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Laboratory evaluation of the behavior of a geotextile reinforced clay

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ABSTRACT

To evaluate the behavior of cohesive soil reinforced with a geotextile, 144 unconfined and 72 unconsolidated–undrained (UU) triaxial compression tests were conducted. The moisture content of soil during remolding, relative compaction, soil type, confining pressure, type and number of geotextile layers were all varied so that the behavior of the sample could be examined. The results provide evidence that as the moisture content increases, the peak strength of both the reinforced and unreinforced samples decreases and the axial strain at failure increases. Moreover, with increasing relative compaction the peak strength of the sample and axial strain at failure increases, whereas the peak strength ratio decreases. The peak strength ratio is the ratio of the peak strength of the reinforced samples to that of the unreinforced samples. For soils with low plasticity indices the main cause of the increase in the strength is the increase in the cohesion of the reinforced sample. However, in soils of higher plasticity index, as the number of geotextile layers increases, the internal friction angle of the reinforced samples increases.

1. Introduction

The main limitation to soil structure stability is the low strength of many cohesive soils. By reinforcing the soil with geosynthetics this problem is somewhat overcome. One of the most common geosynthetic materials used to reinforced soil is geotextiles. Several laboratorial and theoretical investigations have been conducted in this field, most of which are related to granular soils reinforced with geotextile, while limited studies have been made concerning cohesive soils reinforced with geotextiles.

Ingold (1979) used a triaxial apparatus to conduct research on reinforced cohesive soils. Ingold and Miller (1983) reported the results of undrained triaxial tests on Kaolin clay reinforced by aluminum plates and permeable plastic. Fabian and Fourie (1986) defined the effect of the permeability of the reinforcing material on the undrained strength of reinforced clay by conducting UU triaxial test on clay reinforced by materials with different values of permeability. Lafleur et al. (1987) used a series of direct shear tests on highly plastic cohesive soil to evaluate and compare the behavior of woven and non-woven geotextiles on the behavior of clay. Krishnaswamy and Srinivasula Reddy (1988) reported the influence of the distance between the reinforced materials as well as moisture content of the sample by using undrained triaxial experiments on silty clay reinforced with a geotextile. Srivastava et al. (1988) studied the behavior of silty soil reinforced with geotextiles by using unconfined and triaxial tests. By analyzing the confining pressure, the number of reinforcing layers and the ratio of height to the diameter of the sample were evaluated. Al-Omari et al. (1989) performed CU and CD triaxial tests in order to study the behavior of clay reinforced with geomesh. Indraratna et al. (1991) studied the behavior of reinforced and unreinforced soft silty clays through UU triaxial test. Non-woven and woven geotextiles were used in that study. The use of non-woven geotextiles for reinforcing a near-saturated silty clay was evaluated by Ling and Tatsuoka (1993) using a plane strain device. Zornberg and Mitchell (1994) gave a comprehensive review of the experimental and analytical studies which focused on the behavior of reinforced cohesive soil. The behavior of reinforced clay was examined in triaxial compression tests under both static and cyclic loading conditions by Unnikrishnan et al. (2002). Effects of the sand layer thickness, moisture content and reinforcement types were evaluated. Vinod et al. (2007) performed a series of undrained triaxial tests on clay specimens reinforced with sand–coir fiber cores. Influence of variables such as ratio of cross-sectional area of sand–coir fiber core to that of the triaxial test specimen, confining pressure, fiber content and fiber aspect ratio on the behavior of the composite soil specimen was studied. Other studies in this field were reported by Ingold and Miller (1982), Ingold (1983), Fourie and Fabian (1987), Miura et al. (1990), Li et al. (1995), Athanasopoulos (1996), Kolias et al. (2005), Lekha and Kavitha (2006), Tang et al. (2007), Sachan and Penumadu (2007), Wang et al. (2007), Prashant and Penumadu

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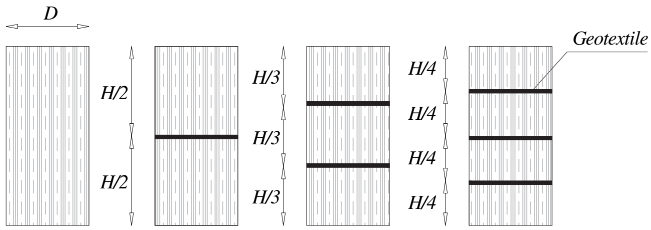

Fig. 1. The arrangement of geotextile in different samples.

