



Influence of sample preparation method on static liquefaction behaviour of geotextile reinforced sandy soil

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Abstract

This paper aims to evaluate the influence of soil deposition methods on the shear strength of reinforced sand using a triaxial test. Non-woven geotextile layers were used as reinforcement in the experiment to reconstitute specimens of natural sand prepared at loose relative density ($D_r = 30\%$). Arrangements of reinforcement layers (0, 1 and 2 layers) on soil samples were prepared using two different deposition methods [dry funnel pluviation (DFP) and wet deposition (WD)] and consolidated under three levels of confining pressures. Result shows that geotextile inclusion improves the mechanical behavior of sand; a significant increase in the shear strength is obtained by adding up layers of reinforcement. However, it reveals also that the soil-geotextile reinforcement interface efficiency is directly related to the number of layers and the soil dispositional method as well as their initial state. Since, the DFP method exhibits samples strain hardening compared to the WD samples which is very sensitive to static liquefaction. It is important to note that the difference in behavior of test on reinforced WD samples was attributed to the strain at failure, where unreinforced soil tested at ($p'_c = 50$ kPa) has attained complete liquefaction at small strain ($\epsilon_{fa} = 1.6\%$ for sample tested at 50 kPa of confining pressure) compared to reinforced samples tests in which failure envelope be found for more axial strain.

Highlights

- The effectiveness of geotextile reinforcement to remediate soil strength is potentially influenced by the soil pore pressure generation
- The effectiveness of geotextile reinforcement is greatly influenced by the deposition method as the WD method directly affects the soil-geotextile reinforcing mechanism.
- Soil wet samples have shown a contractive behavior which is very sensitive to static liquefaction
- The study reveals that the effect of soil deposition has less response with the increment of confining pressure

Keywords Sand-non woven geotextile · Consolidated undrained triaxial (CU) test · Deposition method · Confining pressure

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1 Introduction

The city of El Asnam (formerly Orléanville, today Chlef) in Algeria was severely damaged in 1954 by a major earthquake causing damages on each structure and foundation. Saturated sandy soil located largely on this region especially along Chlef river banks were subjected to severe liquefaction occurred during the last earthquake.

Many researchers have studied the problem of static liquefaction of saturated sand [1, 2]. Castro [3] found that sudden increases of pore water pressure, induced by monotonic shearing under undrained conditions, lead to a significant running down of effective stress up to lose the soil shear strength. The consequences attributed to the phenomena of liquefaction were severe. In this field, Belkhatir et al. [4] reported that a collapse of certain civil and hydraulic structures (earth dams, embankments, bridges, slopes and buildings) was caused on the last Chlef earthquake attributed to the liquefaction phenomenon and can be related by some factors conducting to the soil strength failure. Mahmoudi et al. [5] have reported that the initial effective confining pressure is an important influencing factor on the mechanical characteristics, undrained shear strength, stiffness and deformation characteristics of granular materials. In addition, soil preparation has been increasingly studied in the literature, Della et al. [6] have studied the effect of deposition mode on the instability zone of sand and found that test on the WD deposited sample has larger slope of instability than that of the DFP method. Benahmed et al. [7] in their presented results showing that the samples prepared by dry funnel pluviation (DFP) are more resistant than those prepared by wet deposition. Vaid et al. [8] have shown the same outcome about the effect of initial water in growing up the soil pore pressure and which is expressed by the phenomenon of static liquefaction.

In the other hand, available studies in the literature have explored the possibility to improve the soil liquefaction resistance considering the most effective process [9]. As the soil reinforcement method, adding reinforcing material to the soil has recently become a focus of intense interest [10–12]. A number of experimental tests, including triaxial and direct shear tests have been conducted on this subject by several investigators and have shown an ability to remediate the soil strength by adding reinforcement. Krishnaswamy and Isaac [13] suggested that the addition of geosynthetics to problematic soils reduces its liquefaction potential by increasing the soil effective confining pressure. Benessalah et al. [14] introduced the safety coefficient S to estimate the soil resistance to liquefaction. It was reported that for reinforced samples, the safety limit represents an addition

between the initial confining pressure and a confining pressure created by the geosynthetic layer.

The effectiveness of reinforcement is potentially influenced by the reinforcement properties and the soil characteristics. Gray and Ohashi [15] found through a series of direct shear tests conducted on reinforced dry sand with different types of fibers that natural and synthetic fiber reinforcement exhibits a better performance than metal fibers. Furthermore, Naeini and Gholampoor [16] have reported on their study that liquefaction resistance of sandy soil increases in a manner with the increase of geotextile layers.

The procedure of soil preparation or the mode of soil deposition influences the mechanical behavior of reinforced soils. Benessalah et al. [10] have conducted a series of direct shear tests to evaluate the influence of glass fiber on the shear strength of sand prepared in dry and wet conditions. Test results have indicated that the mechanical characteristics are enhanced with the addition of glass-fibres, especially for wet specimens. Denine et al. [17] in their study of direct shear tests conducted on Chlef sand have showed that the deposition mode and the geotextile reinforcement have significant effects on the behavior of reinforced sand. The increase in shear strength is relatively important for specimens prepared with dry deposition mode. Brahim et al. [18] have conducted a drained triaxial tests in order to study the influence of deposition method on sand reinforced with geotextile layers and found that the wet samples exhibit a dilative behavior when compared to the dry samples.

At present, the influence of deposition method on the undrained monotonic response of reinforced loose sandy soil has received less attention. In this context, the present experimental study aims to evaluate the effect of sample deposition method on the mechanical behavior of reinforced sand. A series of undrained triaxial monotonic compression tests (CU) were performed on unreinforced and reinforced samples with one and two layers of reinforcement taken into account the effect of deposition method of loose prepared samples ($D_r = 30\%$). In the next sections, the experimental program and the laboratory results of triaxial tests on natural sand sample with and without reinforcement are presented and discussed. The study will permit the investigation of the effects of the soil deposition method and confining pressure on the mechanical behavior of geotextile reinforced sandy soil.

