Journal of Earth Science, *Vol. 27*, *No. 6*, *p. 1060–1070*, *December 2016* *ISSN 1674-487X*
Printed in China
DOI: 10.1007/s12583-015-0621-0

Laboratory Study of Geotextiles Performance on Reinforced Sandy Soil

Brahim Abdelkader[1], Arab Ahmed*[1], Belkhatir Mostéfa[1], Shahrour Isam[2]
1. *Laboratory of Materials Sciences and Environment Civil Engineering Department*, *Hassiba Ben Bouali University of Chlef*, *Chlef* 02000, *Algeria*
2. *Laboratory of Civil Engineering and Geo-Environment* (*LGCGE*), *University of Lille* 1, *Villeneuve d'Ascq* 59650, *France*

ABSTRACT: This paper presents the results of triaxial tests conducted for the investigation of the influence of geotextiles on stress-strain and volumetric change behaviour of reinforced sandy soil. Tests were carried out on loose sandy soil. The experimental program includes drained compression tests on samples reinforced with different values of both geotextiles layers (*N*g) and confining pressure (σ'_c). Two methods of preparation were used: air pluviation (AP) and moist tamping (MT). Test results show that the geotextiles induce a quasi-linear increase in the stress deviator (*q*) and volume contraction in the reinforced sand. Method of preparation significantly affects the shear strength; samples prepared by the air pluviation method and mobilized deviator stresses are significantly higher than those prepared by moist tamping method. Geotextiles restrict the dilation of reinforced sandy soil and consequently the contraction increases. The mobilized friction angle increases with increasing number of layers and decreases with increasing initial confining pressure. Samples prepared by moist tamping present mobilized friction angles significantly lower than those prepared by air pluviation method. For samples prepared by the air pluviation method, the secant modulus at ε_1=1% and 5% decreases with increasing geotextile layers; those prepared by the moist tamping method, secant modulus at ε_1=1% and 5% increases with increasing number of geotextile layer sand confining pressure. From 10% axial strain, secant modulus increases with increasing inclusions of geotextile layers.
KEY WORDS: sand, triaxial, geotextile, drained, method of preparation, strength.

0 INTRODUCTION

The region of Chlef is situated in the north of Algeria about 210 km west of the capital Algiers. This region was subjected to intense seismic activity; it's constantly a very instable zone. In the last century, it underwent destructive earthquakes (Orléansville, ex El Asnam and now Chlef) in 1922, 1934, 1954 and 1980. These earthquakes which have been described and reported well by McKenzie (1972), Thevenin (1955) and Rothé (1955), caused the deaths of a great number of people (1 340 died in 1954) and a significant damage to different infrastructures and civil engineering structures. The earthquake of October 10, 1980 at 13:25 (local time) with a magnitude of 7.3 according to Papastamatiou's calculations (1980), followed by two significant jolts of magnitude 6.1 and 6 within an interval of several hours and by numerous shocks over the following months. The earthquake of Chlef (1980) caused the loss of numerous lives (about 2 800 deaths) and caused the destruction of a great number of buildings, bridges and public equipments. The seismic vibration also generated a number of geodynamic phenomena within the surface of the soil: movements of the ground and especially the liquefaction of the sandy soils following a loss of resistance to shearing (landslides, subsidence of the bunk...) are shown in Figs. 1a, 1b and 1c. According to Durville and Méneroud (1982), the phenomenon of liquefaction appeared at a vast alluvial valley crossed by the Chlef River and at the zone of confluence of this river with the Rass and Fodda rivers (Fig. 1d).

Recently, the improvement of soils by the use of synthetic materials is becoming popular in the field of civil engineering, especially, in the soil stability (slope stability, embankment, road, filter, drain). The advantageous effect of this synthetic material is due to the shape in which it is used as reinforcement.

We know that cohesive and noncohesive soils present limits regarding the stability of their structure when they are subjected to high loading condition. Several studies were conducted in laboratories dealing with the reinforcement of granular and cohesive soils with geotextiles. The published literature reported that the soil could be reinforced with synthetic layers, fibers and geocells, etc..

* Corresponding author: ah_arab@yahoo.fr;
abelkhatir@yahoo.com
© China University of Geosciences and Springer-Verlag Berlin Heidelberg 2016

Manuscript received June 7, 2015.
Manuscript accepted October 9, 2015.

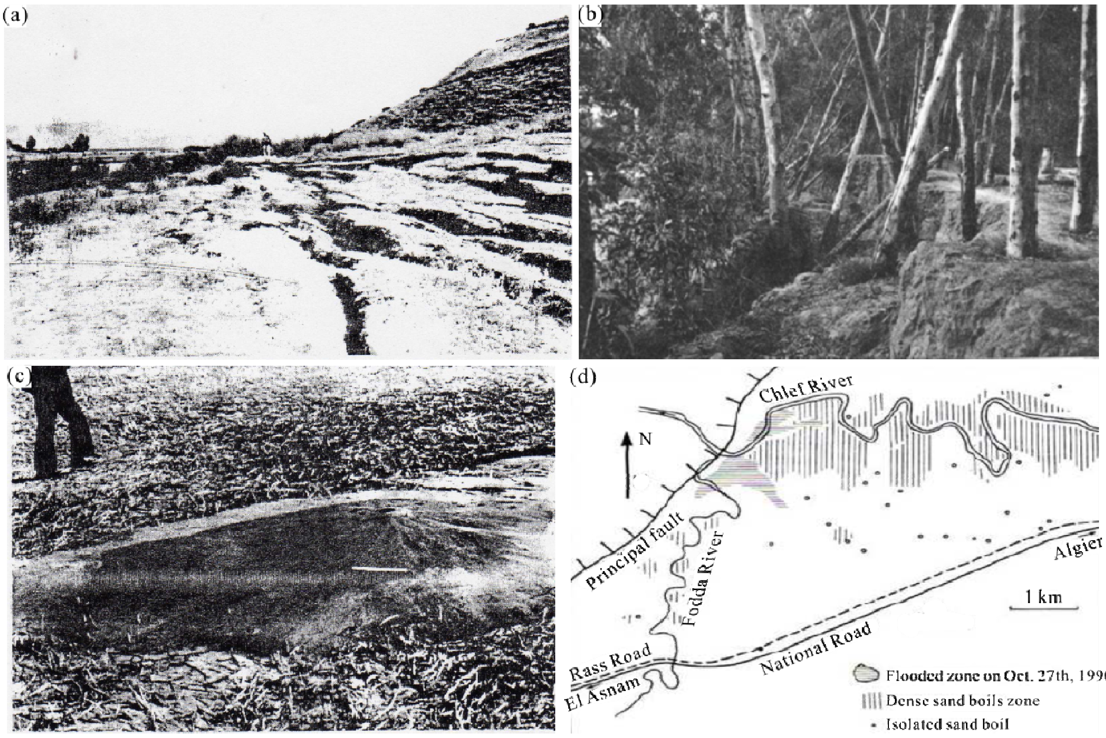

Figure 1. Photographs showing Geodynamic phenomena (a) subsidence banks due to liquefaction, (b) collapse of trees of Chlef River, (c) sand boils, (d) sketch map showing the valley of Chlef River and location of the sands boils due to liquefaction phenomenon.

Since 1970, several researchers have studied stress-strain and the characteristics of the loading conditions in reinforced soils using the triaxial device, shear box, and plane strain by different tests. From 1977, an important experiment was realized on reinforced sand with geotextiles.