(2007), Houston et al. (2008), Kim et al. (2008) and Subaida et al. (2009). In the present study, the mechanical and stress–strain behavior of cohesive soils reinforced with geotextile has been evaluated from a different perspective. Cohesive soils may have a wide range of plasticity indices. The behavior of such soils is also affected by the relative compaction. Although the previously mentioned research was conducted with cohesive soils, the two aforementioned parameters have not yet been evaluated for clays reinforced with geotextile layers.

2. Testing programme

To investigate the effects of varying soil parameters on the mechanical behavior of unreinforced and reinforced cohesive soils, a total of 114 unconfined and 72 triaxial compression tests were performed. Moreover, during the experiments, some of the tests were repeated to determine the accuracy of the results. The experiments were all conducted on a sample of diameter 38 mm and height 76 mm. The procedures for specimen preparation and testing were standardized to achieve repeatability in the test results. All the initial tests were repeated until consistent results were obtained. The different soil and geotextile parameters that were varied during the experiments are:

a. Two types of geotextiles.
b. The number of geotextile layers, illustrated in Fig. 1.
c. Three different moisture contents; two percent below the optimum moisture content, optimum moisture content and two percent above the optimum moisture content (at standard proctor compaction).
d. Three different relative compactions (90, 95 and 100% of the standard compaction).
e. Two types of soil with different plasticity index (Amol clay with plasticity index of 26 and Khalilshahr clay of 11).
f. Three different confining pressures (600, 800 and 1000 kPa).

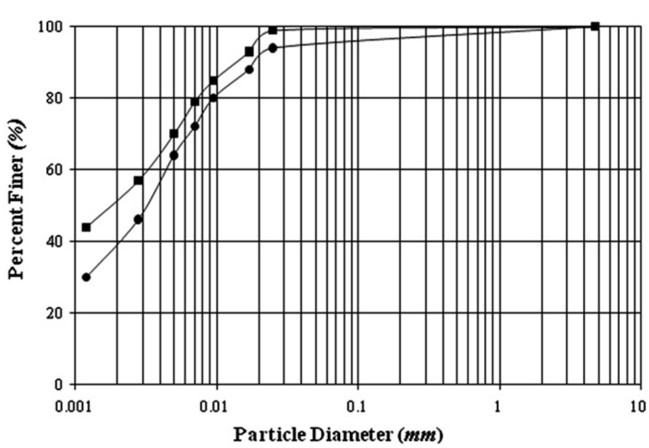

Fig. 2. Grain-size curves for Amol clay.

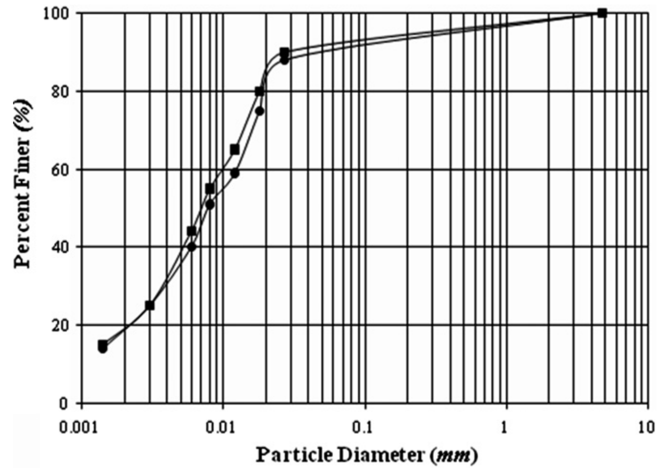

Fig. 3. Grain-size curves for Khalilshahr clay.

All testing was conducted with a strain-controlled rate of 1.5% per minute for the unconfined test and 1% per minute for the triaxial tests.

3. Materials used

Clay soils from Amol and Khalilshahr in the North Iran were used for the testing program. The standard test method for particle-size analysis was done for each soil type, provided in Figs. 2 and 3. The testing procedure was performed according to the ASTM D 422-63 (ASTM, 2003). Clay from Amol classifies as CH using the unified classification system, and clay from Khalilshahr is CL. The clay of Khalilshahr is referred to as type I and the clay of Amol as type II. The physical and compaction properties of the soils are provided in Table 1. All the soil properties were determined by testing as per relevant ASTM standards. Two types of geotextiles were also used in the testing program. The physical and mechanical properties of these geotextiles are provided in Table 2, which were provided by the producing companies, which will be named first type and second type geotextiles, respectively.