2 Materials and methods

2.1 Apparatus

The triaxial testing apparatus used in this study is shown in Fig. 1. All consolidated undrained tests were carried

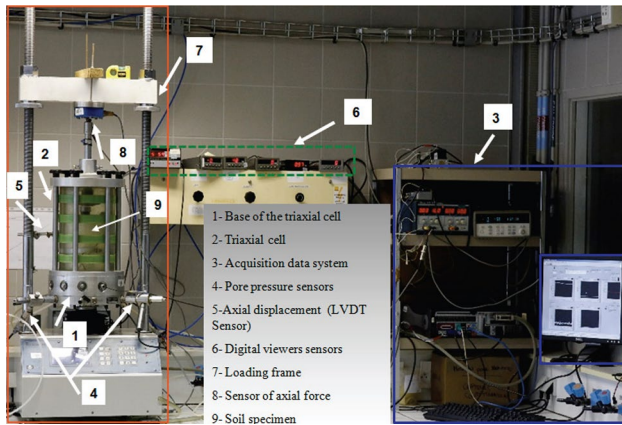


Fig. 1 General view of the triaxial apparatus [19]

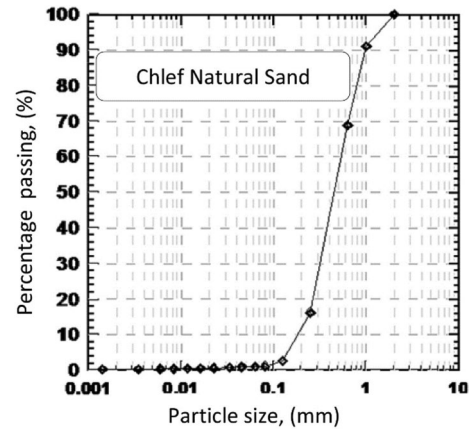


Fig. 3 Grain size distribution curve of Chlef sand

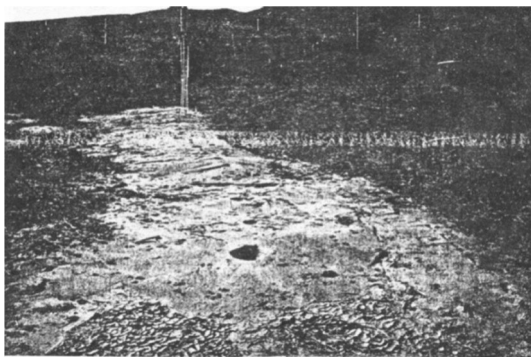


Fig. 2 Craters of liquefied soil on the banks of the Chlef river [21]

out on cylindrical samples in a diameter of 100 mm and a height of 200 mm.

2.2 Materials description

2.2.1 Sand

In this study, Chlef sand was used to reconstitute samples, the natural soil with a small amount of silt (Less than 5%) was extracted from a saturated area that it has encountered by instances of visible sand boils and the formation of a large tremor lake attributed to the phenomenon of soil liquefaction during the last seismic of October 1980 [20] (Fig. 2). The studied soil is classified as poorly graded sand (SP) in the particles form is rounded. Figure 3 presents the grain size distribution of the used sand. All physical characteristics of the soil sand are illustrated in Table 1.

Table 1 Physical characteristics of Chlef natural sand

Sand characteristics	Value
Effective grain size, D_{10} (mm)	0.182
Mean grain size, D_{50} (mm)	0.452
Uniformity coefficient, C_u (-)	2.962
Coefficient of curvature, C_c (-)	1.033
Specific density, G_s (g/cm^3)	2.70
Maximum void ratio, e_{max} (-)	0.98
Minimum void ratio, e_{min} (-)	0.58
Silt plasticity index, $I_{p(s)}$ (%)	5.5%

2.2.2 Reinforcement

Polypropylene based non-woven (NW) geotextile (S41) in the form of planar layers was tested in this work. The geotextile was cut into circular layers of 100 mm in diameter and disposed at different heights of specimen as displayed in Fig. 4, where unreinforced sand samples were studied as a base of reference in order to investigate the influence of reinforcement, initial state and disposition sample method on the undrained behavior of sand. The physical and mechanical characteristics of the geotextile acquired from the manufacturing organization (TenCate Geosynthetics, France) are resumed in Table 2.

2.3 Experimental procedure

Laboratory triaxial tests [22] on reconstituted samples of unreinforced and reinforced sand with non-woven geotextile were prepared using two different preparation methods namely, DFP method and wet deposition method (DFP and WD respectively). These preparation methods were comprehensively detailed by Benhamed et al. [7] and Della et al. [6]. In these experimental tests,

Fig. 4 Geotextile layers arrangement for silty sand samples

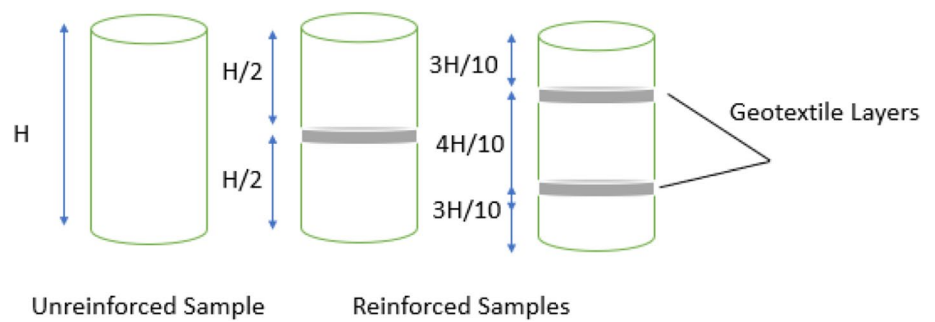


Table 2 Geotextile physical and mechanical characteristics

Non-woven geotextile	Value
Maximum tensile strength T_{max} (kN/m)	12–14
Permeability coefficient, K (m/s)	0.09–0.10
Punching resistance CBR (kN)	1.75
Surface weight, W_s (g/m ²)	155
Nominal thickness (mm)	1.60 (\pm 0.10)
Strain tensile strength, ϵ (%)	70–90

the wet samples were mixed by adding to soil an initial water content of ($w = 3\%$). All specimens were prepared by first estimating the dry weight of specimen corresponding to the loose density ($Dr = 30\%$), then deposited into the mold. For the reinforced specimen, after cutting the geotextile into circular forms, reinforcement inclusions are placed horizontally on the surface as each layer of sand is formed, considering both the relative density of sand and the different arrangements of reinforcement layers as shown in Fig. 4. Table 3 summarizes the experimental program of consolidated undrained (CU) triaxial tests with illustrations of the various parameters.

