Liquefaction Resistance of Chlef River Silty Sand: Effect of Low Plastic Fines and other Parameters

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Abstract: Silty sands are the most common type of soil that could be involved in both static and earthquake-induced liquefaction. Most of the recent earthquakes have revealed the liquefaction of silty sands. Therefore, the selection of the appropriate undrained residual shear strength of liquefied soils to be used in the assessment of the post-liquefaction stability of earth dams and other earth structures is becoming a major challenge. A series of undrained monotonic and cyclic triaxial tests were carried out on reconstituted saturated samples of sand with variation in the fines content ranging from 0 to 50% for the monotonic tests and from 0 to 40% for the cyclic ones, in order to study the influence of fines fraction and other parameters on the undrained residual shear strength and liquefaction potential of loose, medium dense and dense silty sand samples (Dr = 12%, 50%, 60% and 90%). The results of the monotonic tests show that the stress-strain response and shear strength behaviour is controlled by the percentage of fines fraction and the samples become contractive for the studied relative densities (Dr = 12% and 90%). The

undrained residual shear strength decreases as the global void ratio decreases and the fines content increases up to 30% fines content. Beyond that, it decreases with increasing the global void ratio and the fines content. Moreover, the undrained residual strength decreases linearly as the fines content and the inter-granular void ratio increase. Cyclic test results show that the increase of the fines fraction accelerates the liquefaction phenomenon for the studied amplitude and the liquefaction resistance decreases with the increase of the global void ratio and the loading amplitude. We notice that the reduction in the liquefaction resistance of Chlef sand-silt mixtures becomes very marked for the smaller cyclic stress ratios CSR = 0.15 and 0.25.

Keywords: silty sand; residual strength; fines content; undrained tests; liquefaction

1   Introduction

On October 10, 1980 at 13:25:23.7 local time (12:25:23.7 GMT) a destructive earthquake took place near Chlef City, Algeria (formerly known as El-Asnam). Chlef is approximately 200 km west of Algiers. The Richter magnitude, M, of this event was 7.2, which corresponds to a surface wave magnitude, M_s, of 7.3. The epicenter of the quake was located at 36.143° N and 1.413° E, 10 km east of Chlef. The focal depth of the earthquake was about 10 km, and the approximate duration was between 35 and 40 sec. The earthquake devastated the city of Chlef, population estimated at 125,000, and the nearby towns and villages. The large loss of life (reportedly 5,000 to 20,000 casualties) and property was attributed to the collapse of buildings. In several places of the affected area, especially along the Chlef river banks, great masses of sandy soils were ejected on to the ground surface level. Great damage to different structures was recorded.

During earthquakes, the ground shakes, causing cohesionless soils to lose their strength and behave like a liquid. This phenomenon is called soil liquefaction and will cause the settlement of buildings, landslides, the failures of earth dams, or other hazards. Liquefaction occurs due to an increase in the excess pore water pressure and a corresponding decrease in the effective overburden stress in a soil deposit. The understanding of the liquefaction phenomena has significantly improved in recent years. Most liquefaction research was carried out on clean sands with the assumption that the behaviour of silty sand is similar to that of clean sands. Recent researches made by [1; 2, 3, 4, 5, 6, 7, 8] indicate that sand deposited with silt content is much more liquefiable than clean sand. Also, strain properties and pore pressure generation in silty sand samples are quite different from clean sand. These new findings emphasize the importance of deposits with mixture of sand and silt. Moreover, the behaviour of silty sand soils such as hydraulic fills is not clearly known during earthquake. Therefore, a deep understanding of silty sand behaviour is needed for liquefaction assessment of silty sandy soils.

Further, the post-earthquake behaviour of silty sand and, consequently, the stability of structures founded on liquefied soil depends on the post-liquefaction shear strength of soil. The strength of soils mobilized at the quasi-steady state has an important implication for engineering practice [9]. Laboratory studies, especially in the last ten years, have contributed much to clarify the parameters that control the residual shear strength and provide some principles for selecting residual shear strength for design. However, the selection of suitable undrained residual shear strength for design and analysis is still not satisfactory [10].

Soil mixtures such as silty clays, clayey sands and silty sands are more commonly found in nature and in earth structures than pure sands, silts and clays. However, the behaviour of soil mixtures is not well understood since soil mechanics has concentrated mostly on pure soils. It has been indicated since the 1960s that the presence of fines will affect a sand's resistance to liquefaction. Nevertheless, a review of studies published in literature shows that no clear conclusions can be drawn regarding how altering the fines fraction affects the liquefaction resistance of a sand under monotonic and cyclic loading. This is particularly true for soils containing non-plastic silty fines. Several laboratory studies have been carried out, and have shown what appear to be conflicting results. Studies have reported that increasing the fines content in a sand will improve the liquefaction resistance of the sand [11,6], decrease the liquefaction resistance of the sand [12, 13, 14, 15, 16, 17, 18], or decrease the liquefaction resistance until some limiting fines content is reached, and then increase its resistance [19, 20, 22]. Moreover, numerous studies have reported that the behaviour of sand mixed with fine grained soils depends mainly on the fines content and the plasticity index [23]. Indeed, to a certain limiting value of fines content, fine-grained soils occupy solely the voids and do not influence the behaviour of the mixture. For this reason, the notion of skeleton void ratio was suggested to characterize those kinds of soils [24, 25].

