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# The undrained shear strength characteristics of silty sand: an experimental study of the effect of fines

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## ABSTRACT

This laboratory investigation has been conducted to elucidate how the fines fraction affects the undrained residual shear strength and liquefaction potential of sand-silt mixtures (Algeria). A series of monotonic and cyclic undrained triaxial tests were carried out on undrained, reconstituted, saturated samples of sand with varying fines content ranging from 0 to 50%. These were undertaken in order to evaluate the effect of the fines fraction on the undrained residual shear strength and liquefaction potential of loose, medium dense, and dense sand-silt mixtures ($D_r$ = 17%, 53%, 62% and 91%), under an initial confining pressure of 100 kPa. The results of the monotonic tests indicate that the stress-strain response and shear strength behaviour is controlled by the percentage of the fines fraction and the samples become contractive for the studied relative density (Dr = 17% and 91%). The undrained residual shear strength decreases as the gross void ratio decreases, and the fines content increases up to 30%. Above this level of fines, it decreases with increasing gross void ratio. Moreover, the undrained residual strength decreases linearly as the fines content and the intergranular void ratio increase. Cyclic test results show that for the studied amplitude, the increase in fines content leads to an acceleration of liquefaction. The liquefaction resistance decreases with the increase in gross void ratio and the loading amplitude.

**Keywords:** silty sand, residual shear strength, fines content, cyclic tests, liquefaction, pore pressure

## 1. INTRODUCTION

On October 10th, 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 its approximate duration was between 35 and 40 sec. The earthquake devastated the city of Chlef, with the population estimated at 125,000, and 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 in the affected area, especially along the Chlef river banks, huge masses of sandy soils were ejected on to the surface. Important damage to different structures was recorded.

During earthquakes, the ground shakes, causing the loss of strength in cohesion less soils. As the soil behaves in a manner similar to liquids, this phenomenon is called soil liquefaction. Liquefaction can cause settlements, landslides, earth dam failures, or other hazards. Knowledge and understanding of 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 research by ZLATOVIC & ISHIHARA (1995), LADE & YAMAMURO (1997), THEVANAYAGAM et al. (1997), THEVANAYAGAM (1998), YAMAMURO & LADE (1998), AMINI & QI (2000), NAEINI (2001), NAEINI & BAZIAR (2004), indicate that sand deposited with silt can suffer greater liquefaction than clean sand. Also, strain properties and pore pressure generation in silty sand samples are quite different from clean sand. These new discoveries emphasize the specific important features of deposits with a mixture of sand and silt. Moreover, the behaviour of silty-sandy soils such as hydraulic fills during earthquakes is not clearly understood. Therefore, further study of the behaviour of silty sand is needed for liquefaction assessment of silty sandy soils.

Post-earthquake behaviour of silty sand and, consequently, the stability of structures founded on liquefied soil depend on the post-liquefaction shear strength of soil. The strength of soils mobilized at the quasi-steady state has important implications for engineering practice (ISHIHARA, 1993). Laboratory study, especially in the last ten years, has made a great contribution in clarifying parameters that control the residual shear strength, and provide some principles for selecting the appropriate undrained residual shear strength for design and analysis.

## 2. LABORATORY TESTING PROGRAM

### 2.1. Soils tested

Sand samples were collected from the liquefied layer of the deposits close to the epicentre of the Chlef earthquake (October 10th, 1980). Chlef sand has been used for all tests presented here. Individual sand particles are subrounded and the predominant minerals are feldspar and quartz. The tests were conducted on mixtures of Chlef sand and silt. Liquid limit and plastic limit of the silt are 27% and 22% respectively. Chlef sand was mixed with 0 to 50% silt to obtain varying silt contents. The dry pluviation method was employed in the present study to prepare the soil samples for the monotonic and cyclic testing. The index properties of the sand-silt mixtures used during this investigation are summarized in Table 1. Grain size distribution curves of the soils are shown in Figure 1. 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 Fc (the ratio of the weight of silt to the total weight of the sand-silt mixture) is given in Figure 2. We note that the two indices decrease with an increase of the fines content until $F_c$ = 30%, then, they decrease with further increase in the amount of fines.

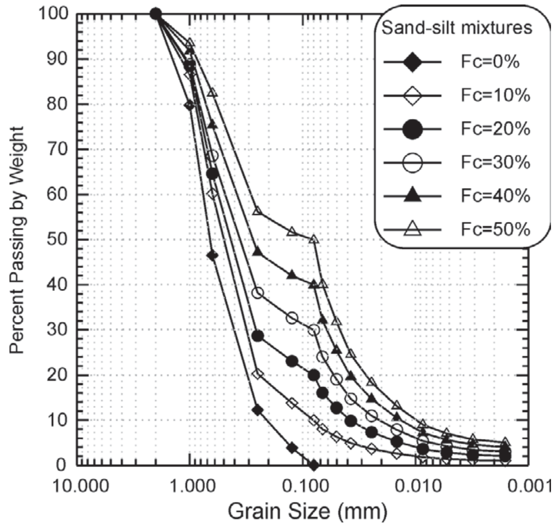


**Figure 1:** Grain size distribution curves of tested materials.

### 2.2. Experimental program and test procedure

The present experimental study has been conducted to elucidate how the fines fraction affects the liquefaction potential and steady-state strength of mixed samples of sand. For this purpose, a series of undrained triaxial compression tests under monotonic loading conditions was carried out on reconstituted saturated samples of Chlef sand with silt content ranging from 0 to 50%. All specimens were prepared by first estimating the dry weights of sand and silt needed for the desired proportion into the loose, medium dense and dense state ($D_r$ = 17, 53, 62% and 91%), using the undercompac-