3 Results discussion and analysis

The experimental program is a series of consolidated undrained triaxial tests performed on unreinforced and reinforced sand samples with different layers of non-woven geotextile reinforcement ($L = 0, 1$ and 2). Samples were prepared with the two different preparation methods (DFP and WD) at the loose state of $Dr = 30\%$ and under three confining pressures of 50, 100 and 200 kPa. Laboratory

results will be discussed in the following sections regarding the effect of geotextile reinforcement, confining pressure and mode of deposition on static liquefaction behavior of Chlef sandy soil. Table 4 shows the summary results of realized tests.

3.1 Unreinforced sand

Figures 5, 6 and 7 show curves of the stress–strain, variation of excess pore water pressure and the stress paths of unreinforced sand specimen prepared with both deposition methods (DFP and WD) under three different confining pressures respectively.

From Figs. 5a, 6a and 7a, it is indicated that specimens prepared with DFP method exhibited more strength than those prepared with the WD method. The maximum values of deviatoric stress at confining pressures of 50, 100 and 200 kPa were respectively 114.71, 147.86 and 275.01 kPa for sand samples prepared with the DFP method and 15.20, 34.51 and 77.04 kPa for those prepared with the WD method the same finding was reported by Della et al. [6].

The variation of pore water pressure of unreinforced specimens found to develop firstly up to a maximum value then followed by a decrease phase particularly for dry samples and for wet sample tested under 200 kPa of confining pressure (Figs. 5b, 6b and 7b). However, it was shown a state of static liquefaction for wet samples tested under 50 and 100 kPa of confining pressure. Further, increasing of confining pressure reduces for both deposition methods (DFP and WD) the potential of static liquefaction especially for wet samples. The excess pore water pressure progressively increased for sand samples prepared with WD method until reached the initial confining pressure ($p'_c = 50$ and 100 kPa) and the collapse of soil

Table 3 Experimental program of consolidated undrained (CU) tests

Sample arrangement	Sample designation	Relative density, Dr (%)	Confining pressure, p'_c (kPa)	Water content, w (%)
Unreinforced	UR	30%	50, 100 and 200	0 and 3
One layer of reinforcement	RL1 ($H = 6$ cm)			
Two layers of reinforcement	RL2 ($H = 6; 14$ cm)			

Table 4 Results of undrained triaxial compression tests

Test samples	Confining pressure p'_c (kPa)	Maximum and peak deviator stress q_{max} (kPa)		Axial strain of failure ϵ_{af} (%)		Peak pore pressure Δu_{max} (kPa)		Soil behavior	
		DFP (w=0%)	WD (w=3%)	DFP (w=0%)	WD (w=3%)	DFP (w=0%)	WD (w=3%)	DFP (w=0%)	WD (w=3%)
UR	50	114.71	15.20	–	1.6	37.80	50	Steady	Liquefied
	100	147.86	34.51	–	3.75	77.93	100	–	–
	200	275.01	77.04	–	–	153.78	181.32	–	Quasi-steady
RL1	50	129.91	15.50	–	3.56	33.10	50	Steady	Liquefied
	100	187.47	35.40	–	9	74.02	100	–	–
	200	280.43	78.90	–	–	144.84	180.09	–	Steady
RL2	50	226.03	19.50	–	3.40	24.22	50	Steady	Liquefied
	100	286.47	37.01	–	–	79.19	97.69	–	Quasi-steady
	200	341.73	80.84	–	–	169.78	180.16	–	Steady

DFP dry funnel pluviation method, WD wet deposition, w initial water content

Bold indicates the text is used in order to indicate the state of soil instability

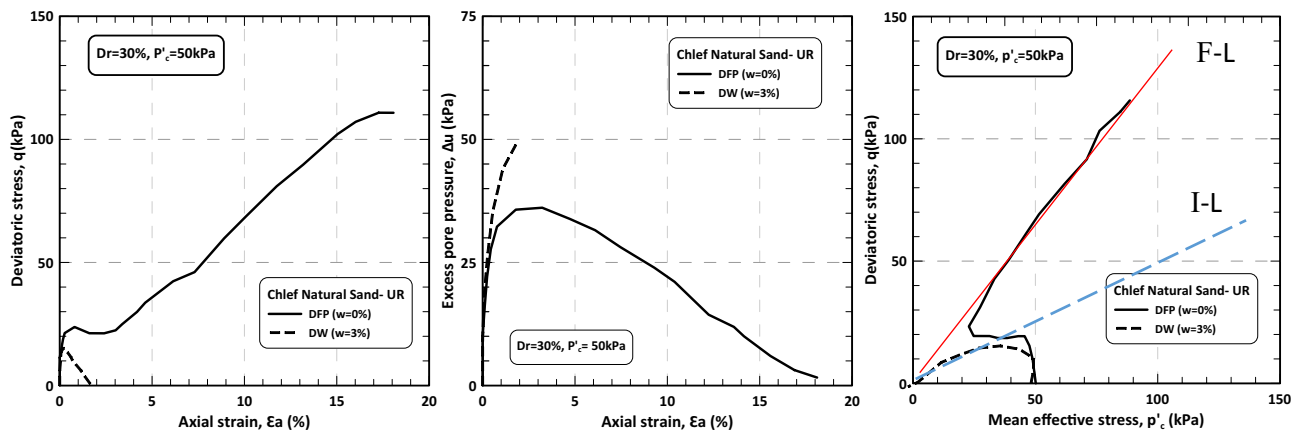


Fig. 5 Effect of deposition method on the undrained response of unreinforced sand samples at $p'_c = 50$ kPa of confining pressure: **a** deviator stress versus axial strain, **b** excess pore pressure against axial strain, **c** effective stress path