2 Experimental Investigation

2.1 Materials Tested

Sand samples were collected from the liquefied layer of the deposit areas close to the epicentre of Chlef earthquake (October 10, 1980). Chlef sand has been used for all tests presented in this research. Individual sand particles are subrounded and the predominant minerals are feldspar and quartz. The tests were conducted on the mixtures of Chlef sand and silt. The liquid limit and plastic limit of the silt are 27% and 22% respectively. Chlef sand was mixed with 0 to 50% silt to get different silt content. The dry pluviation method was employed in the present study to prepare the soil samples for the monotonic and cyclic testing. Several laboratory investigations have reported that the behaviour of sandy soils can be

greatly influenced by the sample preparation methods. However, experimental data related to the effect of depositional methods on the behaviour of sand with low plastic fines is very limited because most of the previous studies have concentrated their efforts on clean sand [26]. According to [26], reconstituted samples prepared by dry methods appeared to exhibit a more contractive or unstable behaviour. The index properties of the soils used during this study are summarized in Table 1. The grain size distribution curves for the soils are shown in Fig. 1. The variation of e_{max} (maximum void ratio corresponding to the loosest state of the soil sample) and e_{min} (minimum void ratio corresponding to the densest state of the soil sample) versus the fines content F_c (the ratio of the weight of silt to the total weight of the sand-silt mixture) is illustrated in Fig. 2. We note that the two indices decrease with the increase of the fines content until $F_c = 30\%$, then they increase with further increase in the amount of fines.

Table 1

Index properties of Chlef sand-silt mixtures

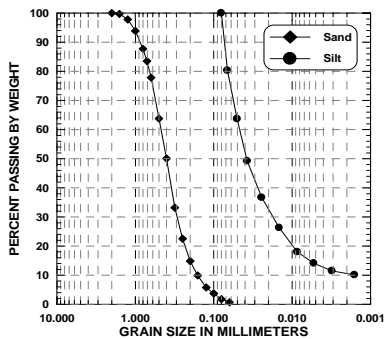| Material	Fines Content (%)	G_S	D_{50} (mm)	C_u	e_{min}	e_{max}	$I_p(\%)$
Clean Sand	0	2.680	0.40	2.90	0.535	0.854	-
Silty Sand	10	2.682	-	-	0.472	0.798	-
Silty Sand	20	2.684	-	-	0.431	0.748	-
Silty Sand	30	2.686	-	-	0.412	0.718	-
Silty Sand	40	2.688	-	-	0.478	0.732	-
Silty Sand	50	2.69	-	-	0.600	0.874	-
Silt	100	2.70	0.004	29.4	0.72	1.420	5.0

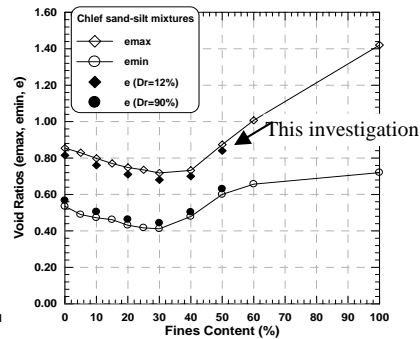
Figure 1

Grain size distribution curves

Figure 2

Extreme void ratios of the sand-silt mixtures e_{max} and e_{min} versus fines content

According to some authors, e.g. [24, 25], the behaviour of sand-silt mixture depends on the inter-granular void ratio (e_s):

$$e_s = \frac{(V_v + V_f)}{V_s} \tag{1}$$

where V_v, V_f, and V_s are the volume of voids, fines and sand grains, respectively. Thus $V_v + V_f$ is the volume of inter-granular void space.

When the specific gravity of the silt and sand are very close to each other, the inter-granular void ratio (e_s) can be determined with the global void ratio (e) and the fines content (Fc) using the following expression [4]:

$$e_s = \frac{e + (Fc/100)}{1 - (Fc/100)} \tag{2}$$

Fig. 3 shows the variation of the global and inter-granular void ratios versus fines content for the initial relative densities (Dr = 12% and 90%). As shown in this figure, the global void ratio (e) decreases with the fines content until the value of 30% and then increases; however, the inter-granular void ratio (e_s) increases hyperbolically with the increase of the fines content. This shows that the global void ratio cannot represent the amount of particle contents in silty sands. As the void ratio and proportion of the coarser of fine grains of soil changes, the nature of their microstructures also changes.
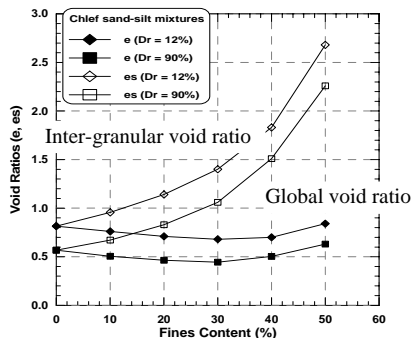