Broms (1977) illustrated by performing triaxial tests on sand samples reinforced with geotextile layers the reduction of the lateral pressure of the soil. He observed that the shear strength peak increased with the decrease of the geotextile layers spacing. However, he noticed that the strength peak was not influenced when the geotextile layers were placed at the end of the sample. McGowan et al. (1978) carried out a series of tests in a cell with plane strain on dry sand reinforced with aluminium and non-woven geotextiles. They concluded that the behaviour in terms of geotextiles relatively low in comparison with that of the aluminum.

Gray and Al-Rafeai (1986) conducted compression tests on reinforced dry sand with different types of geotextiles. The test results demonstrated that these inclusions increased the soil strength, the axial deformation in the failure, and, in several cases, lead to a reduction of the strength after the deviatoric stress peak. Athanasopoulos (1993) reported that the effect of the grain size on the mechanical behaviour of a sand strengthened by geotextile in the direct shear box device. The results indicated that the dilative behaviour of the reinforced sand was significantly influenced by the opening ratio (defined as being the ratio of the geotextile opening to the average grain size of the sand). They found that for the fine sand (high opening ratio values), the reinforcement increased the expansion volume compared with that of the unreinforced sand, whereas the inverse behavior was recorded for the coarse sand (low opening ratio values).

Krishnaswamy and Isaac (1994) presented the results of cyclic triaxial tests on sand samples of diameters of 38 and 100 mm to estimate the liquefaction potential of sand reinforced with geotextile. They have shown that the technique of reinforcement can be a promising solution for the decrease of the liquefaction potential of the tested soil (sand).

To study the reinforcement effect by geotextiles on the stress-strain and volume change response of the sand subjected to monotonic and cyclic loading conditions, Ashmawy and Bourdeau (1998) carried out drained triaxial tests on saturated sand samples reinforced with woven and non-woven geotextile layers. The results revealed that the presence of reinforcement induced a significant increase of the monotonic shear strength of the sand and a decrease of the cyclic shear strain.

Unnikrishnan et al. (2002) studied the behaviour of reinforced clay subjected to monotonic and cyclic loading condition. The effect of the reinforcement thickness, content and type of reinforcement has been evaluated. The results obtained indicated that the reinforcement used improved effectively the stress-strain behaviour under monotonic and cyclic loading. Other studies in the field have been reported by Haeri et al. (2000), Houston et al. (2008), Ling and Tatsuoka (1993), Tang et al. (2007), and Wang et al. (2007).

This paper presents a laboratory study to assess the reinforcement effect of the Chlef sand with one type of non-woven (NW) under monotonic drained tests loading. The tests were carried out on samples with an initial relative density I_d=0.10 and 0.50 and an initial effective pressure σ'_c=50, 100 and 200 kPa; for the purpose to better understand the influence of geotextiles on the stress-strain and volumetric change behaviour of the reinforced soils. Tests were conducted on loose sand reinforced with different geotextile layers. Two preparation methods were used, namely, air pluviation (AP) and moist tamping (MP). Analyses of these results lead to interesting results concerning the influence of the confining pressure and the number of geotextiles layers on the enhancement of the soil behaviour.

1 MATERIALS

Laboratory tests were carried out on loose medium-grained sand from the Chlef River in Algeria. The sand is composed of rounded particles with a mean grain size D_{50}=0.61 mm, D_{10}=0.225 mm and a uniformity coefficient D_{60}/D_{10}=3.38. The granular distribution of the sand is illustrated in Fig. 2. Tests were conducted with a propylene non-woven geotextile (BidimS72). The characteristics of the geotextile are summarized in Table 1. The tensile axial strength is equal to 25 kN/m, while the axial strain at the maximum axial tension is equal to 80%. The filtration opening size is equal to 85 μm, which is equal to 13% of the mean grain size of the Chlef sand used in the study. The physical and mechanical properties of the geotextile are summarized in Table 1.

2 TESTING PROGRAM

To investigate the effect of varying soil parameters on the mechanical behaviour of unreinforced and reinforced sandy soil a total of 70 triaxial compression tests were performed. Moreover, some experiments were repeated to determine the accuracy of the result. The experimental program consisted of performing triaxial compression tests on 70 mm×70 mm dry sand. The sandy soil and geotextiles parameters that were varied during the tests are: (1) method of preparation; (2) number of geotextile layers (Fig. 3); (3) three different confining pressures (50, 100 and 200 kPa); (4) all tests were conducted with a strain-controlled rate of 0.17% per minute.

3 PREPARATION OF SAMPLES

The preparation of the soil sample is of great importance for laboratory research. Two methods were used in our study (air pluviation and moist tamping). The fabrication of the sample was done in a mould of diameter 70 mm×70 mm. The sample was prepared in equal layers. To fabricate the reinforced sample, many reinforced layers are needed (Fig. 3). The sample is firstly swept by carbon dioxide CO_2 for 20 minutes, and then is passed demineralised water through the sample to saturate it. The samples were isotropically consolidated to reach the value of effective confining stress prior to loading. The back pressure used for our tests was 400 kPa, the cell pressure is 450, 500 and 600 kPa. The degree of saturation of the samples was evaluated by measuring the Skempton's coefficient B ($B=\Delta\mu/\Delta\sigma$).

4 RESULTS AND DISCUSSION

In this section, the results of the drained compression triaxial tests are presented, discussed and evaluated.

4.1 Drained Tests Conducted on Loose Sand

The results of stress strain curves for the drained compression tests of the reinforced sandy soil reconstituted with air

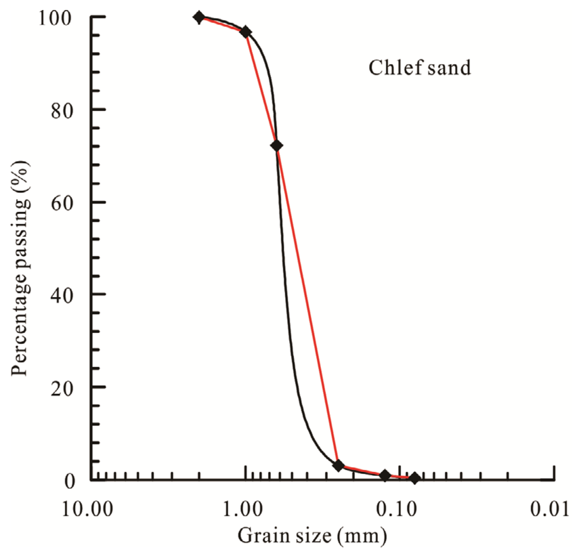

Figure 2. Granular distribution for the soils used in this study.

Table 1 Physical and mechanical properties of reinforcing materials used in experiments.