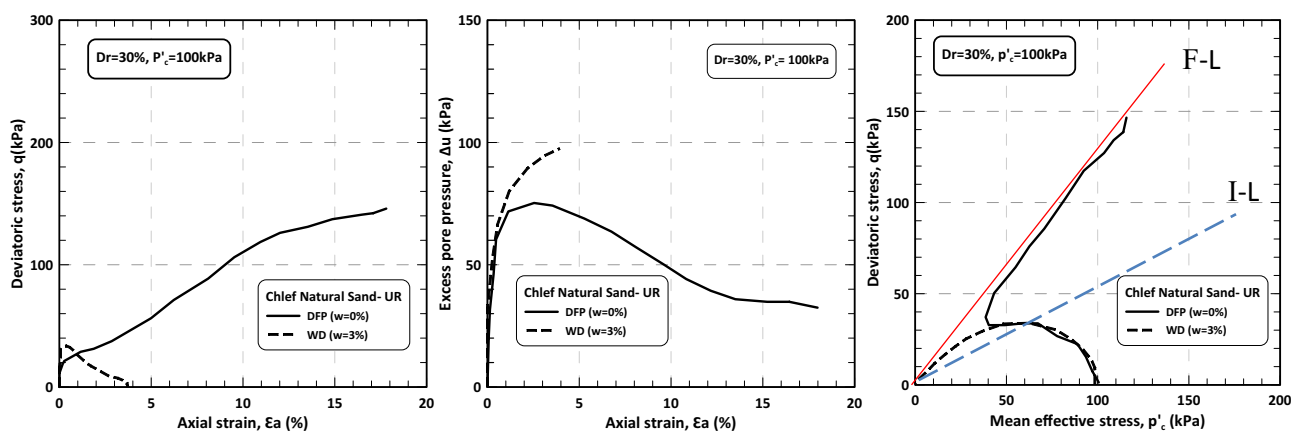


Fig. 6 Effect of deposition method on the undrained response of unreinforced sand samples at $p'_c = 100$ kPa of confining pressure: **a** deviator stress versus axial strain, **b** excess pore pressure against axial strain, **c** effective stress path

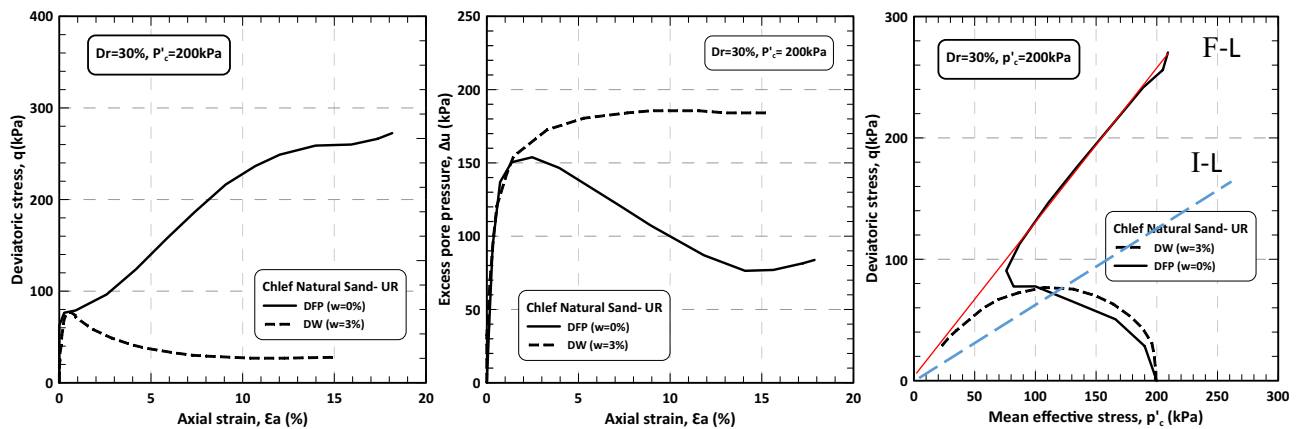


Fig. 7 Effect of deposition method on the undrained response of unreinforced sand samples at $p'_c = 200$ kPa of confining pressure: **a** deviator stress versus axial strain, **b** excess pore pressure against axial strain, **c** effective stress path

sample. From curves of the effective stress paths in the (p', q) plane in Figs. 5c, 6c and 7c, unreinforced samples prepared with the WD method show important contractive behavior, which reached total liquefaction (Table 4). This could be because the soil structure developed in this method has been comparatively looser than that of the DFP method due to the higher pore pressure developed on soil particularly tested at low level of confining pressures. In the other hand, test on (DFP) deposited sample has shown dilative behavior with a significant dissipation of excess pore pressure.

Instability lines (IL) and failure lines (FL) were determined based on the undrained tests of unreinforced and reinforced sand samples prepared with two modes of deposition (WD and DFP) at 30% initial relative density under different confining pressures (50 kPa, 100 kPa, and 200 kPa). From Figs. 5c, 6c and 7c, it can be seen that the slopes of the instability lines are increased with the increase of the confining pressure for both WD method and DFP method. This means that the higher the confining pressure, the higher the peak undrained shear stress the sample can reach. Also, it can be observed that the instability zones for the samples prepared by the WD method are larger than those prepared by the DFP method. These results are in good agreement with those found by Della et al. [6].