4. Preparation of the samples

The preparation of the soil sample is of great importance for laboratorial research. The preparation of the different specimens will be outlined in this section. Initially, the water content of the soils was determined so that the amount of additional water, needed to achieve the desired water content for testing, could be determined. The soils were mixed with water and placed within

Table 1
Physical and compaction properties of the experimented soil types.

Description	Type of soil	
	Type I	Type II
Unified soil classification system	CL	CH
Passing percent No. 200 sieve, %	92	98
Liquid limit, %	35	52
Plastic limit, %	24	26
Plasticity index, %	11	26
Specific gravity of solids, G_s	2.7	2.7
Maximum unit weight (at standard proctor compaction energy), kN/m^3	17.1	15.8
Optimum moisture content (standard proctor compaction), %	18	22

Table 2
Physical and mechanical properties of reinforcing materials used in experiments.

Property	Type of reinforcement	
	First type	Second type
Geotextile	Husker-B40	Terram-3000
Fabrication process	Non-woven	Non-woven
Weight, g/m^2	180	260
Opening size, mm	0.125	0.1
Ultimate tensile strength, kN/m	12.5	18
Strain at ultimate tensile strength, %	30	33
CBR puncture strength, N	2250	3250
Water permeability index normal to the plane, l/m^2/s	75	55

double layered plastic bags and sealed for three days to achieve uniform water content within the soil mass. Moisture content was also determined after the soil has been sealed, which was found to be very close to the target moisture content. A variation of less than 2% in moisture content was observed in the samples.

The construction of the sample was done in a mold of diameter 55 mm and the height 100 mm. The sample was prepared in three equal layers. A static compaction method was applied to the soil layers as reported by Unnikrishnan et al. (2002). In order to obtain a sample with a diameter of 38 mm and a height of 76 mm, a hydraulic jack penetrated statically into the mold.

To construct the reinforced sample, knowing how many reinforced layers are needed, the unreinforced sample is cut by a narrow saw wire, and the reinforcing material is placed. This method of making reinforced samples was reported by Ingold and Miller (1983). In order to cut the sample horizontally, three different molds were made for three different types of the reinforced sample. These molds may leave some grooves on the sample resembling the ones made by narrow saw wire, so that the sample surface is cut exactly in the intended position, as shown in Fig. 4. All reinforced samples were constructed through this method.

5. Results and discussions

In this section, the results of the unconfined and triaxial compression tests are presented, discussed and evaluated. Two parameters, the 'peak strength ratio' and 'residual strength ratio', are defined before discussing the results. The peak strength ratio is the ratio of the peak strength of the reinforced samples to that of the unreinforced samples. The residual strength ratio is the ratio of the residual strength to the peak strength.

5.1. Unconfined test analysis

The stress–strain curves for the unconfined compression tests of the reinforced and unreinforced clays are shown in Fig. 5. The curves provide evidence of an improvement in the mechanical properties of clay with the addition of the geotextile. Stress–strain behavior of soil improved with an increase in the number of geotextile layers. This was also reported by Ingold (1979), Ingold and Miller (1983), Fabian and Fourie (1986), Fourie and Fabian (1987), Indraratna et al. (1991) and Athanasopoulos (1996). One possible explanation for such a behavior could be that the geotextile layers intercept the failure plane within the specimen, distributing the stresses evenly within the soil and hence, increasing the overall strength of the reinforced soil.

As illustrated in Fig. 5, the reinforced samples have higher peak strength in comparison the unreinforced soil, and as the number of geotextiles increases, the strength increases further.

