Triaxial Testing on Geogrid-reinforced Granular Soils

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Abstract. Laboratory triaxial compression tests were carried out to investigate the mechanical behavior of dense sand and geogrid-reinforced granular soils. The tested sand having its mean particle size (D₅₀) equal to 0.6 mm was adopted. Three geogrids with different longitudinal and transverse nominal strengths were used. The dimensions of the cylindrical soil specimen were 70 mm (diameter) × 160 mm (height). The relative density was equal to 70% for all tests. The reinforced sand specimens with one or two geogrid layers were sheared under effective confining pressures (σ’₃) equal to 50 kPa. The test results of unreinforced sand indicate the general stress-strain behavior of dense sand when sheared, whereas the deviatoric stress reaches its peak value, after which it gradually decreases to ultimate value (σ₁ - σ₃) ult. The difference of effective confining pressure indicates that the peak of deviatoric stress Δσ₃ = (σ₁ - σ₃) increases with the increase in effective confining pressure (σ’₃), while the peak principal stress ratio (σ’₁/σ’₃) decreases with the increase (σ’₃). The friction angle (φ’) and cohesion (c’), defined by analytical and graphical methods for unreinforced sand. Geogrid as reinforcement increasing peak shear strength. The increasing peak shear strength is more pronounced with a higher number of geogrid and the geogrid with higher stiffness. Increased in confining stress inside reinforced soil mass (Δσ₃R) can be interpreted by cohesive reinforced soil (Cₐ).

1. Introduction

Geogrids have been spread in civil engineering projects such as slope stabilization, subgrade stabilization, bridge abutments, and drainage. This is due to geogrid high-strength material, simple construction techniques, and low cost. To optimized the designs of geogrid reinforced soils in civil engineering projects, laboratory testing such as triaxial test, direct shear test, and plane strain test were conducted. Many researchers have investigated the mechanism of reinforcing granular soils through a laboratory test using triaxial compression [1][2][3][4], direct shear test [5], plane strain test [6] [7]. The previous studies showed that geogrid as reinforcing materials improved the shear strength compare with unreinforced sand. The behavior of reinforced soils is affected by the type of granular material, effective confining pressure, the stiffness of reinforcement, and the arrangement layers. Until now, little attention has been observations to the shear strength parameter unreinforced sand and Increased confining stress inside reinforced soil mas. This research study focused to observe the shear strength parameter sand and behavior of unreinforced and reinforced granular soils with varying types of geogrid stiffness and the configuration of geogrid layers under the triaxial test.
2. Experimental program
Standard triaxial compression tests were adopted to investigate the behavior of unreinforced sand and reinforced sand. The tested sand having its mean particle size ($D_{50}$) equal to 0.6 mm was adopted. Three geogrids with different longitudinal and transverse nominal strengths were used. The relative density was equal to 70% for all tests. The reinforced sand specimens with one or two geogrid layers were sheared under effective confining pressures ($\sigma'_3$) equal to 50 kPa. The dimensions of the cylindrical soil specimen were 70 mm (diameter) × 160 mm (height). Table 1 shows the detailed experimental program.

| Group | Test Number | Reinforcement Layer | $\gamma_d$ (kN/m$^3$) | Dr (%) | Confining pressure, $\sigma'_3$ (kPa) |
|-------|-------------|---------------------|-----------------------|--------|-------------------------------------|
| UR    | UR01        | None                | 18.45                 | 70     | 50                                  |
|       | UR02        | None                | 18.45                 | 70     | 100                                 |
| R1    | R1L1        | 1                   | 18.45                 | 70     | 50                                  |
|       | R1L2        | 2                   | 18.45                 | 70     | 50                                  |
| R2    | R2L1        | 1                   | 18.45                 | 70     | 50                                  |
|       | R2L2        | 2                   | 18.45                 | 70     | 50                                  |
| R3    | R3L1        | 1                   | 18.45                 | 70     | 50                                  |
|       | R3L2        | 2                   | 18.45                 | 70     | 50                                  |