3.2 Reinforced sand

3.2.1 Variation of the deviator stress

Figures 8 and 9 present the deviator stress response of tests on (DFP) and (WD) deposited sand and reinforced with one and two layers of geotextile respectively. As seen in these figures, the deviator stress shows a sharp increase to maximum value for samples prepared with (DFP)

method. Compared with the samples prepared by (WD) method, wet sample clearly shows two different trends regarding the confining pressure. Consolidated reinforced samples under 50 kPa and 100 kPa have shown a behavior of total liquefaction than consolidated reinforced samples under 200 kPa which attained its final phase of steady state. This can be attributed to the role of confining pressure to improve the soil shear strength. Denine et al. [19] reported in their study on reinforced sand prepared by (DFP) method that the effectiveness of geotextile is more visible under low level of confining pressure.

The figures also show that the maximum deviator stress increases for sample prepared by (DFP) method and by adding more layers of reinforcement. It is noted that geotextile reinforced sand specimens showed more strength with two layers of reinforcement compared with one layer, the maximum values of deviator stress for wet samples tested at 100 kPa of confining pressure were 187.47 kPa and 286.47 kPa for reinforced sand samples with one layer (RL1) and two layers (RL2), respectively. Regarding sand samples prepared with WD method, the maximum deviatoric stress values obtained at 100 kPa of confining pressure were 35.40 kPa and 37.01 kPa for geotextile reinforced sand samples with one and two layers respectively.

As can be seen in particular, the effectiveness of geotextile reinforcement is less important for samples prepared with the WD deposition method compared with those prepared with the DFP deposition method. Where, the strength ratio values defined by Zhang et al. [20] as the ratio between the maximum shear strength of reinforced and unreinforced sand samples ranged from 1.26 to 1.03 for reinforced samples with one layer of geotextile and ranged from 1.83 to 1.1 for reinforced samples with two layers prepared with the DFP and WD methods at $p'_c = 100$ kPa of confining pressure, respectively. This is due to the presence of initial water content, which

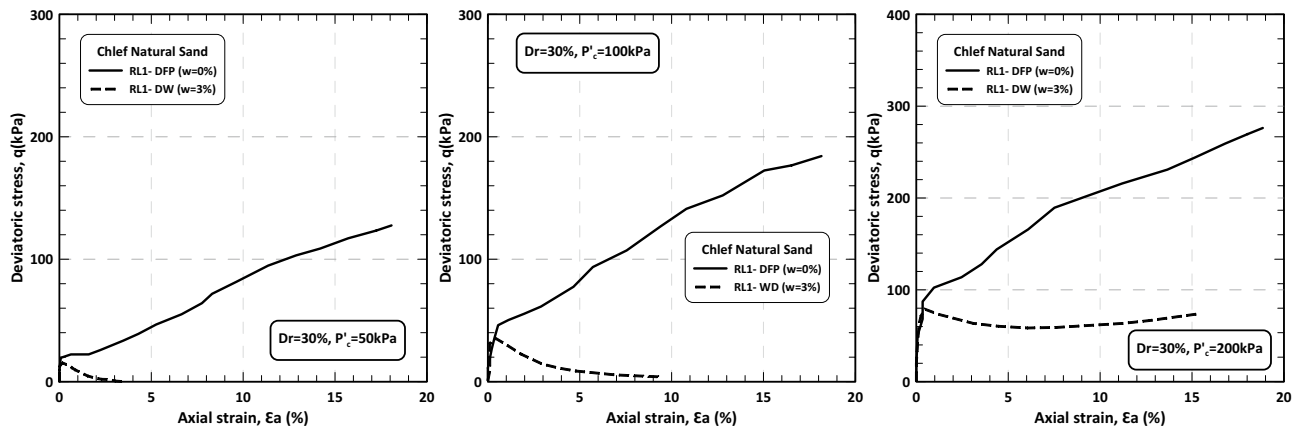


Fig. 8 Deviator stress versus axial strain curves of sand samples reinforced with one layer of geotextile under different confining pressures: **a** $p'_c = 50$ kPa, **b** $p'_c = 100$ kPa, and **c** $p'_c = 200$ kPa

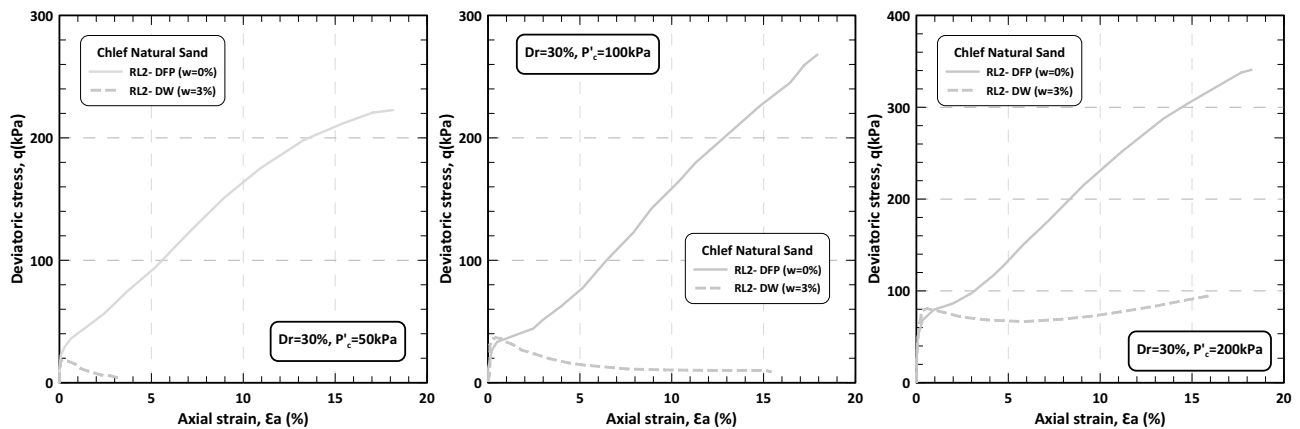


Fig. 9 Deviator stress versus axial strain curves of sand samples reinforced with two layers of geotextile under different confining pressures: **a** $p'_c = 50$ kPa, **b** $p'_c = 100$ kPa, and **c** $p'_c = 200$ kPa

influences the soil-geotextile interface interaction. Since the mechanism of composed soil material can be related to the surface roughness of the geotextile. The peak and residual interface strength increase through the use of geotextile regarding size aperture to ensure more friction with soil particles restricted to water infiltration which ensure more soil void ratio.