It is also evident that the geotextile increases the axial strain at failure and also the residual strength ratio; meaning that the geotextile causes a decrease in the strength loss after the peak strength. This phenomenon is more apparently seen as the number of geotextile layers increases, i.e. reinforced samples behave more ductile than unreinforced samples, which is because of the flexibility of the geotextile that influences the ductility of the reinforced sample. Another reason is geotextile reinforcing prevents shear band development in the samples, which is the main cause of strength loss after the peak strength in unreinforced clay samples. It is important to notice that, non-woven geotextiles have a high axial strain at failure, and therefore it is nearly impossible for geotextiles to rupture during a traditional triaxial test. This point was confirmed by checking the geotextiles at the end of experiments. The geotextiles were visually inspected for rupture following each test. In these experiments, the tensile strength of the geotextiles was not a dominant factor in the failure of samples. Therefore, the post-peak strength decreases gradually or remains constant. On the other hand, the geotextile inclusions reduced post-peak loss of strength. Higher ductility and less loss in post-peak strength of non-woven geotextile reinforced samples are the advantages of this reinforcing material in comparison with the unreinforced.

The stiffness of the reinforced specimen was found to be less than that of the unreinforced specimen. This fact is more clearly seen as the number of geotextiles increases. Referring to the study of Haeri et al. (2000), this behavior may be justified by the load-elongation curve of the geotextile, which is readily provided by the manufacturer. The load-elongation curves of the two types of geotextile used in the present study were nearly identical, as the

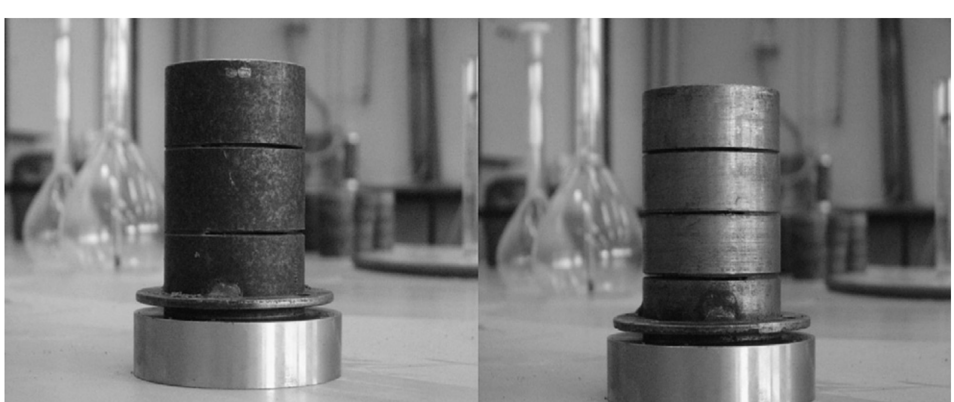

Fig. 4. View of the molds used for preparing the reinforced samples.

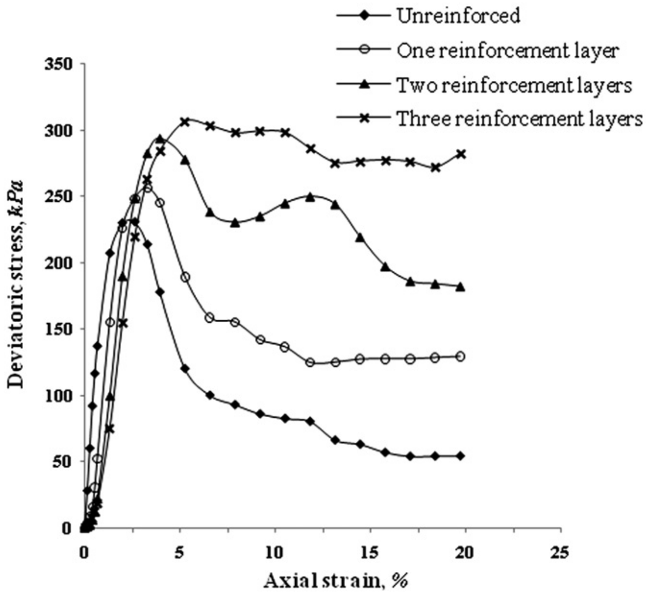

Fig. 5. Stress–Strain curves for unreinforced and reinforced clay of type II with several layers of first type geotextile for the moisture content 22% and the relative compaction of 90%.

failure strain for the first and second type geotextiles was reported as 30 and 33 percent, respectively by the manufacturer. This is why it is observed in Fig. 6 that the stiffness of the samples reinforced with the first and second types of geotextile is nearly the same.