Properties	Type of reinforcement
Geotextile	BidimS72
Manufacturing process	Non-woven
Resistance to traction T_{max} (kN/m)	25
Deformation in the tensile stress ε_{max} (%) SP	80
Deformation in the tensile stress ε_{max} (%) ST	70
Dynamic perforation (mm)	15
Permeability (m/s)	0.055
Opening size (μm)	85
Surface mass (g/m^2)	305
Thickness in 2 kPa (mm)	2.7

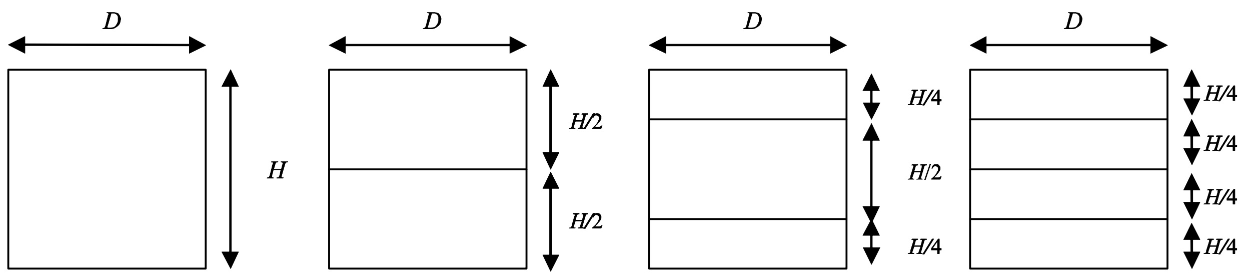

Figure 3. Geotextile arrangements for triaxial tests.

pluviation method are shown in Figs. 4, 5, 6. The curves of the drained response provide evidence of an improvement in the mechanical behaviour of sandy soil with the addition of geotextiles layers. We note that the stress-strain behaviour of sandy soil improved shows an increase of the resistance with increasing number of geotextiles (Figs. 4a, 5a and 6a). Figures 4b, 5b and 6b show the volume behaviour of the sandy soil with the increasing of the number of geotextiles. It's very clear that the increasing in number of geotextiles leads to increase of the contractancy of the sandy soil, where the volumetric strain passes from 2.5% to 25%, consequently the geotextiles reduce significantly the dilalatancy of sandy soil. As reported by many researches (Duncan and Dunlop, 1968), dilation occurs mainly in the center of specimen because the cap and the base restrain lateral deformation and dilation. As it's shown in Fig. 7, the geotextiles effectively restrict the dilation of the samples and consequently the contractancy increases. Effect becomes more apparent when the number of geotextile layers increases. The samples tested at 100 and 200 kPa confining pressure (Figs. 5 and 6) exhibit more contraction than sample tested at 50 kPa, dilation of sandy soil is also reduced by increasing of geotextile layers. Figure 7 shows the pattern of the sample deformation at failure. It can be observed that failure occurs par bulging between layers. This result is similar to that obtained by Madhavi Latha and Murthy (2007).

The stress-strain curves results for the drained compression tests of the sandy soil reconstituted with reinforced moist tamping method are shown in Figs. 8, 9, 10. We note that the stress-strain behaviour of improved sandy soil shows an increase of the resistance with increasing number of geotextiles. By comparing these results with those of the samples prepared with dry pluviation; it can be seen that samples prepared with moist tamping present a lower strength than those prepared with dry pluviation method. These differences of behaviour noted between the two methods of deposition can be explained by the fact that the molecules of water contained in the structures prepared by wet deposition method constitute some macropores easily compressible at the time of the shearing of the sample, and at the same time prevent the grain-grain and grain-geotextile adhesion. The volumetric behaviour of the samples prepared with moist tamping exhibits more contractancy than those prepared with dry pluviation (Figs. 8b, 9b and 10b). The sandy soil dilation is also reduced by increasing confining pressure (100 and 200 kPa). Figure 11 shows clearly the influence of preparation method on the drained

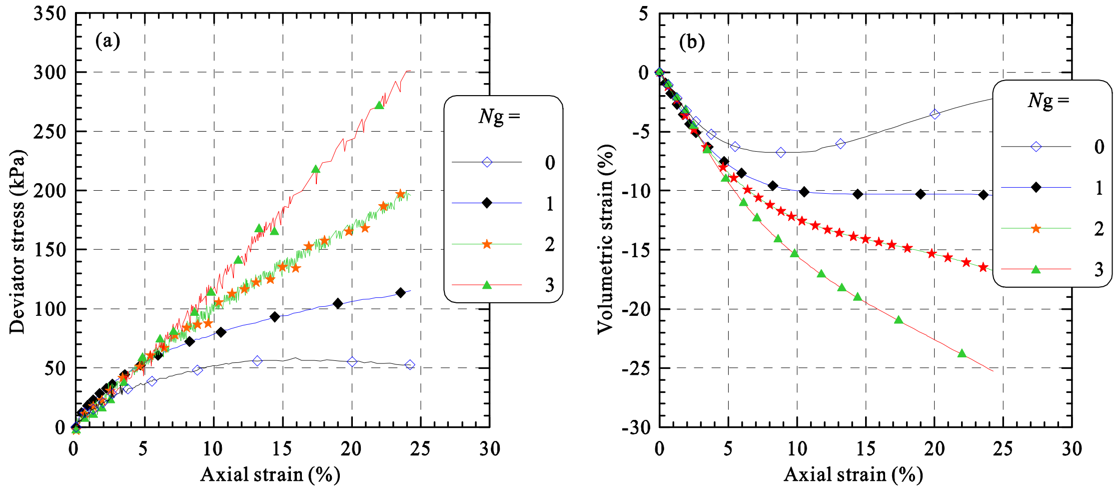

Figure 4. Drained compression tests conducted on reinforced sandy soil (σ'_c=50 kPa).

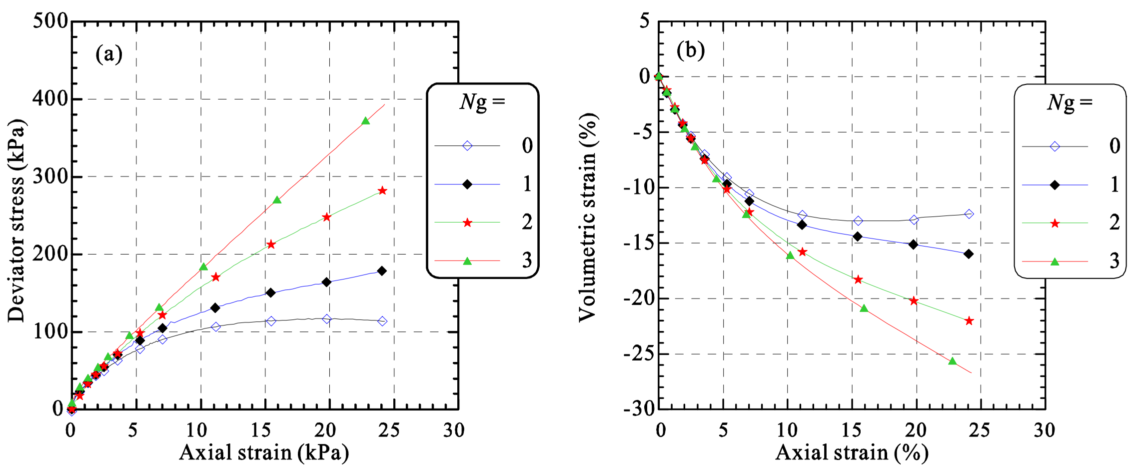

Figure 5. Drained compression tests conducted on reinforced sandy soil (σ'_c=100 kPa).