Note: 1 mm/min strain rate for all tests

2.1. Tested materials
A poorly graded sand (SP) with particle size between No.10 and No. 200 sieves was used in this study. The tested sand has a specific gravity (GS) 2.53, coefficient of uniformity (CU) 4.866 and gradation (CC) 0.94. The dry unit weights of sand were $\gamma_{d,max} = 19.39$ kN/m$^3$ and $\gamma_{d,min} = 16.82$ kN/m$^3$. Three geogrids with different longitudinal and transverse nominal strengths were used. The specification of three geogrids (R1, R2, R3) were shown in figure 1.

Figure 1. Tensile strength test of geogrid (standard specifications: ASTM D6637-A)
2.2. Test procedure
The dimensions of the cylindrical soil specimen were 70 mm (diameter) × 160 mm (height). A small tamper was prepared to compact dense sand into four layers. The relative density was equal to 70% for all tests, unit weight was 18.45 45 kN/m³. The geogrid was arranged in the specimen after compacted. Figure 2 shows the configuration of the reinforcement layer. The specimen sheared under effective confining pressures (σ’₃) equal to 50 kPa. The tests stop when the axial strain reached 15%.

![Figure 2](image_url)

**Figure 2.** Number of reinforcement layers for triaxial compression tests unreinforced sand; (b) 1 layer of geogrid; (c) 2 layer of geogrid.

3. The behavior of unreinforced sand

3.1. The stress and volumetric behavior of unreinforced sand
To investigate the behavior of unreinforced sand, a total of two samples were conducted. By testing the same specimens at the same relative densities (Dr = 70%) and with different confining pressures, the relationship between deviatoric stress Δσ₂ = (σ₁ − σ₃), effective principal stress ratio (σ’₁/σ’₃) and volumetric strain against axial strain (ε₁) could be determined. The test results indicate the general stress-strain behavior of dense sand when sheared, whereas the deviatoric stress reaches its peak value, after which it gradually decreases to ultimate value (σ₁ − σ₃)ult. The difference of effective confining pressure shows that the peak of deviatoric stress Δσ₂ = (σ₁ − σ₃) increases with the increase in effective confining pressure (σ’₃) as shown in figure 3(a), while the peak principal stress ratio (σ’₁/σ’₃) decreases with the increase (σ’₃) as shown in figure 3(b). The volumetric behavior for two samples of unreinforced sand in different (σ’₃) given in figure 4. The behavior of dense sand normally decreases (compression) in volumetric strain slightly at first, then the volumetric strain increases or dilates up when shearing progressed.

![Figure 3](image_url)

**Figure 3.** (a) Deviatoric stress; (b) Principal stress ratio.
3.2. Shear strength parameters of unreinforced sand

To analysis shear strength parameters such as effective friction angle (ϕ′) and cohesion (c′), the summary of unreinforced sand test results was shown in table 2.

Table 2. The summary of unreinforced sand tests results

| Group | Test | $\sigma_c$ (kPa) | $\sigma_u$ (kPa) | $\Delta\sigma_{q/d}$ (kPa) | $\Delta\sigma_f$ (kPa) | $\sigma_{1f}$ (kPa) | $\sigma'_{1f}$ (kPa) | $\sigma'_{1f}$ (kPa) | $p'_{fr}$ (kPa) | $q'_{fr}$ (kPa) |
|-------|------|-----------------|-----------------|--------------------------|----------------------|-------------------|-----------------|-------------------|----------------|----------------|
| UR    | UR01 | 250             | 200             | 210                       | 250                  | 460               | 50              | 260               | 155            | 294            |
|       | UR02 | 320             | 220             | 388                       | 320                  | 709               | 100             | 488               | 105            | 194            |

Friction angle (ϕ′) is calculated by using the analytical and graphical method (Kf line regression method). The $p'_{fr}$ and $q'_{fr}$ of unreinforced sand, tests are parameters to draw the Kf line regression, as shown in figure 5.