The effect of geotextile type is illustrated in Fig. 6. The first type geotextile has a greater influence on the sample strength. The reason may be due to the difference in permeability of the two types of geotextiles. As reported in Table 2, the first type geotextile has a higher permeability than the second type. Therefore, since the sample reinforced by this geotextile has higher peak strength than the one reinforced by the second type geotextile, it can be concluded that the permeability affects the effectiveness of the geotextile even for what is typically considered undrained conditions.

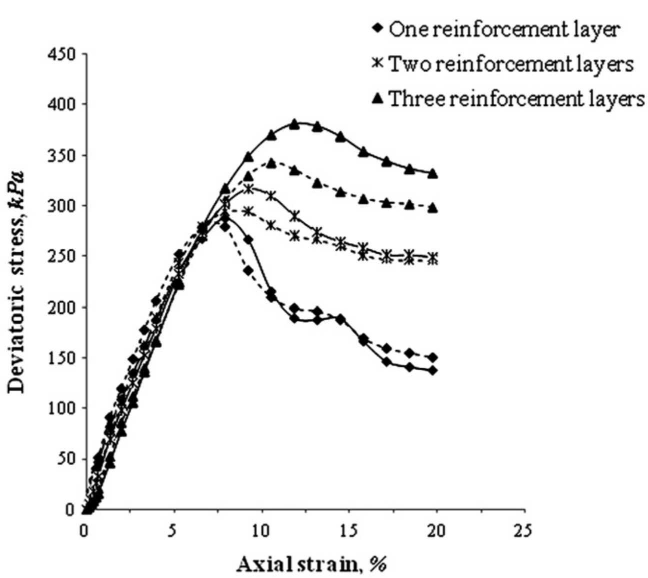

Fig. 6. Stress–Strain curves for type I clay with relative compaction of 100% and moisture content of 20%: first type geotextile (solid line) – second type geotextile (dashed line).

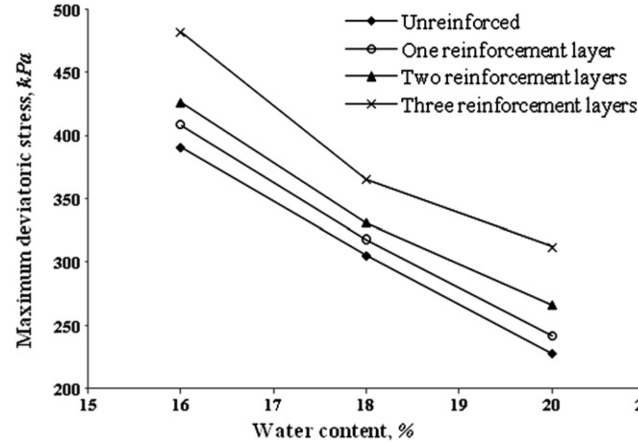

Fig. 7. The variations of unconfined peak strength based on moisture content for type I clay with relative compaction of 95% with different numbers of geotextile layers of the first type.

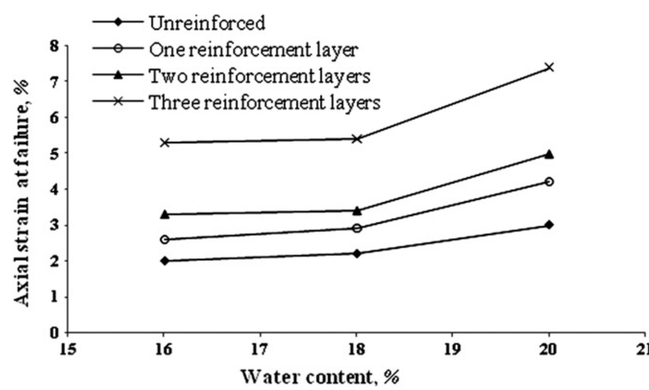

Fig. 8. The variations of axial strain at failure based on moisture content for type I clay with relative compaction of 90% for different numbers of geotextile layers of the first type.

Table 3
Influence of moisture content on peak strength ratio for soil of type I with the first type geotextile.

Water content, (%)	Peak strength ratio		
	One reinforcement layer	Two reinforcement layer	Three reinforcement layer
Dry side of OMC	1.01	1.04	1.13
OMC	1.01	1.05	1.15
Wet side of OMC	1.03	1.14	1.34

Table 4
Influence of moisture content on peak strength ratio for soil of type II with the first type geotextile.