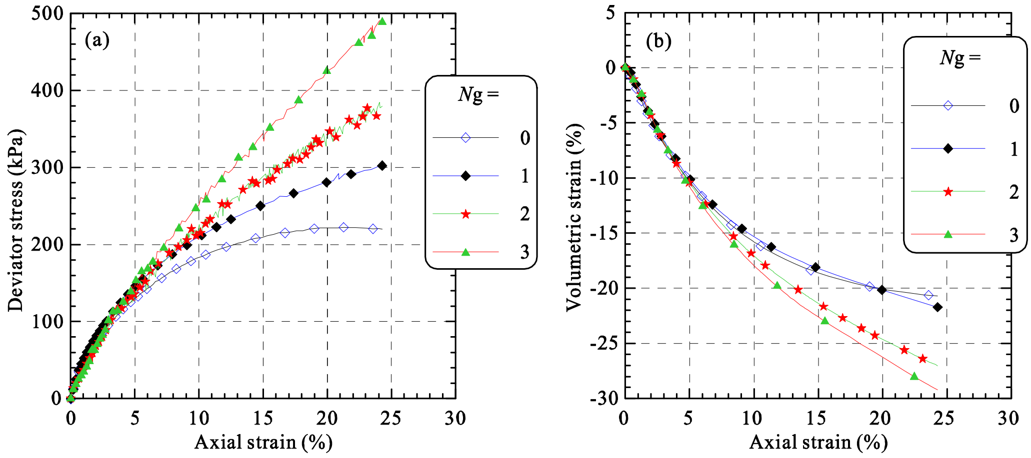

Figure 6. Drained compression tests conducted on reinforced sandy soil (σ'_c=200 kPa).

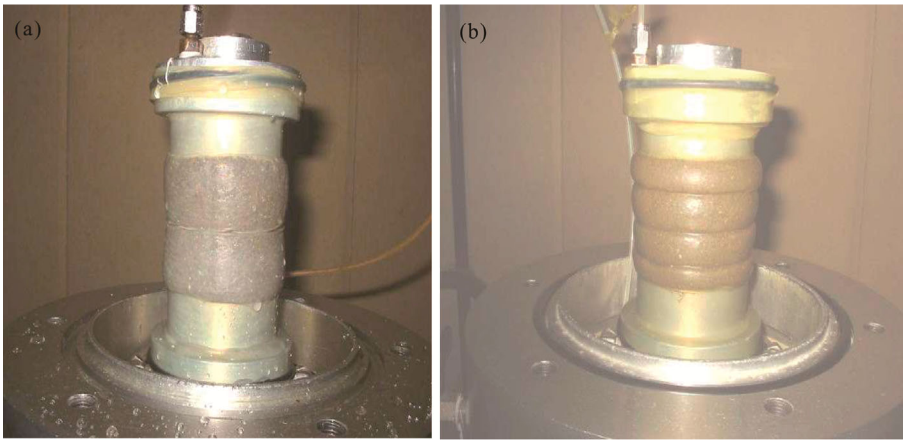

Figure 7. Deformation pattern of the reinforced sand sample.

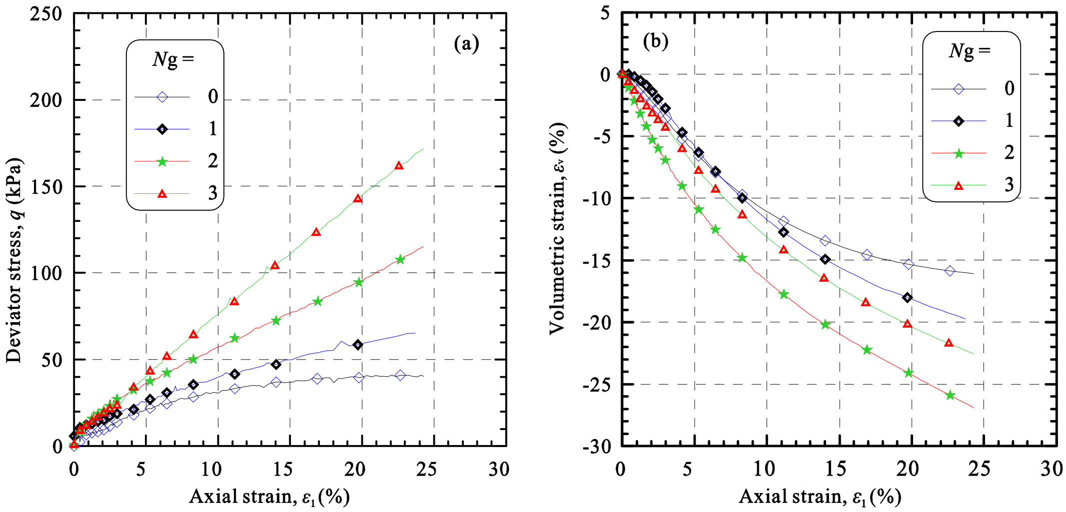

Figure 8. Drained compression tests conducted on reinforced sandy soil (σ'_c=50 kPa).

shear strength response of reinforced sandy soil. The shear strength decreases significantly and we note that the resistance decreases significantly; and there has been a loss of strength of 36%, 34% and 24% for samples prepared with moist tamping and sheared under initial effective stress σ'_c=50, 100 and 200 kPa, respectively.

4.2 Influence of Preparation Method

Figure 12a shows the influence of geotextiles on the shear strength. The test results reveal clearly that the samples prepared by dry pluviation techniques exhibit higher shear stiffness than those prepared by moist tamping. It can be seen that the shear strength of the reinforced samples increases logarithmically (R^2=0.99 for all the curves) with increasing number of geotextiles according to the following expression (1), the values of coefficient A and B are summarised in Table 2. The evolution of the coefficient A (slope) according to the initial confining pressure is illustrated in Fig. 13, we notice that the evolution of these slopes follows an almost linear evolution for both methods of preparation and are almost parallel (C_c=0.97 (AP) and C_c=0.96 (MT).

$$\log (q_{ss})=A\times Ng+B \tag{1}$$

where q_{ss} is deviator stress at the end of shearing, Ng is number of geotextiles.

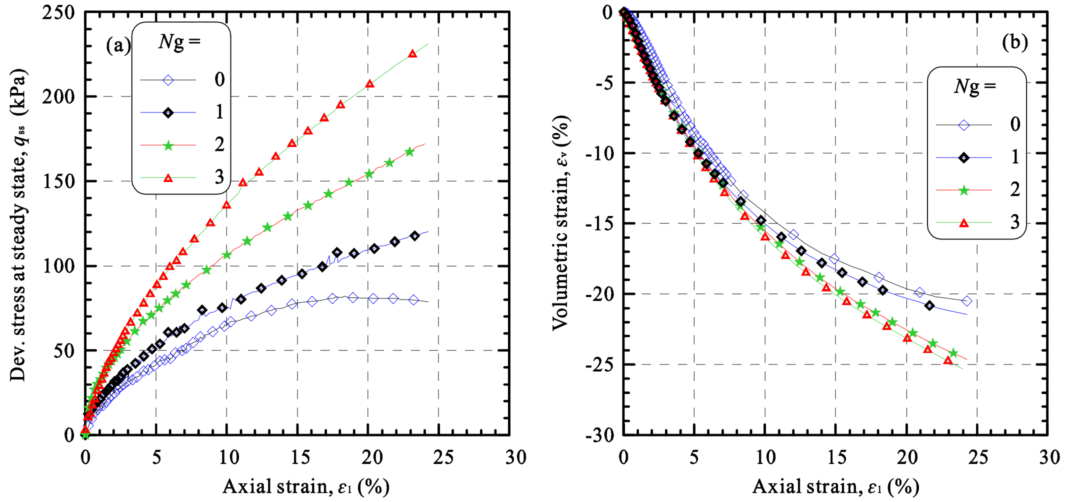

Figure 9. Drained compression tests conducted on reinforced sandy soil (σ'_c=100 kPa).