Figure 3

Variation of void ratios with fines content and relative densities

2.2   Experimental Program and Test Procedure

The present experimental study has been conducted to elucidate how the fines fraction affects the undrained residual shear strength and the liquefaction potential of Chlef sand-silt mixtures. For this purpose, a series of undrained monotonic and cyclic triaxial tests were carried out on reconstituted saturated samples of Chlef sand with variation in silt content ranging from 0 to 50%. All specimens were prepared by first estimating the dry weights of sand and silt needed for a desired proportion into the loose and medium dense state (Dr = 12, 50, 60% and 90%) using the undercompaction method of sample preparation which simulates a relatively homogeneous soil condition and is performed by compacted dry soil in layers to a selected percentage of the required dry unit weight of the specimen [27]. The dimensions of the samples were 70 mm in diameter and 70 mm in height in order to avoid the appearance of instability (sliding surfaces) and buckling. After the specimen was formed, the specimen cap was placed and sealed with O-rings, and a partial vacuum of 20 kPa is applied to the specimen to reduce the disturbances. Saturation was performed by purging the dry specimen with carbon dioxide for approximately 30 min. Deaired water was then introduced into the specimen from the bottom drain line. Water was allowed to flow through the specimen until an amount equal to the void volume of the specimen was collected in a beaker through the specimen's upper drain line. A minimum Skempton coefficient-value greater than 0.96 was obtained at back pressure of 100 kPa. All test specimens were isotropically consolidated at a mean effective pressure of 100 kPa, and then subjected to undrained monotonic and cyclic triaxial loading with a constant strain rate of 0.167% per minute (Cell pressure = 600 kPa and Back pressure = 500 kPa).

All the undrained triaxial tests for this study were carried out at a strain rate which was slow enough to allow pore pressure change to equalize throughout the sample with the pore pressure measured at the base of sample. All the tests were continued up to 24% axial strain.

3   Monotonic Tests Results

3.1   Undrained Compression Loading Tests

Figs. 4 and 5 show the results of the undrained monotonic compression triaxial tests carried out for different fines content ranging from 0 to 50% at 100 kPa mean confining pressure within two separate density ranges (Dr = 12, 90%). We notice in general that an increase in the amount of fines leads to a decrease in the deviatoric stress. This decrease results from the role of the fines in reducing the soil dilatancy and amplifying the phase of contractancy of the sand-silt mixtures,

leading to a reduction of the confining effective pressure and consequently to a decrease in the peak strength of the mixtures as is illustrated by Figs. 4a and 5a. The stress path in the (p', q) plane shows clearly the role of the fines in the decrease in the average effective pressure and the maximum deviatoric stress (Figs. 4b and 5b). In this case, the effect of fines on the undrained behaviour of the mixtures is observed for the lower fines contents (0% and 10%), and becomes very marked beyond 20%. These results are in good agreement with the observations of [12, 13]. Table 2 presents the summary of the undrained monotonic compression triaxial tests.

Table 2

Summary of monotonic compression triaxial tests results

Test No	Materials	Fc (%)	Dr (%)	γ_d (g/cm^3)	e	e_s	S_{us} (kPa)
1			12	1.48	0.815	0.815	17.37
2	Clean sand	0	90	1.71	0.567	0.567	20.94
3			12	1.52	0.760	0.956	15.44
4	Silty sand	10	90	1.78	0.505	0.672	19.61
5			12	1.57	0.710	1.140	14.46
6	Silty sand	20	90	1.83	0.463	0.829	18.42
7			12	1.60	0.680	1.400	13.48
8	Silty sand	30	90	1.86	0.443	1.060	17.34
9			12	1.58	0.700	1.830	10.75
10	Silty sand	40	90	1.79	0.503	1.510	15.92
11			12	1.46	0.840	2.680	06.51
12	Silty sand	50	90	1.65	0.630	2.260	12.00

e = global void ratio

e_s = inter-granular void ratio

(a)   (b)

Figure 4

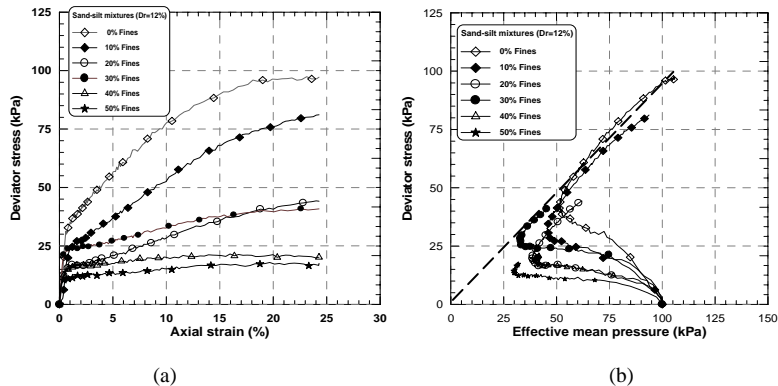Undrained monotonic response of the sand-silt mixtures (σ_3'=100 kPa, Dr = 12%)

