INFLUENCE OF GEOSYNTHETIC REINFORCEMENT ON THE SHEAR STRENGTH CHARACTERISTICS OF TWO-LAYER SUB-GRADE

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Abstract

Due to the low bearing capacity of soft clayey soils in places that because of economic, military or geological conditions we are oblige to build a structure on, geosynthetics will be used to reinforce the soil and improve its bearing capacity. Particularly, A good example is roadways, where geosynthetics are placed between the interface of the granular materials and the soft-soil sub-grade to improve the bearing capacity of the composite layers. In previous research the behavior of one-layer soils that were reinforced with different kinds of geosynthetics were studied by experimental and analytical methods and some numerical models have been developed. In this paper the behavior of two-layer soils (granular base and clayey sub-grade) that were reinforced with some geosynthetics are investigated. Large-scale direct shear tests were performed on unreinforced and reinforced samples with different geosynthetics. The results show that depending on the characteristics of the geosynthetics, the inclusion of these materials may increase or decrease the shear strength parameters of the interface of two-layered soils. It implies that the geosynthetic-reinforced soils in the sub-base layer of roads are so sensitive to the characteristics of geosynthetics and will perform better than non-reinforced soils and consequently the load-carrying capacity of the basement will improve only if the appropriate geosynthetics are used. However, geogrid shows more reinforcement efficiency under higher vertical stresses. Increasing the relative density of the clayey sub-grade would also cause the geogrid reinforcement to be more effective.

1 INTRODUCTION

In places which, because of the economic, military or geological conditions, building a structure on such soils is essential, geosynthetics will be used to reinforce the soil and improve the bearing capacity. Different investigations have been performed to study the interaction of soils/geosynthetics in recent years. Abu-Farsakh et al. (2007) studied the behavior of a large modeled foundation, placed on reinforced soil. Feng et al. (2008) calculated the ultimate strength of red clay, reinforced by geogrid, using a pull-out test. Williams and Okine (2007) used a CBR test to study the effects of some kinds of geogrids on granular base soil. However, an understanding of the soil/geosynthetic interface's shear strength is essential to the design and stability analysis of geosynthetically reinforced soil structures. For example, an interface with a stronger shearing resistance in a geosynthetically lined slope can reduce the tensile forces mobilized in the geosynthetics, as well as increase the slope inclination (Liu and Gilbert, 2003; Palmeira and Viana, 2003). The shear strength of the soil/geosynthetic interface is also essential for a numerical simulation of the behavior of the sub-grade and base layers of roads. Though the shear strength of the soil/geosynthetic interface has been investigated by conducting other tests, such as tilt-table tests (Wu et al., 2008), the direct shear test is still the most common testing method. For example, direct shear tests involving the interfaces between soil and a geotextile have been performed by Richards and Scott (1985), Lee and Manjunath (2000), Mahmood et al. (2000), and Bergado et al. (2006). Studies involving the interfaces between soil and geogrid include Jarret and Bathurst (1985), Cancelli et al. (1992), Bauer and Zhao (1993), Cazzuffi et al. (1993), Bakeer et al. (1998), Abu-Farsakh and Coronel (2006) and Mostafa A. El Sawwaf (2006).

However, the scale effect of the shear box has always been an important parameter in these investigations. Ingold (1982) conducts laboratory direct-shear tests of the soil/geotextile interface by using different sizes of shear boxes. He concluded that the friction angle obtained from a 60 mm × 60 mm shear area was 2–3° higher than that obtained from a 300 mm × 300 mm shear area. Liu et al. (2008) investigated the effect of a special kind of geogrid on some kinds of soils by using a large-scale direct-shear test. Abu-Farsakh et al. (2007) used a large-scale direct-shear test to study the effect of moisture and dry density on the interaction between the soil and the geosynthetics. Cindric et al. (2006) investigated the effects of a geogrid on the elastic and shear moduli of soils. O'Kelly and Naughton (2007) performed some large-scale direct-shear tests to investigate the behavior of a new kind of geogrid, which has the capability of drainage and presented some graphs. Liu et al. (2008) studied the behavior of soil/PET-yarn geogrid interfaces by using large-scale direct-shear tests. Jesmani et al. (2010) conducted some undrained direct-shear tests to study the effect of plasticity index and normal stress on the shear behavior and the shear modulus of remolded clays.

Almost all previous investigations have studied the behavior of geosynthetics in a one-layered soil. Although several investigations were performed in order to find the best depth for embedding the geosynthetics, there have been very limited investigations on the interactions of two-layered soils and geosynthetics. Zhou and Wen (2006) studied a model of sand soil, placed on soft clay that was reinforced by a geogrid, using a compression test. Kazimierowicz-Frankowska (2007) studied the effect of reinforcement on the load-settlement characteristics of a two-layer subgrade. Sireesh et al. (2008) investigated the bearing capacity of circular footing on a geocell–sand mattress overlying a clay bed. Anubhav and Basudhar (2010) conducted experiments in a direct-shear test apparatus to study the shear force-displacement behavior at the soil-geotextile interface using two differently textured woven geotextiles. A non-linear constitutive model was presented for predicting both the pre-peak and the post-peak interface behavior. The predictions made with the developed model were found to be in good agreement with the experimental data obtained from direct shear tests. Palmeira (2009) discussed some experimental, theoretical and numerical methods for the study and evaluation of the interaction between soils and geosynthetics, with particular reference to the applications of these materials in soil reinforcement. The main advantages and limitations of some traditional experimental and theoretical methods for the study of soil–geosynthetic interactions were presented

and new applications of these methods were addressed; and the need for improvements in the experimental and theoretical techniques for a better understanding of soil–geosynthetic interactions was highlighted.