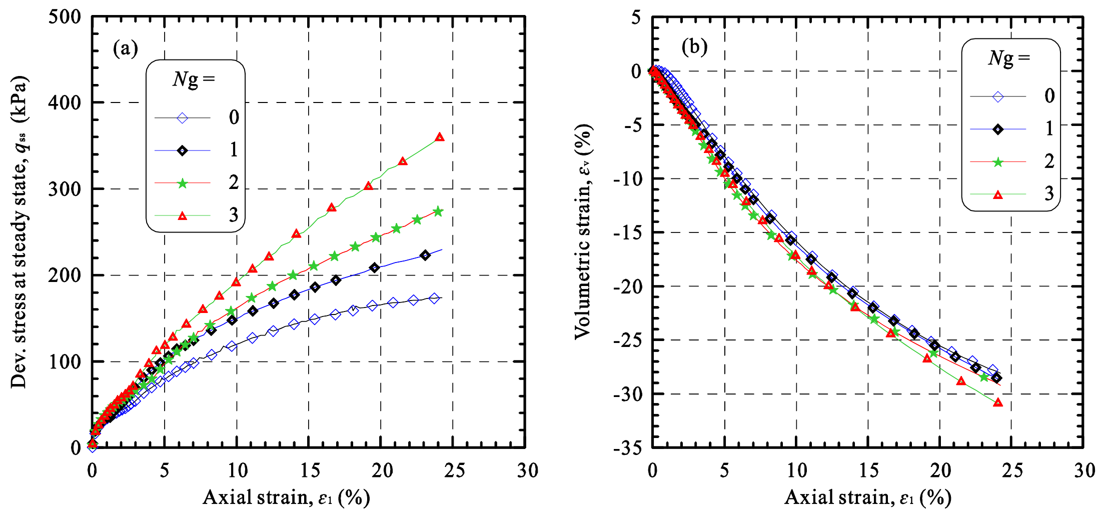

Figure 10. Drained compression tests conducted on reinforced sandy soil (σ'_c=200 kPa).

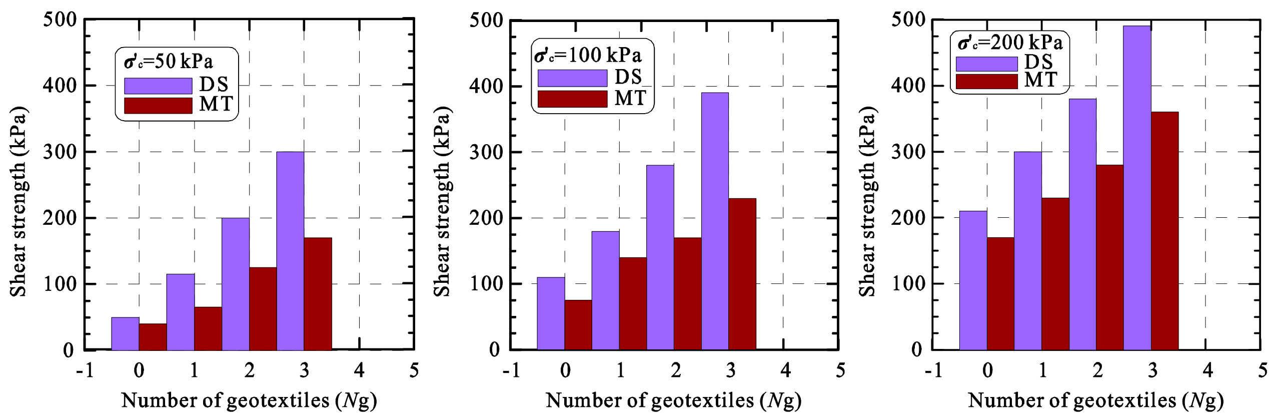

Figure 11. Influence of methods of preparation on the shear strength.

Figure 12b shows the influence of geotextiles on the volume behaviour at the end of shearing. We notice that the evolution of the volumetric strain versus to number of geotextiles displays a linear regression according to the expression (2) inducing influence of geotextiles and confining pressure in increasing the soil contraction, the values of the coefficients C and D are illustrated in Table 3.

$$\varepsilon_v(\%)=C\times Ng+D \tag{2}$$

where $\varepsilon_v(\%)$ is volumetric strain in percentage, Ng is number of geotextiles.

Figure 13a shows the variation of coefficient A (expression 1) with initial effective pressure. It can be seen that the coefficient A decreases linearly with increasing initial effective pressure (R^2=0.97 and 0.96 for DS and MT). Figure 13b shows the evolution of coefficient C (expression 2) with initial confining pressure; as illustrated by Fig. 13b the coefficient C increases linearly with increasing effective pressure (R^2= 0.91 and 0.87 for DS and MT).

The variation of the ratio of the stress deviator (q_{ss}) difference versus number of geotextiles (Ng) is illustrated in Fig. 14a. As it can be seen, the normalised shear strength (q_{ss}) increases linearly with increasing number of geotextiles (R^2=0.98, 0.99 and 1 for σ'_c=50, 100 and 200 kPa, respectively), the increase of the normalised shear strength is very pronounced for samples shearing under effective confining pressure equals at 50 kPa, then it tends to decrease in scale for the effective stress equals at 100 and 200 kPa. We present in Fig. 14b the variation of the ratio of the stress deviator excess to the number of geotextile layers (Rq).

$$Rq_{ss}=(q_{ss(AP)}-q_{ss(MT)})/\sigma'_c \tag{3}$$

$$Rq=(q_{ss(AP)}-q_{ss(MT)})/Ng \tag{4}$$

$q_{ss(AP)}$ and $q_{ss(MT)}$ denote the value of the stress deviator of reinforced sand prepared with air pluviation and moist tamping respectively.

Table 2 Values of coefficients A and B

Initial confining pressure (σ'_c: kPa)	Method of preparation	Coefficient A	Coefficient B	R^2
50	Air pluviation (AP)	0.57	4.05	0.98
50	Moist tamping (MT)	0.49	3.07	0.99
100	Air pluviation (AP)	0.42	4.75	0.99
100	Moist tamping (MT)	0.36	4.40	0.99
200	Air pluviation (AP)	0.26	5.41	0.99
200	Moist tamping (MT)	0.23	5.19	0.99

Table 3 Values of coefficients C and D

Initial confining pressure (σ'_c: kPa)	Method of preparation	Coefficient C	Coefficient D	R^2
50	Air pluviation (AP)	-7.56	-2.32	0.99
50	Moist tamping (MT)	-3.53	-15.99	0.99
100	Air pluviation (AP)	-4.90	-11.96	0.99
100	Moist tamping (MT)	-1.77	-20.30	0.92
200	Air pluviation (AP)	-3.08	-20.04	0.94
200	Moist tamping (MT)	-0.84	-27.99	0.74

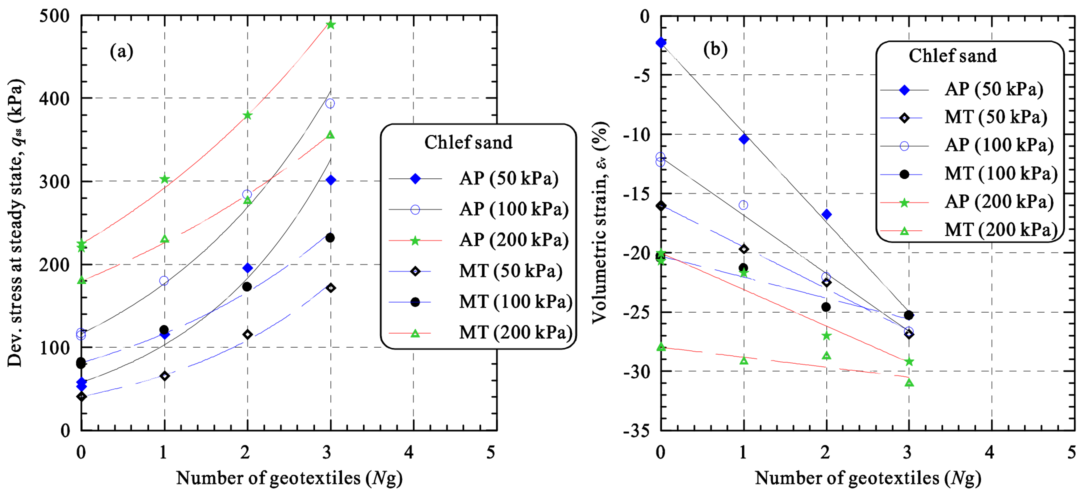

Figure 12. Effect of number geotextiles on deviator stress (a) and volumic strain (b).