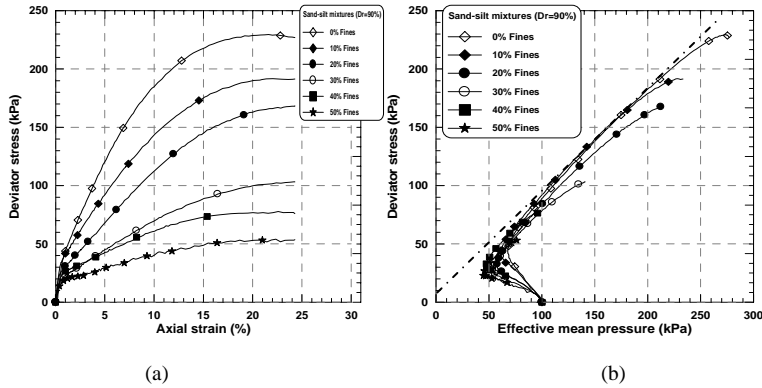(a)   (b)

Figure 5

Undrained monotonic response of the sand-silt mixtures (σ_3'=100 kPa, D_r = 90%)

3.2 Undrained Residual Shear Strength

When loose and medium dense sandy soils are subjected to undrained loading beyond the point of peak strength, the undrained shear strength declines to a near constant value over large deformation. Conventionally, this shear strength is called the undrained steady-state shear strength or residual shear strength. However, if the shear strength increases after passing through a minimum value, the phenomenon is called limited or quasi-liquefaction. Even limited liquefaction may result in significant strains and an associated drop in resistance [9] defined the undrained residual shear strength S_{us} as:

$$S_{us} = (q_s/2)\cos\phi_s = (M/2) \cos\phi_s(p_s') \tag{3}$$

$$M= (6 \sin\phi_s)/(3- \sin\phi_s) \tag{4}$$

where q_s, p_s' and ϕ_s indicate the deviator stress ($\sigma_1'- \sigma_3'$), the effective mean principal stress ($\sigma_1'+ 2\sigma_3'$)/3 and the mobilized angle of inter-particle friction at the quasi-steady state (QSS) respectively. For the undrained tests conducted at a constant confining pressure and various initial relative densities and fines content, the deviatoric stress (q_s) was estimated at quasi-steady state point along with the mobilized internal friction angle (Fig. 6). Furthermore, the undrained residual shear strength was calculated according to the relation (3).
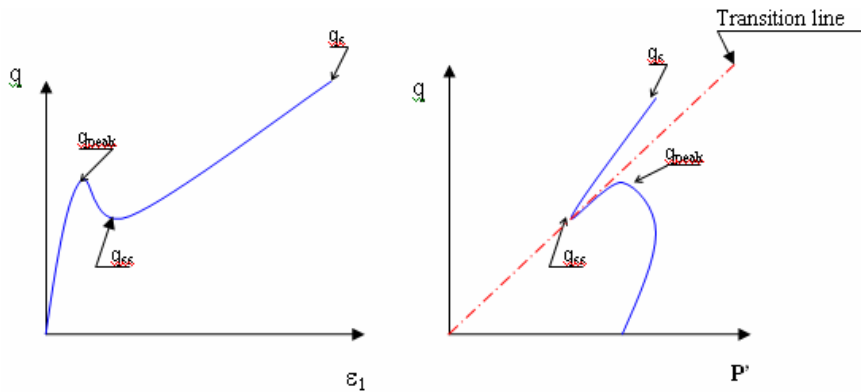
Figure 6
Determination of the phase transition point

Fig. 7 shows the undrained residual shear strength versus the global void ratio and fines content. It is clear from this that the undrained residual shear strength decreases linearly as the global void ratio decreases and the fines content increases for the loose and dense state of the specimen ($D_r = 12\%$ and 90%) up to 30% fines content. It means that with a decrease in the global void ratio and an increase in the fines content, the undrained residual shear strength also decreases. In this case we can say that the global void ratio does not represent the real behaviour of silty sand soil in the range of 0-30% fines content. Indeed, the global void ratio appears to be a parameter not as pertinent in sand-fines mixtures as in clean sands for characterizing the mechanical state of these materials. Beyond $F_c = 30\%$ the undrained residual shear strength continues to decrease almost linearly with increases in the global void ratio and the fines content for the two relative densities ($D_r = 12\%$ and 90%).
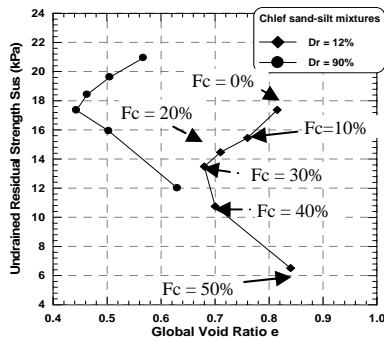

Figure 7
Variation of undrained residual strength with void ratio and relative density (σ_3'=100 kPa)