In this paper, due to the importance of the shear-strength characteristics of reinforced soils, the effects of reinforcing a two-layer soil with geosynthetics on the shear-strength parameters of the interface of two soils were investigated by conducting a series of large-scale direct-shear tests.

2 TEST APPARATUS

The size of the shearing device can influence the direct-shear test results. Generally, the boundary effect and device friction are more significant for a smaller shear box. Ingold (1982) conducted laboratory direct-shear tests of the soil/geotextile interface by using different sizes of shear boxes. He concluded that the friction angle obtained from a 60 mm × 60 mm shear area was 2–3° higher than that obtained from a 300 mm × 300 mm shear area. The dimension of the shear box, as regulated by ASTM D5321 (ASTM 2002), with minimum dimensions of five times the maximum opening size (in plan) of the geosynthetic being tested , should be used in the direct shear test of the geosynthetic/soil interface. It also mentioned that the depth of each container that contains the soil must be a minimum of 50 mm or six times the maximum particle size of the coarser soil being tested, whichever is greater. In this study a large-scale direct-shear device was used, which has length, width, and thickness of 150mm × 150mm × 100mm. The movement of the upper shear box in the horizontal direction is controlled by a set of gears that are mobilized by an electric motor. The vertical loading applied by a hydraulic jack is transferred through the rigid reaction frame and adds on a rigid load plate that is placed on top of the soils in the upper shear box. The normal load is constant during the test, satisfying the requirement regulated by ASTM D5321 (ASTM 2002).

A rigid plate is conventionally used as the loading plate in direct-shear tests (for example, Bakeer et al., 1998; Lee and Manjunath, 2000). The system is capable of applying a vertical force and a shear force of up to 50 kN. Fig. 1 shows a frontal view of the large-scale direct-shear device used in this study. The vertical force applied on the rigid plate and its vertical displacement are measured during the tests. The horizontal movement of the upper shear box and the shear force exerted during the shearing testing are also recorded. These data are collected by using two load cells and two linear variable displacement

Figure 1. Large-scale direct-shear device (Imam Khomeini International University).

transformers (LVDT). The capacity for both load cells is 50kN. The capacities for the vertical and horizontal LVDT are 50 mm and 100 mm, respectively.

3 TEST MATERIAL

The tests herein use two soils, including a clay soil and a granular soil that has been grained in a manner that satisfies the suggestion of AASHTTO M147 for the sub-base soil of roads. In order to obtain comparable results with practical projects, the clay soil was chosen from Varamin region, Iran, where a railroad project was under construction. Table 1 lists the physical characteristics of each soil, while Fig. 2 shows their grain-size distribution curves. It should be noted that in this figure the upper and lower limit means the maximum and minimum grain size that the AASHTO M147 suggested. The granular soil is classified as SW according to the Unified Soil Classification System (USCS). The dry unit weight for the granular soil sample was 21.33 kN/m³. The clay soil is classified as CL according to Unified Soil Classification System (USCS)

Table 1. Soil Characteristics.

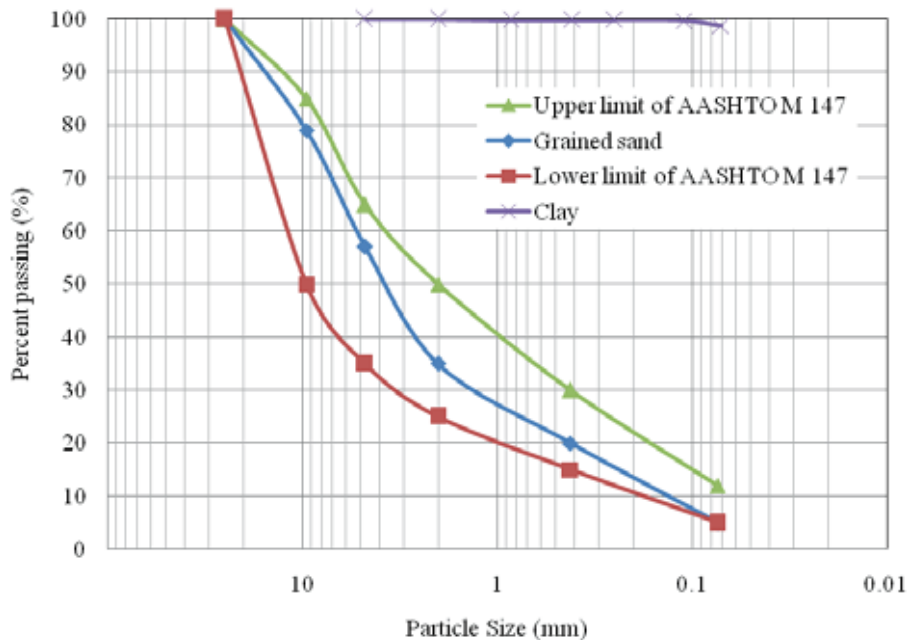| Property	D_{10} (mm)	D_{30} (mm)	D_{60} (mm)	Coefficient of uniformity	Coefficient of curvature	Liquid limit	Plastic limit	USCS symbol
Sand	0.12	1.20	5.15	42.92	2.33	-	-	SW
Clay	-	-	-	-	-	32	19	CL

Figure 2. Grain-size distribution curves of soils.