Two zones can be distinguished: the first zone where the ratio Rq increases linearly until Ng=1, the second zone where the Rq decreases with increasing number of geotextiles (Fig. 13b).

Figure 15a shows the variation of normalised volumetric strain versus number of geotextiles. As it can be seen the normalised volumetric strain increases linearly with increasing number of geotextiles Ng significantly for initial confining pressure σ'_c=50 kPa (R^2=0.997, 0.99 and 0.83 for σ'_c=50, 100 and 200 kPa) according to the expression (5), the normalised volumetric strain slope line is very pronounced for 50 kPa, compared to 100 and 200 kPa.

Figure 15b illustrates the variation of Rv (ratio of volumic strain) versus number of geotextiles. We notice that the volume tric strain increases dramatically for Ng=1, then there is a tendency to lower volumetric strains when the number of geotextiles Ng from 2 to 3 for all initial confining pressure.

$$R_v=(\varepsilon_v(\%)_{(AP)}-\varepsilon_v(\%)_{(MT)})/Ng \quad (5)$$

$\varepsilon_v(\%)_{(AP)}$ and $\varepsilon_v(\%)_{(MT)}$ denote the value of the volumetric strain in percentage of reinforced sand prepared with air pluviation and moist tamping respectively.

Figure 16a illustrates the evolution of the mobilized friction (φ) angle versus number of geotextiles (Ng). We note that the sample prepared by air pluviation mobilizes friction angles higher than those prepared by the moist tamping method. The evolution of the friction angle depending versus number of geotextiles follow as linear progression (R^2=0.99, 0.97 and 0.99 for σ'_c=50, 100 and 200 kPa (AP), and R^2=0.99, 0.99 and 0.99 for σ'_c=50, 100 and 200 kPa (MT)). Figure 16b shows the variation of mobilised frictional angle (φ) versus initial confining pressure (σ'_c). It was found that the internal friction angle mobilized increases significantly with increasing number of geotextiles and decreases with increasing confinement logarithmically according to expression (5) (R^2=0.92 and 0.99 for Ng=0 and Ng=1, 2 and 3 (AP), R^2=1, 0.97, 1 and 0.99 for Ng=0 and Ng=1, 2 and 3 (MT) respectively). Our results are in good agreement with those found in the literature (Arab, 1998; Al-Mahmoud, 1997; Kolymbas and Wu, 1990; Fukushima and Tatsuoka, 1984).

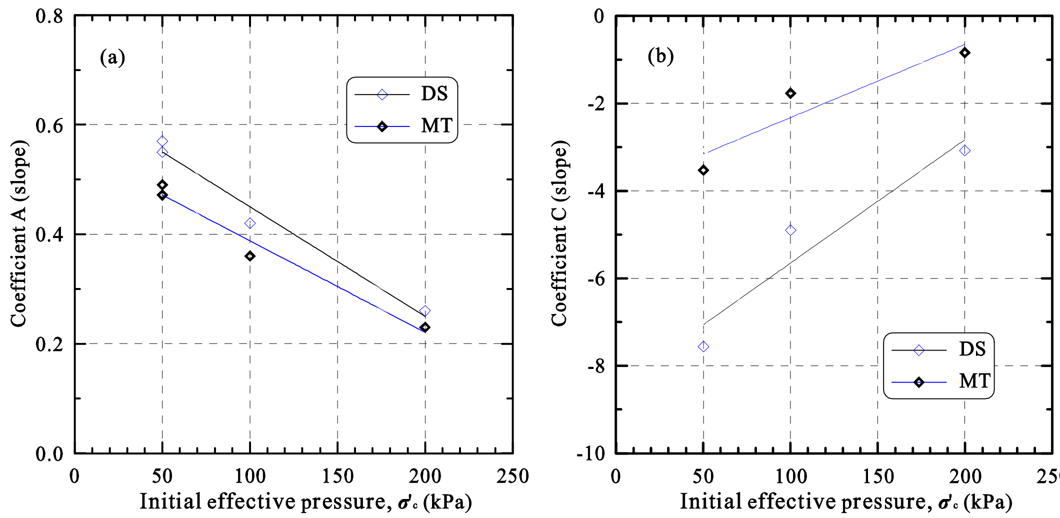

Figure 13. variation of coefficient A and C versus initial confining pressure.

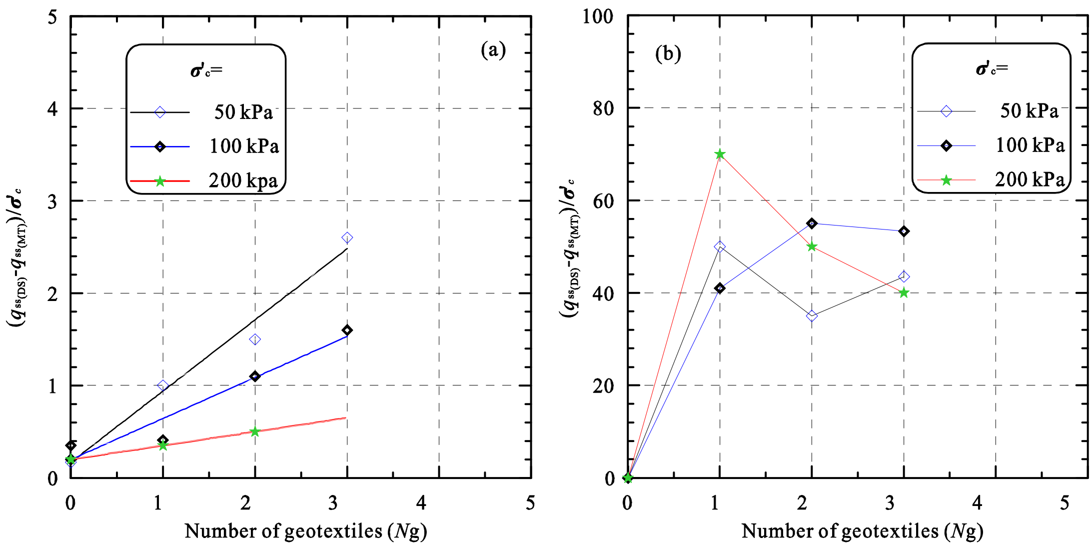

Figure 14. Variation of shear stress ratio versus number of geotextiles.