Fig. 8 illustrates the undrained residual shear strength versus the fines content. Due to obtained results, it can be concluded that we could assume that the undrained residual shear strength of the sand-silt mixtures S_{us} varies linearly and independently of the effective vertical overburden pressure σ_v' for the two relative densities (Dr = 12% and 90%) allowing us to estimate it in the field with no need for in-situ physical parameters measurements in the case of normally consolidated soils ($\sigma_v' = \sigma_3'$). In this laboratory investigation, for the range of 0% to 50% fines content in normally consolidated undrained triaxial compression tests, the following expressions are suggested to evaluate the undrained residual shear strength which is a function of the fines content (Fc):

$$S_{us} = 17.9552 - 0.1982(Fc) \qquad \text{for Dr} = 12\%$$

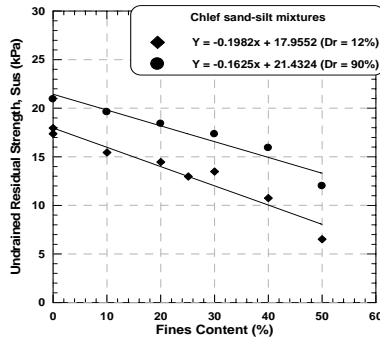$$S_{us} = 21.4324 - 0.1625(Fc) \qquad \text{for Dr} = 90\%$$

Figure 8
Undrained residual shear strength versus fines content and relative density (σ_3'=100 kPa)

Fig. 9 shows the undrained residual shear strength versus the inter-granular void ratio. It seems that the variation of the undrained residual strength due to the amount of fines is related to the inter-granular void ratio in the range of 0-50% fines content. In this case, the behaviour of silty sand samples is influenced by the contacts of coarser grains, which is quantified by the inter-granular void ratio. By increasing the fines content in the range of 0-50%, the contact between sand grains decreases and therefore the inter-granular void ratio increases and the undrained residual shear strength decreases. Hence, in the range of 0-50% fines content, we could assume that the undrained residual shear strength decreases linearly with the increase of the inter-granular void ratio. In this case, the following expressions are suggested to evaluate the undrained residual shear strength which is a function of the inter-granular void ratio (e_s) for the range of 0% to 50% fines content in normally consolidated undrained triaxial compression tests:

$$S_{us} = 21.1432 - 5.5378(e_s) \qquad \text{for Dr} = 12\%$$

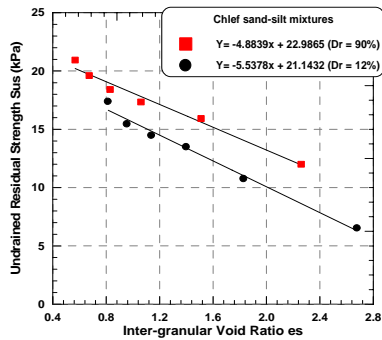$$S_{us} = 22.9865 - 4.8839(e_s) \qquad \text{for Dr} = 90\%$$

Figure 9

Undrained residual shear strength versus inter-granular void ratio and relative density (σ_3'=100 kPa)

Fig. 10 shows the variation of residual shear strength (S_{us}) with relative density (Dr) at various fines contents. It is clear from this Figure that an increase in the relative density results in an increase in the undrained residual shear strength at a given fines content. [3, 28] report similar behaviour of increasing residual shear strength with increasing relative density. The present laboratory study focuses on the effect of the fines content and the inter-granular void ratio on the undrained residual shear strength of silty sands at various relative densities (Dr = 12% and 90%). It can be noticed from the results of this study that there is a significant decrease in the undrained residual shear strength with increase in the fines content or the inter-granular void ratio, but there is a significant increase in the undrained residual shear strength with an increase in the relative density and a significant decrease of the undrained residual strength with an increase in the fines content. The aspect of the present study is in good agreement with the experimental work reported by [9] on Tia Juana silty sand, [29] on silty sands retrieved from the Lower San Fernando Dam, and by [8] on Adebil sand with different fractions of fines content.
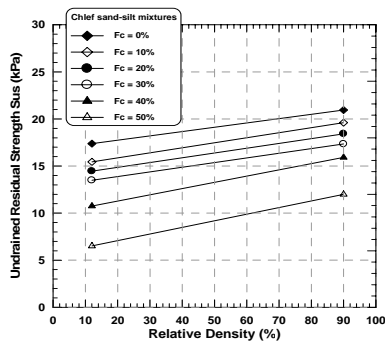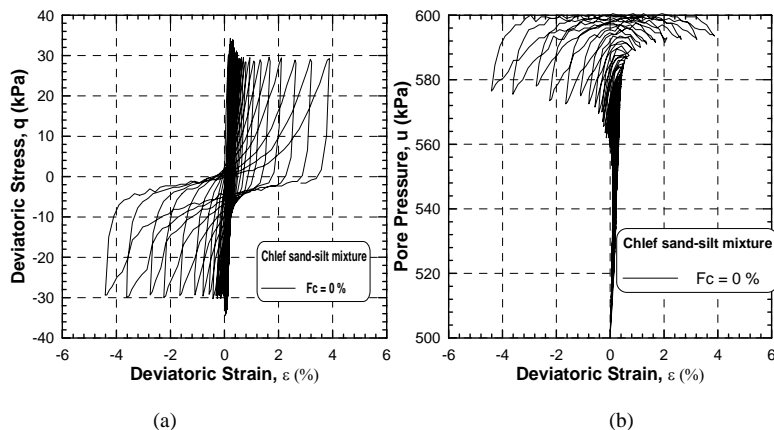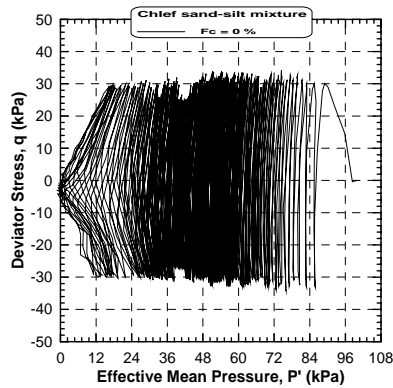