Water content, (%)	Peak strength ratio		
	One reinforcement layer	Two reinforcement layer	Three reinforcement layer
Dry side of OMC	1.07	1.15	1.30
OMC	1.06	1.12	1.23
Wet side of OMC	1.01	1.05	1.20

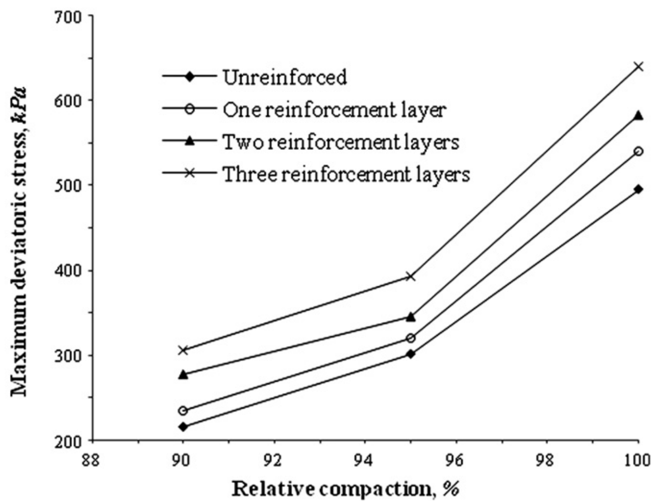

Fig. 9. The variations of peak strength based on relative compaction for type II clay with moisture content of 20% for different numbers of geotextile layers of the first type.

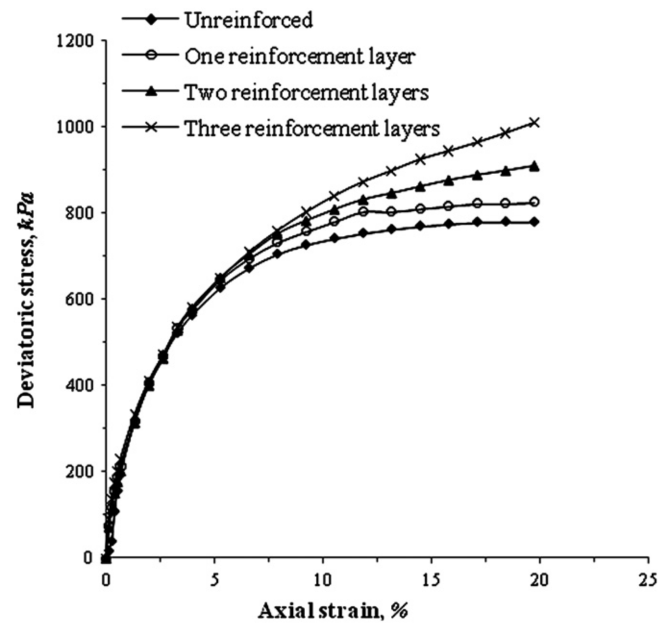

Fig. 11. Stress–Strain curves for unreinforced and reinforced clay of type I, for the relative compaction of 95% and the confining pressure 600 kPa.

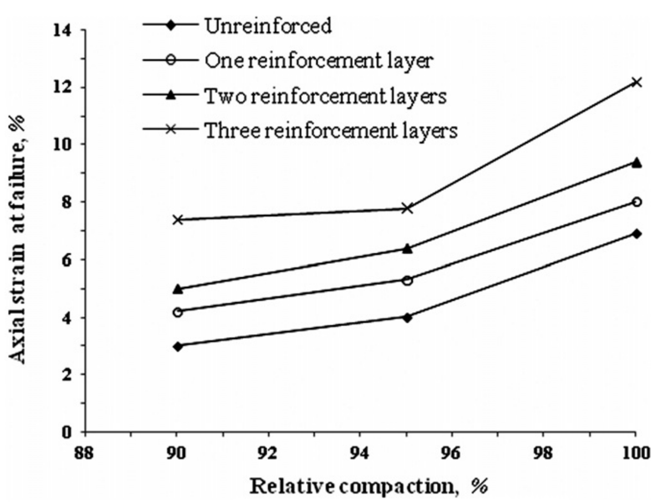

Fig. 10. The variations of axial strain at failure based on relative compaction for type I clay with moisture content of 20% with different numbers of geotextile layers of the first type.