Table 2. Geosynthetic characteristics.

Geosynthetic	Material	Aperture size (mm)	Weight (g/m²)	Tensile strength-MD (kN/m)	Tensile strength-CD (kN/m)
GG	Polyethylene (HDPE)	10 × 10	700	7.6	7.6
GT-NW	Polyester	0.062	300	8	11.1
GT-W	Yarns polyester	-	2286	100	10

and its dry unit weight was 19.20 kN/m³. Both soils were prepared in a relative density (R) of 90% of the maximum density with the optimum water content (16.0% for clay and 9.0% for sand), which has been calculated according to B-method of AASHTO T180. The maximum sand sizes were smaller than 9.5 mm. This satisfies the general requirement that the ratio of the minimum size of the shear box to the maximum size of the soil particle is greater than 6. Research on the interface of geosynthetic and cohesive materials is becoming popular (for example, Fourie and Fabian, 1987; Abu-Farsakh and Coronel, 2006). The rationale behind the application of a cohesive material is that of an economic concern.

This study uses two geotextile and one geogrid, and they are denoted as GT-NW (Non-Woven Geotextiles), GT-W (Woven Geotextiles) and GG (Geogrids), respectively (Fig. 3). The woven geotextile has a tensile strength in one direction, but the non-woven one has in it two directions. Table 2 lists the physical characteristics of these geosynthetics.

4 TEST PROCEDURES

The soil used for the large-scale direct-shear testing program is dried in an oven and after wetting to the optimum water content, compacted to the target unit

weight within the shear box. Each soil is compacted in three layers. The compaction of the sand and clay is conducted by using a manual plastic hammer to hit the steel plate, which was placed on top of the soil until reaching the target unit weight. The geosynthetic is positioned on top of the lower shear box and at the interface of the sand and clay soils. These tests are conducted using normal stresses of 44, 96, and 192 kPa, which represent the normal stresses that different vehicles may apply to a sub-grade layer of roads. The normal loading is applied on the specimen and the vertical deformation of the test specimen is then monitored. Shear loading is not applied until the vertical deformation reaches its equilibrium. According to ASTM D5321, a shear rate of 1 mm/min is used in this test program in order to satisfy the undrained failure condition. However, it should be noted that undrained conditions cannot be guaranteed in direct shear tests. But for the purpose of this research, the suggestion of the ASTM standard would be precise enough.

The test stops when the shear displacement reaches about 30 mm, i.e., about 20% of the shear strain. The maximum shear strength during the shear process is recorded as the peak shear strength. The direct-shear tests for the soil-soil interfaces and the soil-geosynthetic were conducted under the same normal loading and same testing procedures for the sake of the comparison.

Figure 3. Geosynthetic specimens a) Geogrid, b) Non-Woven Geotextile, c) Woven Geotextile.
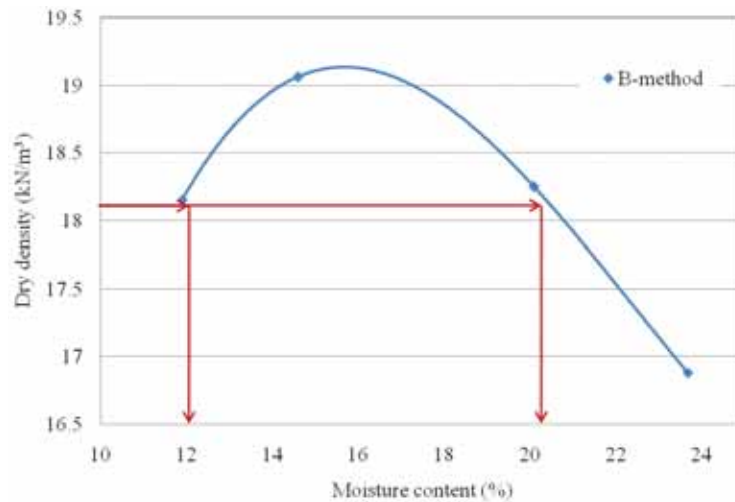
Figure 4. Choosing different water contents of the clay soil.

Tests with various clay relative densities (75%, 85% and 90%) and different clay moisture contents (12%, 16% and 20%) were also performed to investigate the effects of the sub-grade relative density and water content. To make the test results comparable, the moisture contents were chosen based on the compaction curve of the clay soil to obtain the same density in both 12% and 20% water contents (Fig. 4).