Figure 10

Undrained residual shear strength versus relative density at various fines contents (σ_3'=100 kPa)

4   Cyclic Tests Results

4.1   Undrained Loading Tests

Three series of stress-controlled cyclic triaxial tests were carried out on isotropically consolidated soil specimens with different fines content ranging from 0 to 40% and alternated symmetric deviator stress under undrained conditions simulating essentially undrained field conditions during earthquakes in order to produce liquefaction potential curves of the sand-silt mixtures. All along the test program, a frequency of 0.5 Hz was used. The first one included three alternated cyclic tests and was realized on clean sand samples (Fc = 0%) with a relative density of 50% and a confining initial pressure of 100 kPa. The loading amplitudes of the cycles (q_m) used are respectively 30, 50 and 70 kPa. The tests of the second one were realized on the mixture sand-silt samples with a fines content of 10% and loading amplitudes of 30, 40 and 60 kPa; while the third series of tests concerned samples with a fines content of 40% and loading amplitudes of 20, 30 and 50 kPa. It is noted that the presence of fines affects considerably the liquefaction of the samples. Fig. 11 illustrates the results of the test carried out on clean sand samples (Fc = 0%) with loading amplitude of 30 kPa. It is clear from the figure that increases in the pore water pressure during the cycles result in a reduction of the average effective pressure. The rate of increase in the pore pressure remains low, because liquefaction is obtained only after 158 cycles (Fig. 11); for the test with qm = 30 kPa and a fines content of 10%, we notice an important increase in the pore water pressure during the 27th cycle with a significant development of the axial strain (2.5%) leading to the liquefaction of the sand-silt mixture sample (Fig. 12).
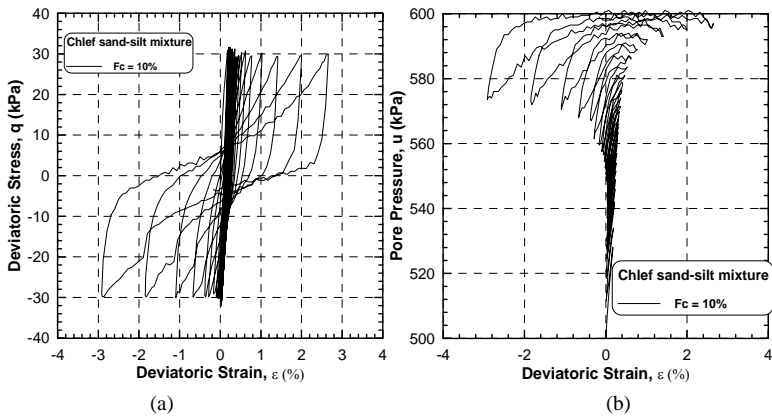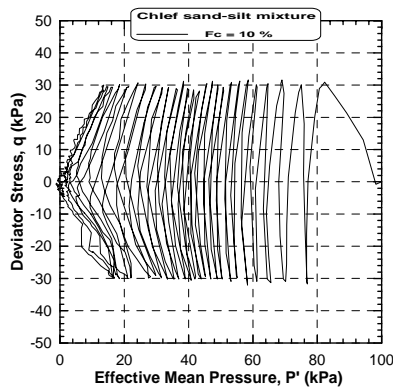

(a)   (b)

(c)

Figure 11

Undrained cyclic response of clean sand (Fc=0%, q_m=30 kPa, Dr=50%, σ_3'=100 kPa)

(a)

(b)

(c)

Figure 12

Undrained cyclic response of sand-silt mixture (Fc =10%, qm =30 kPa, Dr =50%, σ_3'=100 kPa)