Regarding the results illustrated in Table 4, it is important to note that the difference in behavior of test on reinforced wet deposited samples (WD) was attributed to the strain at failure. Where unreinforced soil tested at ($p'_c = 50$ kPa) has attained complete liquefaction at small strain ($\epsilon_{fa} = 1.6\%$ for sample tested at 50 kPa of confining pressure) compared to reinforced samples tests in which failure envelope was found for more axial strain.

3.2.2 Variation of the excess pore pressure

Figures 10 and 11 present the variation of excess pore pressure vs axial strain of geotextile reinforced sand specimens prepared with two different deposition (DFP and WD) methods. A similar trend is observed regarding the pore pressure development and dissipation for all samples prepared by the DFP method. At first, a rapid increase of pore pressure is observed at small stress strains ranging from 0 to 3%, and then gradually decreased until it reached a steady state accompanying further shearing after which an increasing soil dilative behavior. It was shown also that the excess pore pressure increased relatively with the addition of reinforcement layers especially for higher level of confining pressure.

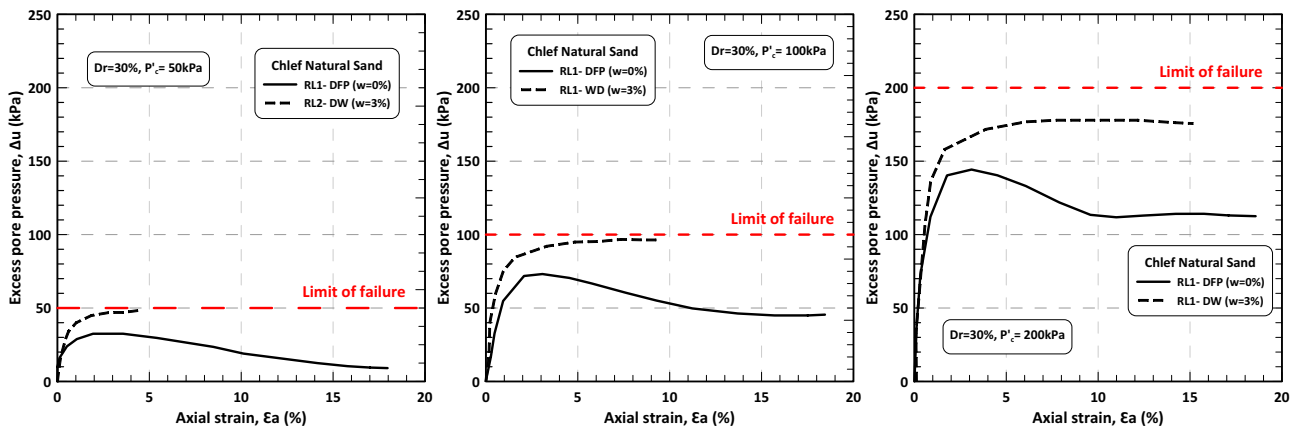


Fig. 10 Excess pore water pressure versus axial strain curves of sand reinforced with one layer of geotextile under different confining pressures: **a** $p'_c = 50$ kPa, **b** $p'_c = 100$ kPa, and **c** $p'_c = 200$ kPa

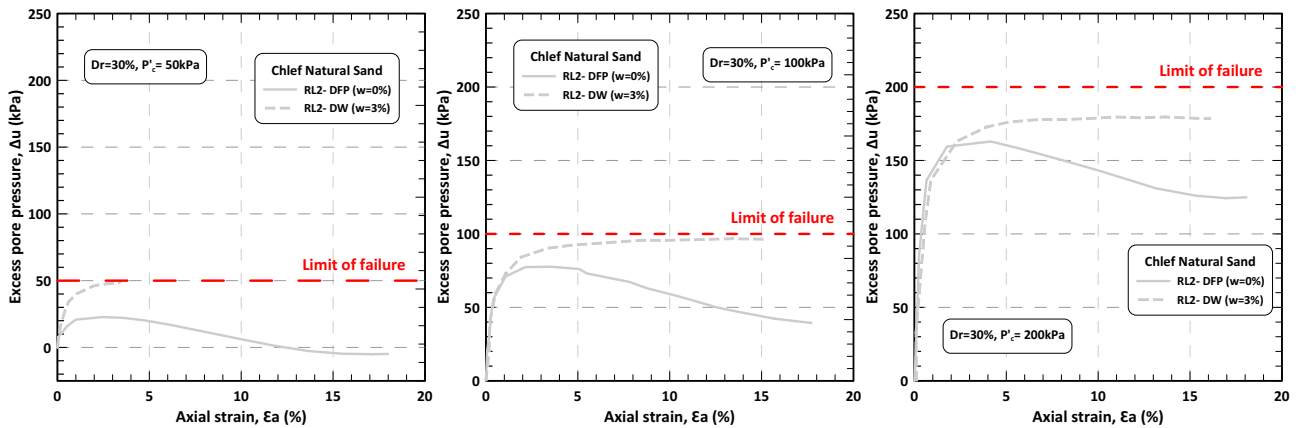


Fig. 11 Excess pore water pressure versus axial strain curves of sand reinforced with two layers of geotextile under different confining pressures: **a** $p'_c = 50$ kPa, **b** $p'_c = 100$ kPa, and **c** $p'_c = 200$ kPa

In the other hand, we observe that sample prepared by the WD deposition method initiates the liquefaction phenomena. Thus, the obtained peak excess pore pressure of reinforced (WD) deposited samples is relatively less compared to unreinforced (WD) deposited samples. Focusing on the influence of confining pressure, the result has been confirmed for test on (WD) deposited samples, reinforced samples show slower progression of excess pore pressure and more axial strain at failure than unreinforced sample. Moreover, it is clearly indicated that (WD) deposited samples tested at 200 kPa of confining pressure attain a maximum value of excess pore pressure close to the obtained value of unreinforced sample and for both methods of deposition (DFP and WD). This is in a good agreement with the finding of Benessalah et al. [23], thus they reported that the effectiveness of the geotextile inclusions to the improvement of the shear strength decreases when increasing the confining pressure for medium dense sand samples.