Richards and Scott (1985) concluded that the test using a large solid block as the lower shear box is the best at replicating the testing results of the soil/geotextile interface. Jewell (1996) stated that the geotextile and geomembrane can be tested with a solid block or soil in the lower part of the shear box, while the geogrid must be tested by a device in which both parts of the shear test device have to be filled with soil. The set-up of the direct shearing device is not strictly regulated by testing standards. For example, only the minimum size of the shearing box is stated explicitly in ASTM D5321 for the direct shear test of the soil-geosynthetic interface. Few

research studies focus on the appropriate setup of direct-shear testing device for the soil/geogrid interface. For example, the difference in the measured shear strength when using different sizes of a lower shear box has not been discussed. Liu et al. (2009) tested three different set-ups of a lower shearing box. The effects of different set-ups on the test results were evaluated by comparing the direct-shear test results using different set-ups of the lower shear box. The set-ups included: a box with the same size that was filled with soil, a box with a larger size filled with soil, and a larger lower shear box filled with a solid block (Liu et al., 2009). It was observed that among these set-ups, a box with the same size that was filled with soil produces the greatest interface shear strength. For conducting the direct shear test against the geosynthetic interface in the soil, the sizes of the upper and lower shear boxes should be same and they must be filled with the necessary soil. Therefore, in this study, all of the tests were performed in manner such that the sizes of the upper and lower shear boxes were the same and they were filled with the necessary soils (Fig. 5).
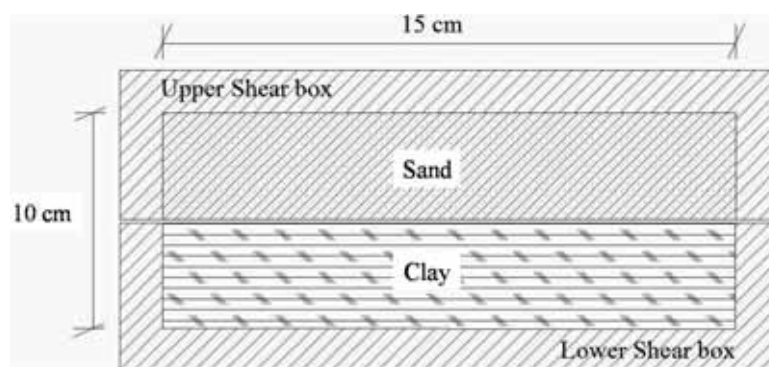

Figure 5. Set-up of upper and lower shear box.

5 RESULTS AND DISCUSSIONS

The results of the large direct-shear tests are presented in this section in terms of the shear behavior for different soil/geosynthetic interfaces.

5.1 SHEAR BEHAVIOR OF THE SOIL/ GEOSYNTHETIC INTERFACE AT OPTIMUM WATER CONTENT AND R=90%

To discuss the stress–strain behavior of the soil/geosynthetic interface, the results of the direct-shear tests for samples with a relative density of 90% and optimum water content are presented in Fig. 6. The results of the vertical deformation versus the shear displacement for these samples are shown in Fig. 7.

The results show that there is no well-defined peak shear strength observed for the soil/geosynthetic interface in higher normal stresses. In general, a yield shear stress is reached at a small shear displacement (usually less than 15 mm). The shear stress of the soil/geosynthetic interface became constant after a displacement of about 15 mm. It is clear that the geogrid causes the soils to show more shear strength than the two other geotextiles. However, these differences become smaller when increasing the normal stress. Calculating the values of the cohesion (c) and the internal friction angle (φ) for the geogrid-reinforced soil reveals that c would decrease by 22% and φ would increase by 13% relative to the non-reinforced two-layer soil. Similar calculations for woven and non-woven geotextiles show 31% and 81% decreases for the cohesion and a 5% decrease and a 3% increase for the internal friction angle, respectively.

As is clear from Fig. 7, during the initial stage, when the shear displacement is small, the geosynthetics-reinforced soil undergoes a vertical contraction. Following the contraction, the vertical deformation behavior then depends on the geosynthetic type. The contraction would become constant for soils reinforced by non-woven geotextile and would decrease for other reinforced soils, especially for geogrid-reinforced soils.

5.2 PEAK SHEAR STRENGTH OF SOILS AND SOIL/GEOSYNTHETIC INTERFACES AT THE OPTIMUM WATER CONTENT AND R=90%

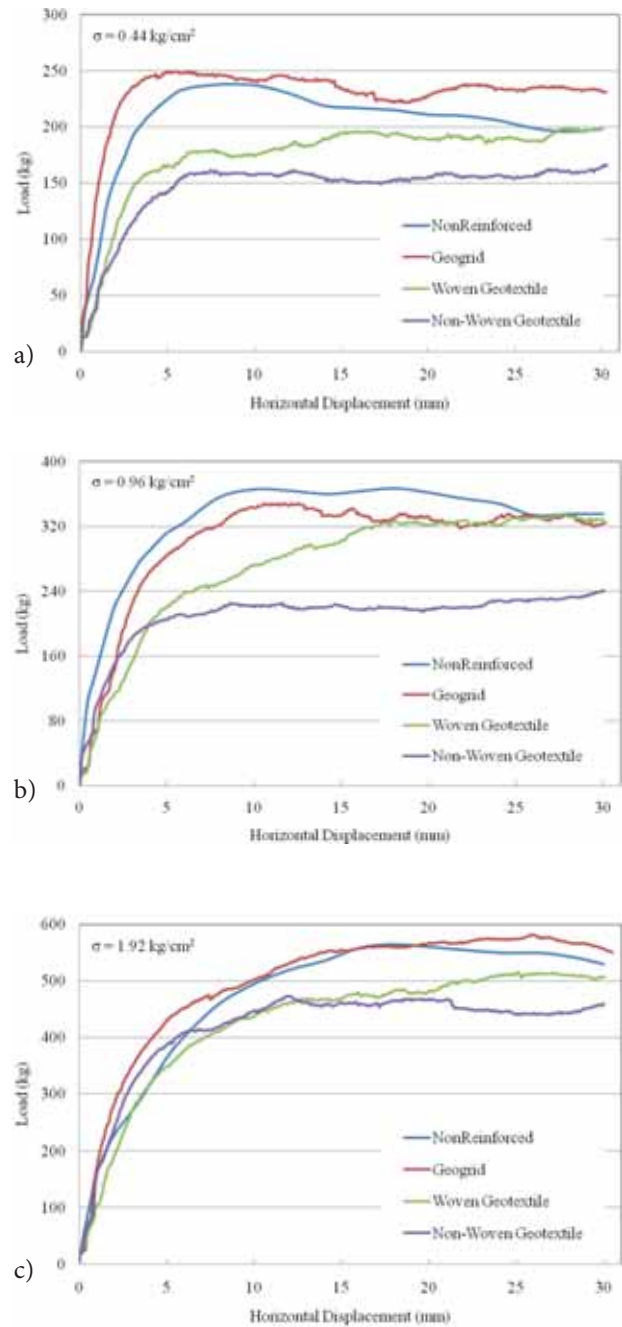Fig. 8 shows the peak shear strength for soils and the soil/geosynthetic interface. The results show an apparent