While for the test with a loading amplitude of 30 kPa and a fines content of 40%, we notice an important increase in the pore water pressure during the 3rd cycle with a significant development of the axial strain leading to the liquefaction of the sample at the 4th cycle (Fig. 13). This shows that the increase in the amount of fines in the range of (0-40%) leads to an increase in the risk of liquefaction.
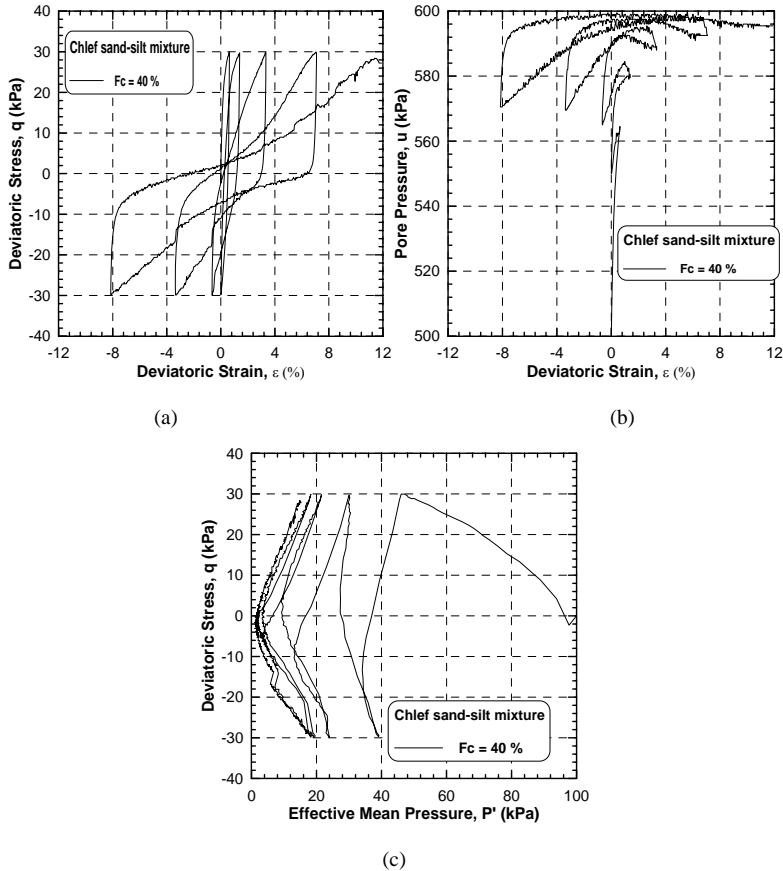

(a)

(b)

(c)

Figure 13

Undrained cyclic test on Chlef sand-silt mixture (Fc = 40%, qm =30 kPa, Dr =50%, σ_3'=100 kPa)

4.2 Effect of Fines on the Liquefaction Potential

Figs. 14a and 14b illustrate the variation of the cyclic stress ratio (CSR = q_m / $2\sigma_3$') and the cyclic liquefaction resistance (CLR) with the number of cycles (N_c) and fines content respectively. The cyclic liquefaction resistance (CLR) is defined as the cyclic stress ratio leading to liquefaction for 15 cycles according to [9]. The test results show that the liquefaction resistance of the sand-silt mixture decreases with increase in the fines content. These results confirm those found during

monotonic tests showing that the increase in the fines content reduces the dilatancy phase and amplifies the phase of contractancy. Consequently, the increase of the contractancy phase induced a reduction in the liquefaction potential with the increase in the fines content. For the mixtures of Chlef sand-silt, the presence of fine-grained soils increases the phase of contractancy resulting in a significant decrease in the liquefaction potential particularly, for the fines contents less than 10%. It should be noted that for the studied amplitude (qm= 30 kPa), the increase in fines content accelerates the liquefaction processes of the Chlef sand-silt mixtures. Fig. 14c presents the loading cycles till the liquefaction versus the fines content. We notice that the liquefaction resistance decreases with the increase in the fines content and loading amplitudes. The soil samples sheared with higher level loading (CSR = 0.25) are more vulnerable to liquefaction than those sheared with smaller loading level (CSR = 0.15).
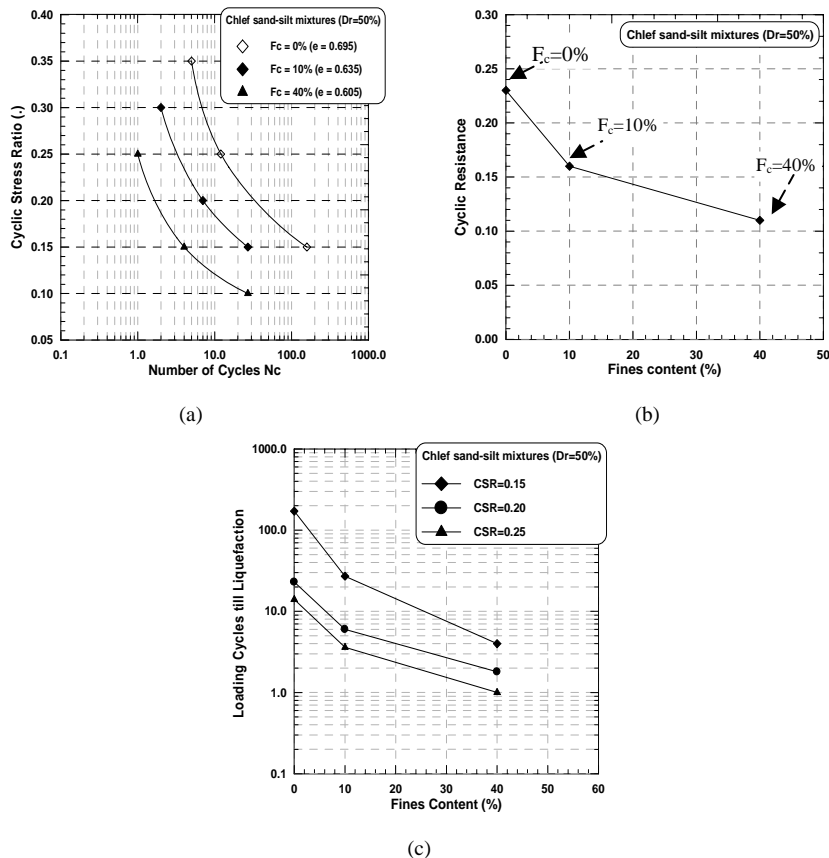