Figure 5. Kf line regression of unreinforced sand tests.

The stress paths of the consolidated drained (CD) test were presented in figure 6. No excess pore water pressure for the drained triaxial test, the drained total stresses path (T-u) SP is equal to the effective stresses path (ESP). The circle shown in Figure 6 represents a failure in terms of the p-q diagram. The
identical failure, as shown in figure 7 is the same failure on the Mohr $\tau$- $\sigma$ diagram. The equation of the $K_f$ line is

$$ q_f = p_f \tan \psi' + a $$

(1)

Where $a$ = the intercept on the $q$-axis, in the stress units, and $\psi'$ = the angle of the $K_f$ line concerning the horizontally, in degrees.

The equation Mohr-Coulomb failure envelope is

$$ \tau_f = \sigma_f \tan \phi' + c $$

(2)

From the Mohr-Coulomb, it can be shown that

$$ \sin \phi' = \tan \psi' $$

(3)

$$ \phi' = \sin^{-1} \tan \psi' $$

(4)

$$ \phi' = \sin^{-1} \tan \cdot 32.6^\circ = 40^\circ $$

(5)

And then,

$$ C'_{UR} = \frac{a}{\cos \phi'} $$

(6)

$$ C'_{UR} = \frac{5.53}{\cos \phi'} = 7.2 $$

(7)

From the $p$-$q$ diagram $\phi'$ and $c'$ can be calculated based on the obtained $a$ and $\psi'$. The theory for calculating shear strength parameters of soil in this research study was based on that developed by Head and Epps [8].

Figure 6. $K_f$ line of CD tests of unreinforced sand.
Figure 7. Mohr-Coulomb failure envelope of CD tests of unreinforced sand. Figure 8 shown illustration between the $K_f$ line and the Mohr-Coulomb failure envelope.

4. The behaviour of reinforced sand

4.1. Stress-strain and volumetric behaviour of reinforced sand

The behavior of geogrid-reinforced sand with a different type of stiffness and number of geogrid layers are shown in figure 9, compared with unreinforced specimens, all reinforced specimens increase in the peak of stress-axial strain behavior in terms of increases in peak deviatoric $\Delta \sigma_d = (\sigma_3 - \sigma_2)$ and principal stress ratio $(\sigma'_1/\sigma'_3)$. In addition, geogrid inclusion increases peak strength significantly. An increase in peak deviatoric stress and principal stress ratio is more pronounced as a higher number of geogrid and the nominal geogrid strength increases (higher stiffness). In the initial stage of shearing volumetric strain both unreinforced and reinforced dense specimen decrease (compression) slightly, then the volumetric strain increased (dilation) as shearing progressed.
4.2. The effect of stiffness and number of geogrid layer
The effect of stiffness and the number of geogrid layers are shown in figure 11. The curve shows that specimens reinforced with geogrid type 3 (R3) have a higher peak of deviatoric stress than specimens reinforced with geogrid type 2 (R2) and finally specimens reinforced with geogrid type 2 (R2) have a higher peak deviatoric stress than geogrid type 1 (R1). The results showed that the increasing peak shear strength is more pronounced with specimens that have a higher number of geogrid layers and the geogrid with higher stiffness.
4.3. Analysis of Pseudo-cohesion concept theory

Vidal [9] proposed the principle of reinforcing earth and the pseudo-cohesion concept for reinforced soil. The pseudo-cohesion concept theory was implemented to interpret the increased shear strength of geogrid-reinforced soil. The illustration apparent cohesion ($C_R$) using test R3L2 shown in Figure 12. The increased in confining stress inside reinforced soil mass ($\Delta \sigma_{3R}$) can be interpreted by cohesive reinforced soil ($C_R$). The mechanism of the development of cohesion is due to the increased confining stress generated in the soil due to the membrane stress in the wall of geogrids.