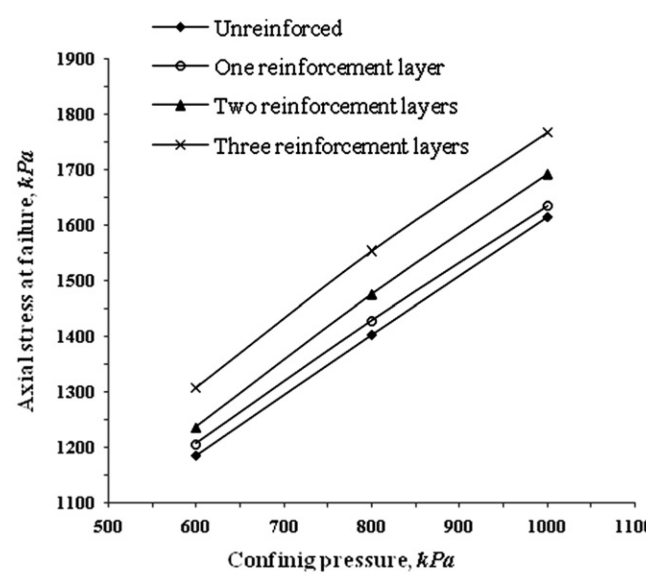

Fig 12. Failure envelopes for unreinforced and reinforced clay of type I for different numbers of geotextiles of first type for the relative compaction of 90%.

Table 5
Influence of relative compaction on peak strength ratio for soils of type I and II.

Type soil	Relative compaction, (%)	Peak strength ratio		
		One reinforcement layer	Two reinforcement layer	Three reinforcement layer
Type I	90	1.13	1.22	1.38
	95	1.01	1.07	1.19
	100	1.00	1.04	1.15
Type II	90	1.08	1.12	1.28
	95	1.04	1.10	1.18
	100	1.05	1.07	1.15

Table 6
Strength parameters for unreinforced and reinforced clay of type I, the relative compaction of 100%.

Geotextile arrangements	Shear strength parameters	
	Cohesion (C), kPa	Angle of internal friction (Φ), °
Unreinforced	258	8.4
One reinforcement layer	276	8.6
Two reinforcement layers	297	8.4
Three reinforcement layers	323	8.7

The influence of the moisture content on the behavior of samples of type I soil is shown in Figs. 7 and 8. As the moisture increases, the peak strength decreases and the axial strain at failure increases. This phenomenon can be explained according to the structure of cohesive soil. Studies of compacted soils at the micro level have shown that soils compacted on the dry side of optimum moisture content have a flocculent structural arrangement of particles. On the wet side of optimum moisture content, the structure is dispersed. Samples compacted dry of optimum tend to be more rigid and stronger than samples compacted wet of optimum. This change in behavior can be clearly seen in the results

Table 7
Strength parameters for unreinforced and reinforced clay of type II, the relative compaction of 90%.

Geotextile arrangements	Shear strength parameters	
	Cohesion (C), kPa	Angle of internal friction (Φ), °
Unreinforced	257	2.1
One reinforcement layer	260	2.3
Two reinforcement layers	250	4.0
Three reinforcement layers	253	5.3

of the present work. As illustrated in Figs. 7 and 8, it is found that for a cohesive soil a constant relative compaction, an increase in the moisture content results in lower peak strength, with an increase in the axial strain at failure. This was true for both soil types and for different relative compactions.

The influence of the moisture content on the behavior of the reinforced is also provided in reference to the peak strength ratio, as provided in Tables 3 and 4 for both soil types. For the samples of type I soil as the moisture content increases, peak strength ratio also increases indicating that the reinforcement effect on the dry side of the OMC is less than that of higher water contents. This phenomenon is also in agreement with the results of the experiments of Fabian and Fourie (1986). However, for samples of type II soils, as the moisture content increases, peak strength ratio decreases and reinforcement effect on the samples on the wet side of OMC is less than other states.

The influence of relative compaction changes on the sample behavior is illustrated in Figs. 9 and 10. As the relative compaction increases, the peak strength of the sample also increases. However, the more interesting phenomenon is that as the relative compaction increases, the axial strain at failure also increases. This means that the sample of higher compaction has a higher axial strain at failure than the sample of lower compaction. This phenomenon occurs for both the soil types and over the range of moisture contents. In order to study relative compaction effect on reinforced samples, Table 5 has also been presented. As seen in the table, when the relative compaction increases, the peak strength ratio decreases, or the effect of the reinforcement on the strength decreases as the relative compaction increases.