(a)

(b)

(c)

Figure 14

Effect of fines on the liquefaction potential of the Chlef sand-silt mixtures (σ_3'=100 kPa, Dr=50%)

4.3 Effect of the Relative Density

Undrained cyclic tests were performed on Chlef sand–silt mixtures (Fc = 5%) for two relative densities (Dr = 12% and 60%). For each density, we varied the loading amplitude in order to draw the liquefaction potential curve. The tests were carried out for the amplitudes qm = 30, 50 and 70 kPa. We note that the liquefaction was reached quickly for the higher amplitudes: after 2 and 3 cycles respectively for the loading amplitudes of q_m = 70 and 50 kPa, whereas the liquefaction under the loading amplitude qm = 30 kPa for the tests carried out with a relative density Dr = 12% was reached at 24 cycles. For the tests carried out with a relative density of Dr = 60% and for the same loading amplitudes, the liquefaction was reached at 5, 13 and 167 cycles.

Fig. 15 summarizes the results of all these tests. Fig. 15a illustrates the influence of the relative density on the liquefaction potential of Chlef sand-silt mixture (Fc = 5%). It shows clearly that the increase in the relative density leads to an increase in the liquefaction resistance of Chlef sand-silt mixture. Fig. 15b shows the influence of the global void ratio on the liquefaction resistance of the Chlef sand-silt mixtures. This figure shows clearly that the liquefaction resistance decreases with the increase in the global void ratio and the loading amplitude. Note that the reduction in the liquefaction resistance of Chlef sand-silt mixture becomes very marked for the smaller cyclic stress ratios CSR = 0.15 and 0.25.
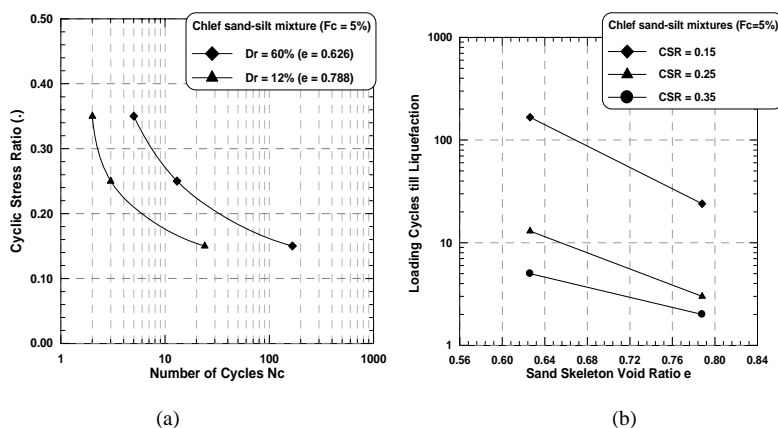

(a)   (b)

Figure 15
Effect of relative density on the liquefaction potential of Chlef sand-silt mixture
(σ_3'=100 kPa, Fc =5%)

Conclusion

A series of undrained monotonic and cyclic triaxial tests were carried out on silty sand collected from liquefied sites at Chlef River, Algeria. The effect of variation in the fines content was studied. In the light of the experimental evidence, the following conclusions can be drawn:

Undrained monotonic tests performed with initial relative densities of 12% and 90% showed contractive behaviour of the Chlef sand-silt mixtures samples at the initial confining pressure in the global void ratio range tested. The undrained residual shear strength decreases as the global void ratio decreases and the fines content increases up to 30% fines content. Beyond that, it decreases with increases in the global void ratio and the fines content. The undrained residual shear strength decreases linearly with an increase in the fines content and the inter-granular void ratio. The peak and residual strengths of the soil are sensitive to the presence of fine grained soils. As the fines content increases, the peak and the residual strengths decrease. The strength of silty sand up to 50% fines content is less than that of the clean sand. This means that the soil becomes weakened with an increase in the fines content up to 50%.

Undrained cyclic tests show that the increase in fines content reduces the liquefaction potential of the sand-silt mixtures and the samples sheared with higher level loading (CSR= 0.25) are more vulnerable to liquefaction than those sheared with smaller loading level (CSR = 0.15). They also show that the liquefaction resistance increases with the relative density but it decreases with the increase in the global void ratio and the loading amplitude. Note that the reduction in the liquefaction resistance of Chlef sand-silt mixture becomes very marked for the smaller cyclic stress ratios CSR = 0.15 and 0.25.

The results of cyclic tests confirm those found during monotonic tests showing that the increase in the fines fraction reduces the soil dilatancy and amplifies the phase of contractancy inducing an important decrease in the liquefaction potential of Chlef sand-silt mixtures. Note that the increase in the fines content accelerates the liquefaction phenomenon for the studied amplitude (qm = 30 kPa).

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