Figure 6. Stress–strain behavior of the soil/geosynthetic interface under different normal loading (R=90%, ω=16%), a) σ=44 kPa, b) σ=96 kPa, c) σ=192 kPa.

cohesion in these peak shear strengths. Possible explanations are in the non-linear relationship of the shear stress and in the machine friction under this stress level. The interface shear strength of the soil against the geotextile is significantly lower than the internal shear strength of the corresponding soils.
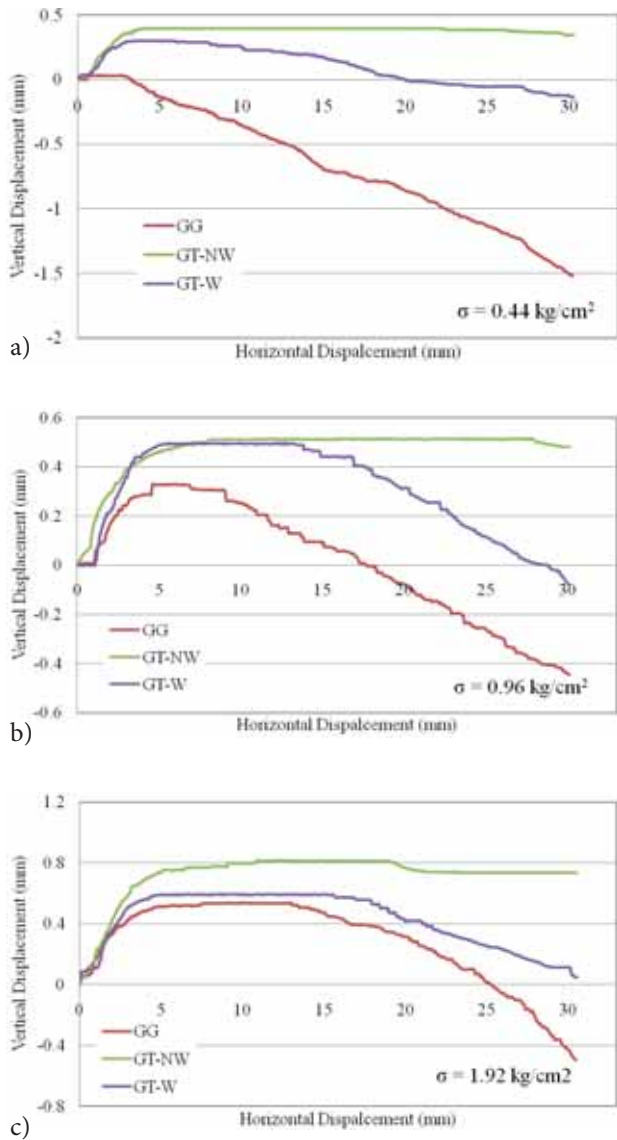
Figure 7. Vertical deformation versus horizontal displacement under different normal loading ($R=90\%$, $\omega=16\%$), a) $\sigma=44$ kPa, b) $\sigma=96$ kPa, c) $\sigma=192$ kPa.
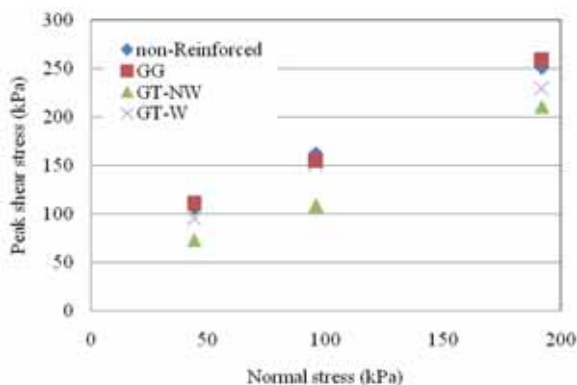

Figure 8. Peak shear strength versus normal stress for the soil/geosynthetic interface ($R=90\%$).

