pseudo cohesion and mobilized frictional angle of the reinforced soil. The mobilized pseudo cohesions and mobilized frictional angles for the specimens reinforced by three geotextiles are presented in Figure 12.

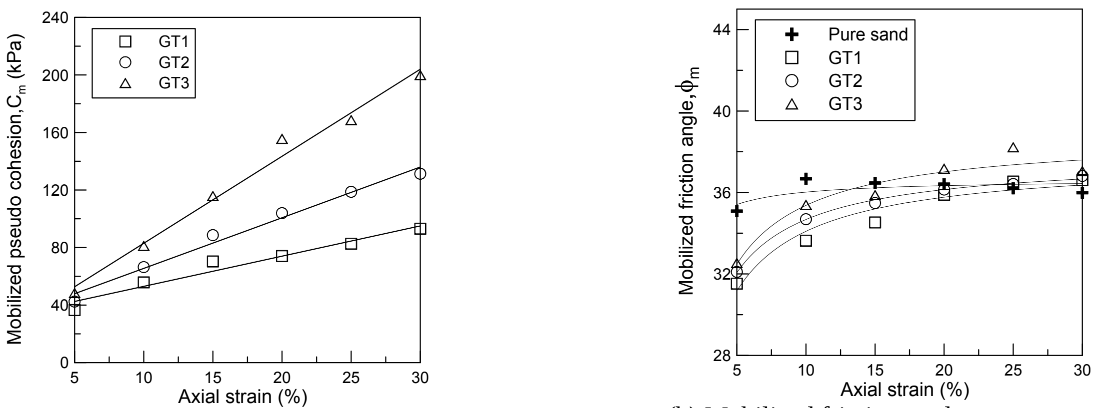
The results reveal that the mobilized friction angles for reinforced specimens increase gradually with the increase in axial strain whereas the angle for the pure sand reaches a constant angle. At low axial strains (axial strain less than 20%) the mobilized friction angles are lower than those from pure sand, but mobilized friction angles at greater axial strain are higher than the constant pure sand angle. At a specific axial strain the mobilized friction angles from specimens reinforced by different geotextiles do not exhibit significant differences. The maximum discrepancy in the mobilized friction angle of different geotextiles is 2.6 degrees.

For specimens reinforced using all three geotextiles, the mobilized pseudo cohesions increase linearly with the increase in axial strain. The stronger geotextile has more pronounced values.



(a) Axial strain 10% (b) Axial strain 20% (c) Axial strain 30%

Figure 11. The p-q plots at various axial strains for reinforced specimens



(a) Mobilized pseudo cohesion (b) Mobilized friction angle

Figure 12. Mobilized pseudo cohesion and friction angle of reinforced sand specimens

CONCLUSIONS

The responses for granular columns encapsulated in flexible tensile resistant sleeves were studied. The encapsulated column is subjected to a continuous incremental increase in lateral pressure compounded from the circumferential tensile stress developed in the extended encapsulating material and lateral restraint due to the counteraction of the surrounding in-situ soil upon column expansion. The results indicated:

1. The encapsulating sleeve increases the strength and impedes the volumetric strain of the sand specimen. The effects are more pronounced for a stronger geotextile. But the variation in volumetric suppression with different chamber pressures is trivial at greater axial strain. The increase in axial strain results in an increase in the deviatoric stress ratio. The reinforcing sleeve has pronounced effect on the deviatoric increase for the specimen subjected to low chamber pressure.
2. The geotextile has greater degree of lateral strain suppression for reinforced specimens subjected to lower chamber pressure. The radial strain reduction subsides at greater axial strain.
3. The sleeve provides a marked increase in deviatoric stress. But the magnitude of the additional confining pressure varies insignificantly with chamber pressure because volume expansion is not sensitive to chamber pressure.
4. The mobilized friction angles for reinforced specimens increase gradually with the increase in axial strain, whereas the angle for the pure sand reaches a constant angle. At low axial strains, the mobilized friction angles are lower

than those from pure sand, but mobilized friction angles at high axial strain are higher than the constant pure sand angle.

5. The mobilized pseudo cohesion increases linearly with the increase in axial strain. The stronger geotextile has a more pronounced effect.

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