5.2. Triaxial test analysis

Fig. 11 is an example of a stress–strain diagram obtained by UU triaxial test. The results provide evidence that the reinforcement has improved the strength properties of the soil specimen. By reinforcing the sample the peak strength increases; which is more obvious when more layers are present. It was also found that for low axial strains, almost below 5%, the reinforcement does not considerably influence the behavior of axial stress–strain of the samples. This was observed in all triaxial tests. These results further indicate that at low axial strains, in which displacements and stresses along the soil–geotextile interface are low, the effects of the geotextile are negligible. Only at higher axial strains will the geotextile on the soil strength be appreciable.

Table 8
Influence of confining pressure on peak strength ratio for soil of type I, the relative compaction of 90%.

Confining pressure, (kPa)	Peak strength ratio		
	One reinforcement layer	Two reinforcement layer	Three reinforcement layer
600	1.11	1.22	1.28
800	1.05	1.13	1.23
1000	1.03	1.09	1.2

Table 9
Influence of confining pressure on peak strength ratio for soil of type II, the relative compaction of 90%.

Confining pressure, (kPa)	Peak strength ratio		
	One reinforcement layer	Two reinforcement layer	Three reinforcement layer
600	1.03	1.09	1.19
800	1.04	1.11	1.23
1000	1.04	1.14	1.26

The failure envelope for the soil specimens moves upward as the number of reinforcing layers increases, as shown in Fig. 12. For the same confining pressure and relative compaction, the peak strength increases as the number of geotextiles increases. An interesting result from this research was that for type I soil, the main increase in the peak strength is caused by an increase of cohesion in the reinforced sample with number of geotextile layers, and there is no considerable change in the internal friction angle. This data is also provided in Table 6, which provides the cohesion and internal friction angle of type I soil. These results are in agreement with the results reported by Srivastava et al. (1988). However with the type II soil, as the number of geotextile layers increases, the internal friction angle of reinforced samples increases but there is no considerable changes in the cohesion, as provided in Table 7.

The influence of confining pressure on the behavior of the reinforced samples is presented in Tables 8 and 9 for soils of type I and II. For samples of type I soil, as the confining pressure increases, the peak strength ratio decreases which is also in conformity with Srivastava et al.'s (1988) observations. However, for samples of type II soils, as the confining pressure increases, the peak strength ratio increases.

6. Conclusions

By conducting unconfined and triaxial compression tests on unreinforced and reinforced clays of Amol and Khalilshahr the following results were obtained:

1. Reinforcing improves the mechanical properties of soil, which means the existence of geotextiles increases the peak strength, axial strain at failure and decreases strength loss after the peak strength. Also, the reinforced samples are less stiff than the unreinforced ones. The improvement of mechanical properties increases as the number of geotextile layers increases.
2. The comparison of samples reinforced by two different types of geotextiles provides evidence that the permeability of the geotextile may have an important role on the strength of the sample. The more permeable the geotextile, the higher the peak strength of the clay soil.
3. As the moisture content increases, for both soil types and for unreinforced and reinforced samples of different numbers of geotextiles, the peak strength of the samples decreases and the axial strain at failure increases. Also, for sample of type I soil as the moisture content increases, the peak strength ratio increases. However, for samples of type II as the moisture content increases, the peak strength ratio decreases.
4. For unreinforced and reinforced samples of both soil types with different numbers of geotextile layers, and for different moisture contents, as the relative compaction increases, the peak strength of the samples and the axial strain at failure increase. Also as the relative compaction increases, the peak strength ratio decreases.

5. For sample of type I soil, the main increase in the peak strength is caused by the increase of cohesion in the reinforced sample, and there is no considerable change in the internal friction angle (slightly decreases or increases). However, for samples of type II soil, as the number of geotextiles increases, the internal friction angle of reinforced samples increases but there was no considerable change in the cohesion.
6. In samples made of soil type I as the confining pressure increases, the peak strength ratio decreases. But for samples made of soil type II as the confining pressure increases, the peak strength ratio increases.

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