3.2.3 Stress path

The plots of typical undrained effective stress paths are presented in Figs. 12 and 13 respectively for one-layer reinforced sand (RL1) and two layers reinforced samples (RL2). It is noticeable that test on (WD) deposited sand sample has shown a similar tendency, the slopes of instability zone are quite approximately to a same value for reinforced samples. Conversely, it is observed from the figures that samples prepared by the DFP method are denser, which show dilative behavior and attain maximum values of stress ratio. In general, these results confirm the trend obtained by Della et al. [6] regarding the influence of confining pressure on the increase of liquefaction resistance.

Also, the influence of the geotextile is clearly visible on the stress path variation. The addition of reinforcement layers increases the average effective stress and the maximum stress deviator, the effect of reinforcement showed stable behavior especially for samples sheared under confining

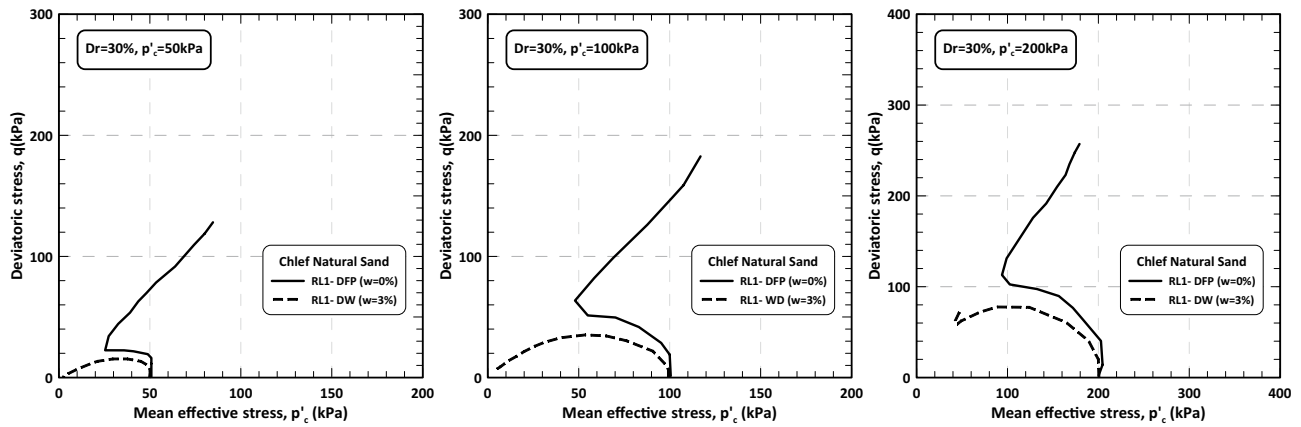


Fig. 12 Effective stress path curves of sand unreinforced and reinforced with one layer of geotextile under different confining pressures: **a** $p'_c = 50$ kPa, **b** $p'_c = 100$ kPa, and **c** $p'_c = 200$ kPa

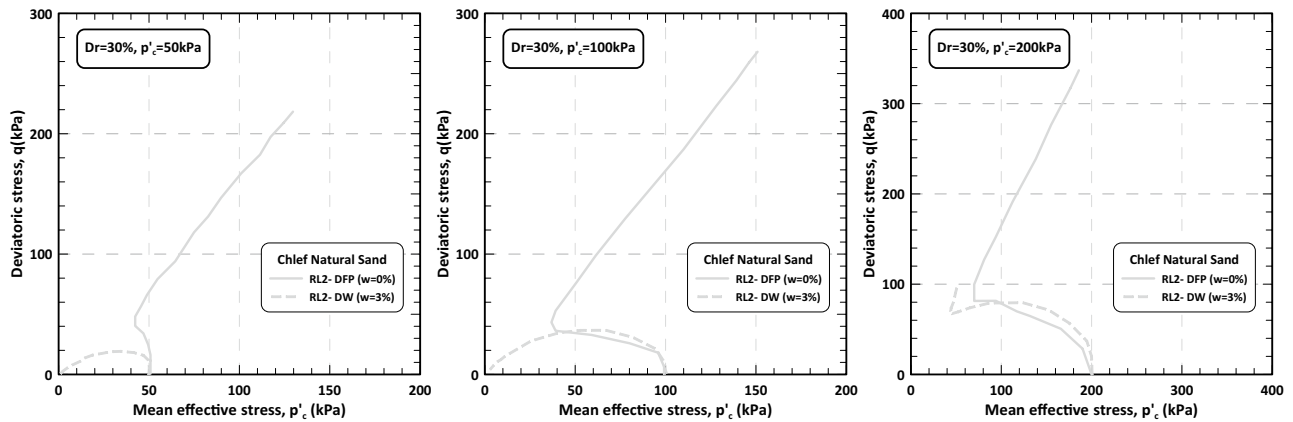


Fig. 13 Effective stress path curves of sand unreinforced and reinforced with two layers of geotextile under different confining pressures: **a** $p'_c = 50$ kPa, **b** $p'_c = 100$ kPa, and **c** $p'_c = 200$ kPa

pressure of 200 kPa, Where an accelerated instability has been hown on samples prepared by (WD) deposition method and tested at 50 and 100 kPa of confining pressure which exhibited a very weak resistance and even provokes the phenomena of liquefaction of sand and leading to the collapse of unreinforced and reinforced sand samples.