Figure 11. Effect of geogrid stiffness on deviatoric stress ratio-strain curves for reinforced sand configuration layers (a) single layer and (b) two layers.

Figure 12. Illustration of the apparent cohesion CD triaxial tests.
To analyze the cohesive reinforced were used the following equation

$$\sigma'_{1f} = \sigma'_{3f} \cdot \tan^2 \left( 45 + \frac{\Phi_{UR}}{2} \right) + 2C'_{\text{New}} \cdot K_p$$  \hspace{1cm} (8)

From equation 15 we get the $C'_{\text{New}}$ and we input the value the equation 16

$$C_R = C'_{\text{New}} - C'_{\text{UR}}$$  \hspace{1cm} (9)

The summary result of cohesive reinforced is given in Table 3.

**Table 3. The Summary of apparent cohesion ($C_R$).**

| Group | Test    | The number of geogrid layers | $\sigma'_{3f}$ (kPa) | $\sigma'_{1f}$ (kPa) | Kp | $C'_{\text{New}}$ | $C'_{\text{UR}}$ | $C_R$ |
|-------|---------|------------------------------|----------------------|----------------------|----|------------------|------------------|------|
| R1    | R1L1    | 1                            | 50                   | 384                  | 4.6 | 36               | 7.2             | 29   |
|       | R1L2    | 2                            | 50                   | 407                  | 4.6 | 41               | 7.2             | 34   |
| R2    | R2L1    | 1                            | 50                   | 410                  | 4.6 | 42               | 7.2             | 35   |
|       | R2L2    | 2                            | 50                   | 453                  | 4.6 | 52               | 7.2             | 45   |
| R3    | R3L1    | 1                            | 50                   | 430                  | 4.6 | 47               | 7.2             | 40   |
|       | R3L2    | 2                            | 50                   | 466                  | 4.6 | 55               | 7.2             | 48   |

4.4. Failure Pattern

The typical image before failure is shown in figure 13 a and the typical image after failure are shown in figure 13 b to 13 d. The investigation of the failed specimen unreinforced sand shows in figure 13 b. The results indicate that unreinforced specimens fail along at the angle $(45 + \Phi'/2)$ while the geogrid-reinforced specimen bulging occurred between the surface of the geogrid layer (figure 13 c to 13 d).

![Figure 13. The failed samples (a) unreinforced specimen before loaded; (b) unreinforced specimen after loaded; (c) reinforced 1 layer; (d) Reinforced 2 layer.](image)

5. Conclusions

The conclusion is drawn as follows:

1. The test results of unreinforced sand indicate the general stress-strain behavior of dense sand when sheared. The peak of deviatoric stress $\Delta \sigma_3 = (\sigma_1 - \sigma_3)$ increases with the increase in $(\sigma'_{3})$, while the peak of principal stress ratio $\frac{(\sigma'_{1})}{(\sigma'_{3})}$ decreases with the increase in $(\sigma'_{3})$.
2. The shear strength parameters sand in terms $\phi'$ and $c'$ can be defined by analytical and graphical methods.
3. Geogrid as reinforcement increasing peak shear. The increasing peak shear strength is more pronounced with a higher number of geogrid and the geogrid with higher stiffness.
4. In the initial stage of shearing volumetric strain both unreinforced and reinforced dense specimen decrease (compression) slightly, then the volumetric strain increased (dilation) as shearing progressed.
5. The increased in confining stress inside reinforced soil mass (Δσ_{3R}) can be interpreted by cohesive reinforced soil (C_R).
6. The investigation of the failed specimen indicates that the unreinforced specimen failed along with a planner at the angle \left(45 + \frac{\phi'}{2}\right), while the geogrid-reinforced specimen bulging between the surface of the geogrid layer.

6. References

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