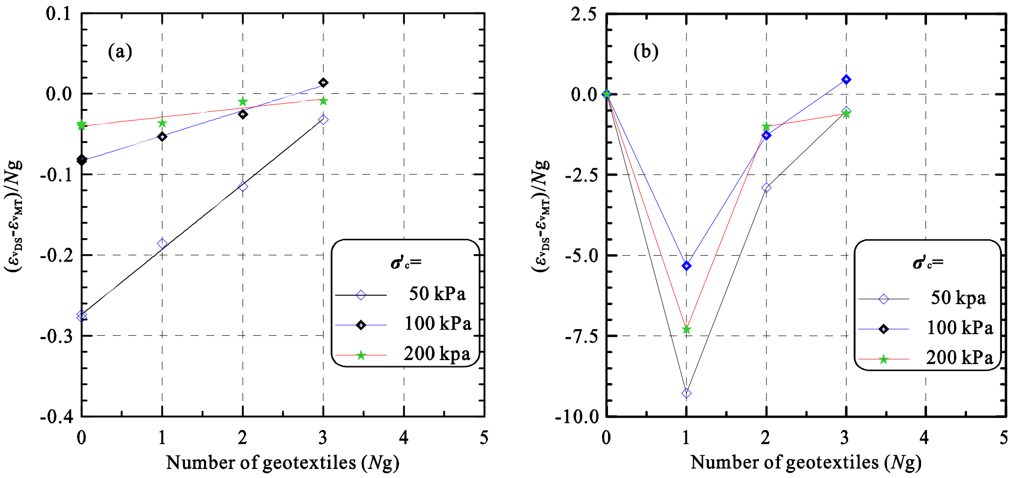

Figure 15. Variation of volumetric strain versus number of geotextiles.

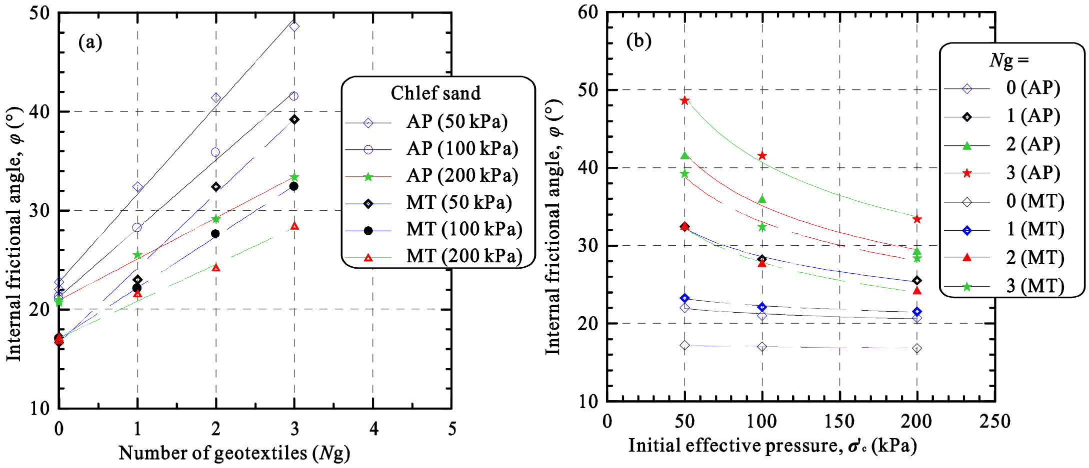

Figure 16. Evolution of internal friction angle mobilized. (a) Internal friction angle versus initial confining pressure, (b) internal friction angle versus number of geotextiles.

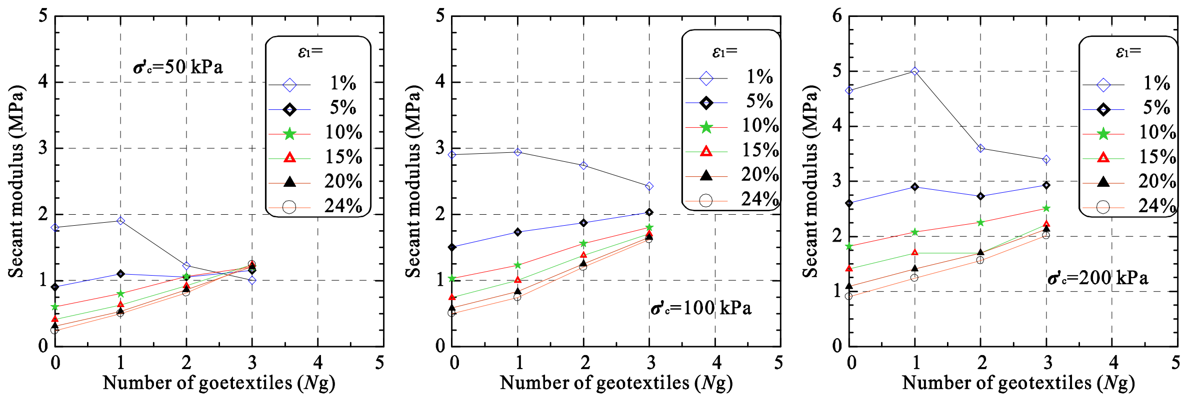

Figure 17. Variation of secant modulus versus number of geotextiles (samples prepared by AP).

Figures 17 and 18 illustrate the variation of the secant modulus according to number of geotextiles at different axial strain values for samples prepared by two methods of air pluviation and moist tampping. As it can be seen in those figures, method of preparation affects significantly the evolution of secant modulus. We notice that for samples prepared by air pluviation method the secant modulus determined at 1% and 5% decreases with increasing number of geotextiles in our opinion the contribution of geotextiles to soil resistance is negligible for the weak and becomes significant for large deformations. In contrast, samples prepared by the moist tamping method, the secant modulus at 1% and 5% deformation increases with increasing number of geotextiles and confining pressure, from 10% axial strain secant modulus

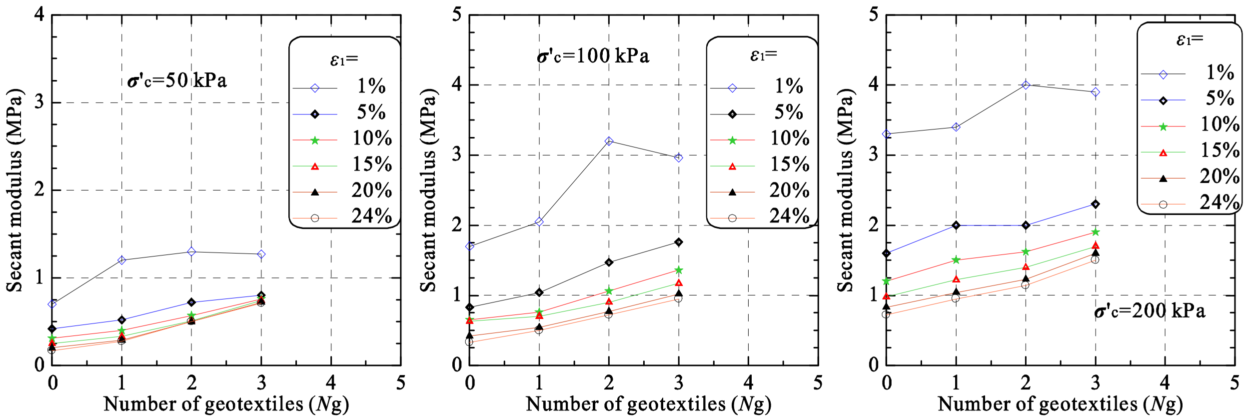

Figure 18. Variation of secant modulus versus number of geotextiles (samples prepared by MT).

increases with increase inclusions number of geotextiles. Samples prepared by air pluviation secant modulus were significantly higher than those prepared by the moist tamping.

5 CONCLUSIONS

This paper presents a series of drained triaxial tests on reinforced soil to evaluate the performance of geotextiles in improving the shear strength. Influences of geotextile layers, initial confining stress and preparation method on the static behaviour of reinforced soil are investigated. The main conclusions from the present study are summarized as follows.