It is observed that under different normal stresses, the shear strength of the soil-geogrid interface is always higher than that of soil–geotextile interface. Alfaro et al. (1995) and Tatlisoz et al. (1998) stated that the direct-shear resistance of the soil/geogrid interface is composed of the soil-to-geosynthetic shear resistance and the soil-to-soil shear resistance within the geogrid openings. As presented in Fig. 8, the internal geogrid reinforced soil shear strength is higher than the interface shear strength of the soil against the geotextile, and therefore this increase in the shear strength is mainly attributed to the interlocking of soil particles that penetrate through the geogrid apertures. This conclusion is in agreement with the presented results of Liu et al. (2009).

5.3 EFFECT OF DENSITY ON THE SHEAR STRENGTH OF SOIL/GEOSYNTHETIC INTERFACES

To investigate the effect of sub-grade soil density, samples with different clay densities (75%, 85% and 90%) were prepared. However, in order to avoid clay settlement and consequently changing the shear plane due to placing a sand layer in upper box, in samples with a clay relative density other that 90%, clay soil was placed in upper box of the shear apparatus and sand with a relative density equal to 90% placed in the lower box. Fig. 9(a) shows the failure envelopes of non-reinforced and reinforced samples with $R=75\%$ for the clay layer, as an example. It is clear that the geogrid is the only geosynthetic that increases the shear strength of the samples. Similar envelopes in $R=90\%$ are presented in Fig. 9(b). Comparing these two figures reveals that the reinforcing effect of the geogrid is more significant in a lower relative density of the sub-grade soil. However, specimens with a higher relative density of clay soil show a higher shear strength.

5.4 EFFECT OF WATER CONTENT ON SHEAR STRENGTH OF SOIL/GEOSYNTHETIC INTERFACES

Samples with different water contents (12% and 20%) for the clay layer, in addition to the optimum water content (16%), were prepared and tested in order to study the effect of the moisture content of the sub-grade soil on the shear strength of reinforced roads at the interface of the sub-grade and base layers. Values of the water content chosen in a manner to let us investigate just the effect of the water content with all the other parameters, including the dry density (γ), kept constant (Fig. 4). The behavior of the non-reinforced and the reinforced samples at different water contents and a relative density of $R=90\%$ are shown in Fig. 10.
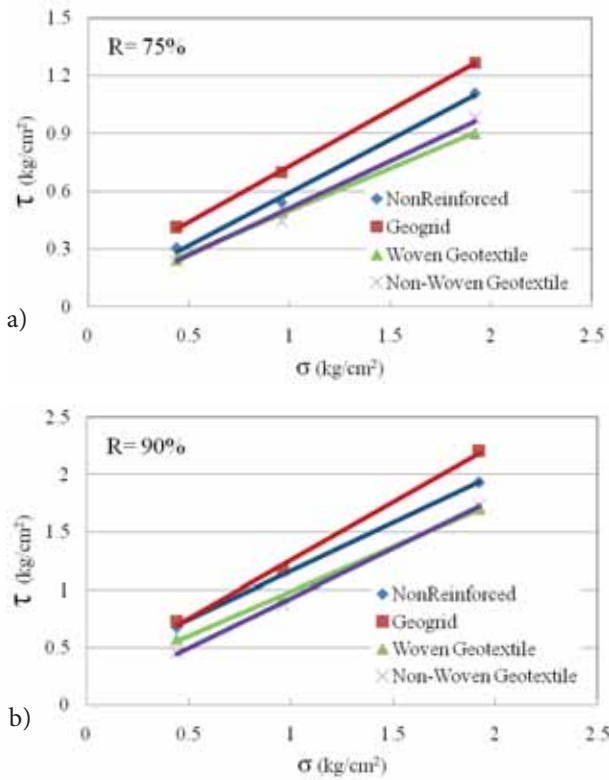
Figure 9. Failure envelopes of non-reinforced and reinforced soil samples with the optimum water content, a) R=75%, b) R=90%.

Figure 10. Failure envelopes of non-reinforced and reinforced soil samples with different water contents and R=90%, a) ω=12%, b) ω=20%.

It is clear that the samples with ω=20% for the clay layer show a higher shear strength at the interface of two layers; and the samples reinforced by the geogrid reveal a higher shear strength in comparison to the geotextiles, which could be up to a 9% increase in the internal friction angle relative to the non-reinforced two-layer soil. However, the reinforcing effect of the geosynthetics is negligible and has almost the same procedure in both water contents.

Figs. 9 and 10 reveal that the geogrid is the best geosynthetic that can be used to improve the shear-strength parameters of the two-layer soils. Both geotextiles decrease the shear-strength characteristics of the interface plane of two soils, which probably is because of the lack of apertures in the geotextile and consequently the lack of interlocking between the two soils.

5.5 EFFICIENCY OF REINFORCEMENT

The efficiency of all the geosynthetics for reinforcing the two-layer soils was evaluated using equation (1).