4 Maximum shear strength instability line friction angle

From Table 4, it was shown that adding reinforcement enhances the soil shear strength except increasing confining pressure has a significant effect on the effectiveness of reinforcement. Figure 14 presents the influence of the level of confining pressure on the maximum deviator stress including the effect of deposition method the deviator stress ratio is given by:

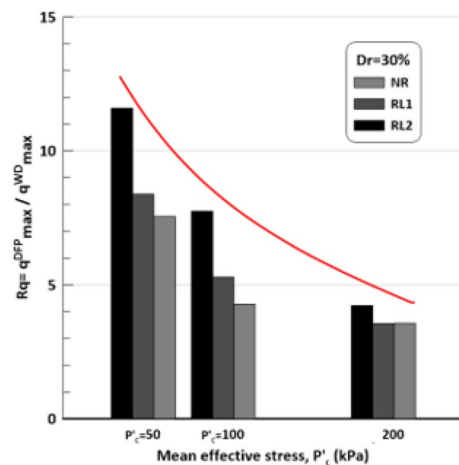


Fig. 14 Deviator stress ratio (R_q) of unreinforced and reinforced sand

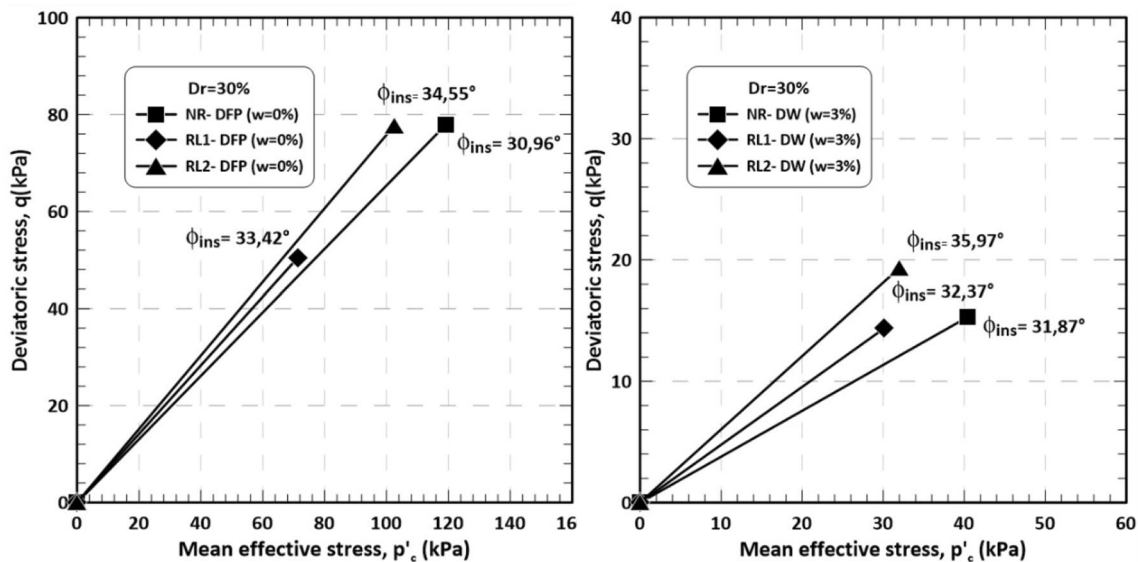


Fig. 15 Instability lines of unreinforced and reinforced sand prepared by DFP and WD methods

$$Rq = q_{max}^{DFP} / q_{max}^{WD} \tag{1}$$

where q_{max}^{DFP} and q_{max}^{WD} represent the maximum values of deviator stress for samples prepared by DFP and WD methods, respectively.

From Fig. 14, it can be seen that the ratio of deviator stress tends to decrease by increasing confining pressure until the calculated parameter reaches a constant value (of approximately $Rq \approx 4$) for samples of unreinforced (UR), one-layer reinforced sand (RL1) and two layers reinforced sand (RL2). Even from the based results, it is also concluded that the mechanical behavior of samples has shown that the effect of sample preparation is more visible for low level confining pressure. This is due to the growing up of the confining which required significant degree of the compaction particularly for WD prepared samples.

The friction angle of instability is plotted in Fig. 15. Results indicate that the effect of deposition method has not been apparent on the soil instability line (IL). However, it was found that the friction angle of instability increases by adding more layers of reinforcement where it ranged respectively from 30.96° to 34.55° for DFP samples and from 31.87° to 35.97° for WD samples for unreinforced and two layers reinforced sand.

5 Conclusions

The study of soil deposition method influence on the mechanical behavior of reinforced sand was carried out by a series of undrained consolidated triaxial tests.

Considering the different levels of confining pressure, tests on (WD) deposited samples have shown a behavior of complete liquefaction, however increasing confining pressure or adding reinforcement layers change soil to limited liquefaction or steady behavior. Also, results show that the DFP method exhibits samples strain hardening compared to the wet deposited samples (WD) which is very sensitive to static liquefaction.

In the other hand, adding reinforcement enhances the soil shear strength except increasing confining pressure has a significant effect on the effectiveness of reinforcement pressure. Focusing on the influence of confining pressure, the result has been confirmed for test on (WD) deposited samples, reinforced samples show slower progression of excess pore pressure and more axial strain at failure than unreinforced sample. Moreover, it is clearly indicated that (WD) deposited samples tested at 200 kPa of confining pressure attain a maximum value of excess pore pressure close to the obtained value of unreinforced sample and for both methods of deposition (DFP and WD). Results indicate that the effect of deposition method has not been noticeably apparent when increasing confining pressure on the soil instability line (IL). However, it was found that the friction angle of instability increases by adding more layers of reinforcement.

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Declarations

Conflict of interest No potential conflict of interest was reported by the author(s).

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