(1) Results show that the shear strength increases with increasing number of geotextiles, for higher values of axial strain, geotextiles induces a quasi-linear in the mobilised stress deviator.

(2) Tests conducted on reinforced samples prepared with moist tamping present a lower strength than those prepared with dry pluviation method. These differences of behaviour can be explained by the fact that the molecules of water contained in the structures prepared by wet deposition method constitute some macropores easily compressible at the time of the shearing of the sample, and at the same time prevent the grain to grain and grain-geotextile adhesion.

(3) Increasing the number of geotextiles and confining pressure induces an increase of the sandy soil contractancy, and consequently the geotextiles reduce and restricts significantly the dilalation of reinforced sandy soil.

(4) Samples prepared with moist tamping method show contractive behaviour in comparison to those prepared by air pluviation.

(5) Results of tests show that the mobilised friction angle increases with increasing number of geotextile layers and decreases with increasing initial confining pressure. Samples prepared bymoist tamping, mobilize friction angles significantly lower than those prepared by air pluviation method.

(6) Normalised shear strength (Rq_{ss}) increases linearly with increasing number of geotextiles for the initial confining pressure under consideration. The normalised shear strength slope line for 50 kPa is very marked in comparison with 100 and 200 kPa.

(7) Secant modulus determined at 1% and 5% for samples prepared by air pluviation method, decreases with increasing number of geotextiles; in contrast, the secant modulus for samples prepared by the moist tamping method at 1% and 5% deformation increases with increasing number of geotextiles and confining pressure. Secant modulus beyond 10% increases with increase of inclusions for the two samples preparation methods. Secant modulus of samples prepared by air pluviation has significantly higher values than those prepared by moist tamping.

ACKNOWLEDGMENTS

Thanks go to the reviewers and the editors for their helpful advices. The final publication is available at Springer via http://dx.doi.org/10.1007/s12583-015-0621-0.

REFERENCES CITED

Al-Mahmoud, M., 1997. Etude en Laboratoire du Comportement des Sables Sous Faibles Contraintes: [Dissertation]. USTL Lille1, Villeneuve d'Ascq

Arab, A., 1998. Behaviour of Chlef, O/Rass and Hostun Rf Sands under Monotonic and Cylic Loading: [Dissertation]. Hassiba Ben Bouali University of Chlef, Chlef (in French with English Abstract)

Ashmawy, A. K., Bourdeau, P. L., 1998. Effect of Geotextile Reinforcement on the Stress-Strain and Volumetric Response of Sand. Proceedings of the 6th International Conference on Geosynthetics, Atlanta. 2: 1079–1082

Athanasopoulos, G. A., 1993. Effect of Particle Size on the Mechanical Behaviour of Sand-Geotextile Composites. *Geotextiles and Geomembranes*, 12(3): 255–273. doi:10.1016/0266-1144(93)90029-n

Broms, B. B., 1977. Triaxial Tests with Fabric-Reinforced Soil. Proceedings of International Conference on the Use of Fabric in Geotechnics, Ecole National des Ponts et Chaussees, Paris. 3: 129–134

Duncan, J. M., Dunlop, P., 1968. The Significance of Cap and Base Restraint. *Journal of Soil Mechanics and Foundation Division*, *ASCE*, 94(1): 271–290

Durville, J. L., Méneroud, J. P., 1982. Geomorphological Phenomena Induced by the El Asnam Earthquake, Algeria. *Bull*. *Liaison*. *Labo*. *Ponts*. *et Chaussées*, *Juillet-Août*, 120: 13–23 (in French with English Abstract)

Fukushima, S., Tatsuoka, F., 1984. Strength and Deformation Characteristics of Saturated Sand and Extremely Low Pressures. *Soils and Foundations*, 24: 11–54

Gray, D. H., Al-Refeai, T., 1986. Behavior of Fabric-Versus Fiber-Reinforced Sand. *Journal of Geotechnical Engineering*, 112(8): 804–820. doi:10.1061/(asce)0733-9410(1986)112:8(804)

Haeri, S. M., Noorzad, R., Oskoorouchi, A. M., 2000. Effect of Geotextile Reinforcement on the Mechanical Behavior of Sand. *Geotextiles and Geomembranes*, 18(6): 385–402. doi:10.1016/s0266-1144(00)00005-4

Houston, S. L., Perez-Garcia, N., Houston, W. N., 2008. Shear Strength and Shear-Induced Volume Change Behavior of Unsaturated Soils from a Triaxial Test Program. *Journal of Geotechnical and Geoenvironmental Engineering*, 134(11): 1619–1632. doi:10.1061/(asce)1090-0241(2008)134:11(1619)

Kolymbas, D., Wu, W., 1990. Recent Results of Triaxial Tests with Granular Materials. *Powder Technology*, 60(2): 99–119. doi:10.1016/0032-5910(90)80136-m

Krishnaswamy, N. R., Isaac, N. T., 1994. Liquefaction Potential of Reinforced Sand. *Geotextiles and Geomembranes*, 13(1): 23–41. doi:10.1016/0266-1144(94)90055-8

Ling, H. I., Tatsuoka, F., 1993. Laboratory Evaluation of a Nonwoven Geotextile for Reinforcing on Site Soil. *Proceeding of Geosynthetics*, 93(2): 533–546

McGowan, A., Andrawes, K. Z., Al-Hasani, M. M., 1978. Effect of Inclusion Properties on the Behaviour of Sand. *Géotechnique*, 28(3): 327–346. doi:10.1680/geot.1978.28.3.327

McKenzie, D., 1972. Active Tectonics of the Mediterranean Region. *Geophysical Journal International*, 30(2): 109–185. doi:10.1111/j.1365-246x.1972.tb02351.x

Papastamatiou, D., 1980. El-Asnam, Algeria Earthquake of October 10, 1980: Field Evidence of Ground Motion in the Epicentralregion. Geognosis, London

Rothé, J. P., 1955. Earthquake of Orléansville and Seismicity of Algeria. *Nature Journal*, *Paris*, 3237: 1–9 (in French with English Abstract)

Tang, C. S., Shi, B., Gao, W., et al., 2007. Strength and Mechanical Behavior of Short Polypropylene Fiber Reinforced and Cement Stabilized Clayey Soil. *Geotextiles and Geomembranes*, 25(3): 194–202. doi:10.1016/j.geotexmem.2006.11.002

Thevenin, J., 1955. The Effects of the Earthquake of September 1954 on Hydraulic Structures in the region of Orléanville. Revue Terre et Eaux, No 24, 1st Trimestre, Paris (in French with English Abstract)

Unnikrishnan, N., Rajagopal, K., Krishnaswamy, N. R., 2002. Behaviour of Reinforced Clay under Monotonic and Cyclic Loading. *Geotextiles and Geomembranes*, 20(2): 117–133. doi:10.1016/s0266-1144(02)00003-1

Wang, G. H., Sassa, K., Fukuoka, H., et al., 2007. Experimental Study on the Shearing Behavior of Saturated Silty Soils Based on Ring-Shear Tests. *Journal of Geotechnical and Geoenvironmental Engineering*, 133(3): 319–333. doi:10.1061/(asce)1090-0241(2007)133:3(319)

Madhavi Latha, G., Murthy, V. S., 2007. Effects of Reinforcement form on the Behavior of Geosynthetic Reinforced Sand. *Geotextiles and Geomembranes*, 25(1): 23–32