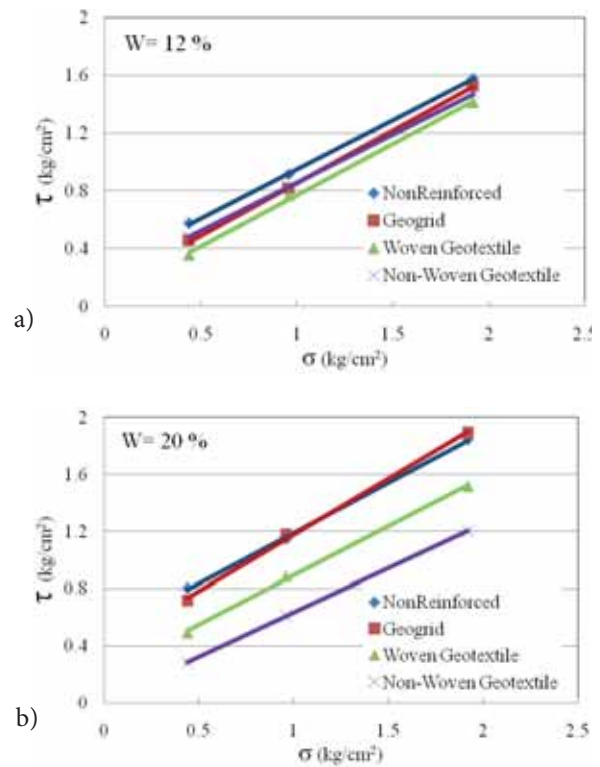$$\eta = \tau_{reinforced} / \tau_{unreinforced} \quad (1)$$

Calculating the efficiency of the reinforcement (η) for all the geosynthetics in various water contents and different relative densities shows that in samples with a relative density equal to 90%, with respect to the amount of water content, by increasing the normal stress, the efficiency of the reinforcement would increase (Fig. 11). However, at R=75%, increasing the normal stress would cause a decrease in the efficiency of the reinforcement.

It is clear that the geogrid has the higher efficiency in all conditions.

5.6 COEFFICIENT OF INTERACTION

The frictional resistance obtained from the direct-shear test on soil-geogrid specimens is a combination of the soil-to-reinforcement interface friction and the soil-to-soil shear resistance at the geogrid openings (Abu-Farsakh et al., 2007). Bergado et al. (1993) suggested the following equation to calculate the frictional resistance force at the soil-geogrid interface:

$$P_s = A[\alpha_{ds} c_a + (1- \alpha_{ds}) c] + A\sigma_n[\alpha_{ds} \tan \delta + (1- \alpha_{ds}) \tan \varphi] \quad (2)$$
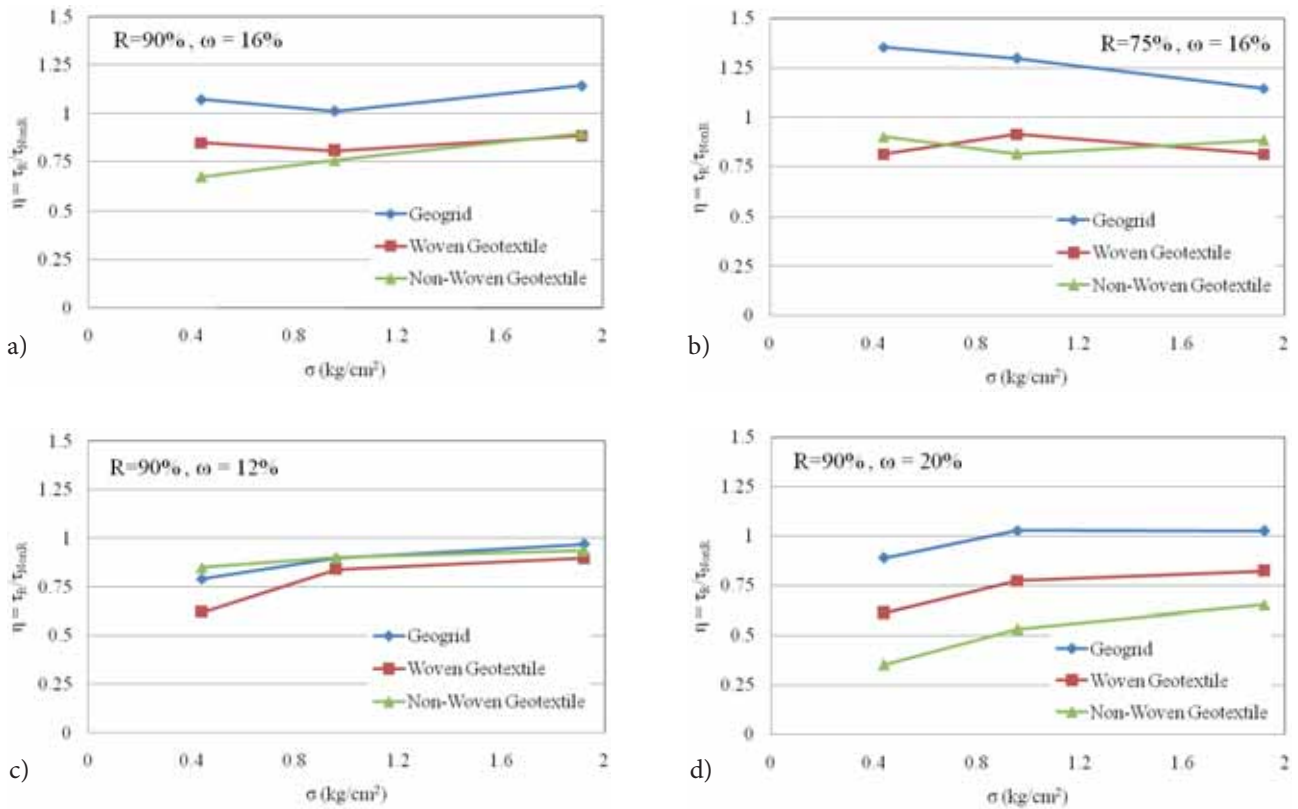
Figure 11. Efficiency of geosynthetic reinforcement in various water contents and different relative densities, a) R=90%, ω=16%, b) R=75%, ω=16%, c) R=90%, ω=12%, d) R=90%, ω=20%.

Where

P_s = direct shear resistance;
A = total shear area of the samples;
α_{ds} = ratio of the reinforcement shear area to the total shear area;
c_a = soil-geogrid adhesion;
c = soil-soil cohesion;
σ_n = normal stress at the shear plane;
δ = soil-geosynthetic interface friction angle;
φ = frictional angle of soil-to-soil from direct shear test.

Eq. (2) can be divided into two parts: the soil-to-geogrid frictional shear force, and the soil-to-soil direct frictional shear force. The soil-to-geogrid direct shear force can be formulated using the Mohr–Coulomb criterion as

$$F_{s-g} = (c_a + \sigma_n \tan \delta) A_{s-g} \qquad (3)$$

Where

A_{s-g} = area between the soil and the geogrid. Similarly, the soil-to-soil direct shear force is given as

$$F_{s-s} = (c + \sigma_n \tan \varphi) A_{s-s} \qquad (4)$$

Where

A_{s-s} = area of the soil-to-soil friction at the geogrid openings.

Eqs. (2)-(4) yield the following equation:

$$\tau_{s-g} = (P_s - A(1- \alpha_{ds})(c + \sigma_n \tan \varphi)) / (A \, \alpha_{ds}) =$$
$$= c_a + \sigma_n \tan \delta \qquad 5)$$

where

τ_{s-g} = shear stress along the soil-geosynthetic interface.

By plotting τ_{s-g} versus σ_n, c_a and δ were calculated (c_a=0.44 kg/cm² and δ=43º).

Many researchers have discussed the importance of using the "interface efficiency" or "Coefficient of interaction," c_i (Abu-Farsakh et al., 2007). As a primary design parameter in the geosynthetically reinforced soil structures (Cowell and Sprague, 1993; Koutsourais et al., 1998; Tatlisoz et al., 1998), the interface efficiency is used to calculate the effective length of the reinforce-

ment required beyond the critical failure plane for MSE walls and reinforced slopes (Abu-Farsakh et al., 2007). The interface efficiency for cohesive soils is defined as the ratio of the shear strength at the soil-reinforcement interface to the shear strength of the soil at the same overburden condition (Cowell and Sprague, 1993).

$$c_i = (c_a + \sigma_n \tan \delta_a) / (c + \sigma_n \tan \varphi) \qquad (6)$$

Calculating c_i for a two-layer soil reveals that the average Coefficient of interaction is equal to $c_i =1.19$, which shows a considerable adhesion between the soil and the geogrid. However, the interface efficiency depends on the normal (confining) stress (i.e., the depth of the reinforcement layer) in addition to the material characteristics (Abu-Farsakh et al., 2007). Based on the value of ci, the most appropriate geosynthetic could be chosen in a specific project with a specific soil and specific circumstances.

6 CONCLUSIONS

In this study we conducted a series of large-scale direct-shear tests on a two-layer soil that was reinforced by three kinds of geosynthetics. The geosynthetics were placed at the interface of the two soils, which was located between the upper and lower boxes of the direct shear device. The following specific conclusions can be drawn from the study:

1. The interface shear strength of the soil/geotextile is lower than the soil shear strength. It indicates that the geotextile placed within the soil usually acts as a weak interface in terms of direct sliding.
2. The direct-shear resistance of the soil/geogrid interface is higher than the soil/ geotextile; and it usually increases the value of the shear resistance of the two-layer soil. This increment is due to the creation of the shear resistance between the soil and surface and the opening of the geogrid, and the bearing resistance provided by transverse ribs. These resistances would be created when a relative movement occurs in the soil and geogrid interface.
3. It is clear that the geogrid increases both the internal friction and the cohesion of the soil (it increases the cohesion by about 23% and the internal friction by about 3 degrees), but geotextiles just increase the cohesion of the two-layer soil (about 9%) and the internal friction would decreases by about 1 to 3 degrees.
4. The improvements of the shear strength for the geogrid--reinforced specimens are more in the higher normal stresses. In other words, the geogrid shows more reinforcement efficiency at higher vertical stresses.

5. Increasing the relative density of the clayey sub-grade would cause the geogrid reinforcement to be more effective. Also, the improvement of the shear strength by increasing the normal stress would be more at a denser sub-grade.
6. The internal friction angle (φ) and cohesion (c) of two-layer reinforced soil at a relative density of 90% could be up to 34% and 40% higher than samples at a relative density of 75%, respectively.
7. Increasing the water content of the clay soil in the geogrid-reinforced specimens increases the shear strength of the two-layer soil. This could be because of the increment of cohesion, while the internal friction would be produced by the geogrid.
8. Increasing the water content of the clay-geogrid--reinforced specimens from 12% to 20% could increase the internal friction angle and the cohesion by up to 5% and 200%, respectively.

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