**Table 1:** Index properties of Chlef sand-silt mixtures.

| Material | Fc (%) | $G_S$ | $D_{10}$ (mm) | $D_{50}$ (mm) | $C_u$ | $C_c$ | $e_{min}$ | $e_{max}$ | $I_p$ (%) |
|---|---|---|---|---|---|---|---|---|---|
| Sand | 0 | 2.680 | 0.22 | 0.68 | 3.36 | 1.36 | 0.535 | 0.854 | - |
| Silty Sand | 10 | 2.682 | 0.08 | 0.50 | 7.75 | 2.76 | 0.472 | 0.798 | - |
| | 20 | 2.684 | 0.038 | 0.43 | 15.26 | 1.31 | 0.431 | 0.748 | - |
| | 30 | 2.686 | 0.022 | 0.37 | 23.18 | 0.57 | 0.412 | 0.718 | - |
| | 40 | 2.688 | 0.015 | 0.29 | 27.33 | 0.64 | 0.478 | 0.732 | - |
| | 50 | 2.69 | 0.011 | 0.08 | 28.18 | 0.67 | 0.600 | 0.874 | - |
| Silt | 100 | 2.70 | - | - | - | - | 0.72 | 1.420 | 5.0 |

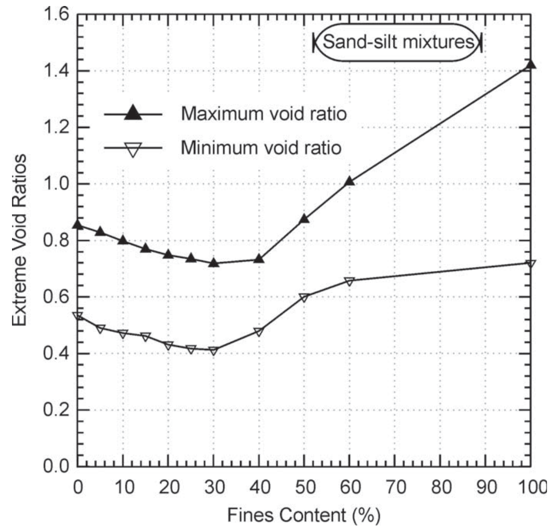


**Figure 2:** Extreme void ratios of the sand-silt mixtures versus fines content.

tion method of sample preparation. This simulates a relatively homogeneous soil condition and is performed by compacting dry soil in layers to a selected percentage of the required dry unit weight of the specimen (LADD, 1978). The samples were 70 mm in diameter and 70 mm in height in order to avoid the appearance of shear banding (sliding surfaces) and buckling. After the specimen has been formed, the specimen cap is placed and sealed with O-rings, and a partial vacuum of 25 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. 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 a 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 triaxial loading with a constant strain rate of 0.167% per minute (Cell pressure = 600 kPa and Back pressure = 500 kPa).

All 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.

According to some authors KENNY (1977); MITCHELL (1993), the behaviour of a sand-silt mixture depends on the intergranular 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 intergranular void space.

When the specific gravity of the silt and sand are very close to each other, the intergranular void ratio ($e_s$) can be determined with the gross void ratio (e) and the fines content ($F_c$) using the following expression THEVANAYAGAM (1998):

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

Figure 3 shows the variation of the gross and intergranular void ratios versus fines content at 100 kPa mean confining pressure for the initial relative densities ($D_r$ = 17%, 91%). As could be seen in this Figure, the gross void ratio (e) decreases with the fines content until the value of 30% and then increases, however, the intergranular void ratio ($e_s$) increases hyperbolically with the increase in fines content. This showed that the gross void ratio cannot represent the amount of particle contacts in silty sands. As the void ratio and proportion of the coarse and fine-grained soil changes, the nature of their microstructures also changes.

**Table 2:** Summary of monotonic compression triaxial test results.

| Test No | Material | $F_C$(%) | $D_r$(%) | $\gamma_d$ (g/cm³) | e | $e_f$ | $e_s$ | $S_{us}$(kPa) |
|---|---|---|---|---|---|---|---|---|
| 1 | | | 17 | 1.49 | 0.815 | 0.800 | 0.800 | 16.85 |
| 2 | Sand | 0 | 91 | 1.72 | 0.567 | 0.564 | 0.564 | 20.14 |
| 3 | | | 17 | 1.54 | 0.760 | 0.743 | 0.937 | 15.05 |
| 4 | | 10 | 91 | 1.79 | 0.505 | 0.501 | 0.668 | 18.84 |
| 5 | | | 17 | 1.59 | 0.710 | 0.694 | 1.118 | 13.93 |
| 6 | | 20 | 91 | 1.84 | 0.463 | 0.459 | 0.824 | 17.64 |
| 7 | Silty sand | | 17 | 1.61 | 0.680 | 0.666 | 1.380 | 12.81 |
| 8 | | 30 | 91 | 1.87 | 0.443 | 0.439 | 1.056 | 16.52 |
| 9 | | | 17 | 1.59 | 0.700 | 0.689 | 1.815 | 10.95 |
| 10 | | 40 | 91 | 1.79 | 0.503 | 0.501 | 1.502 | 14.83 |
| 11 | | | 17 | 1.47 | 0.840 | 0.827 | 2.654 | 07.45 |
| 12 | | 50 | 91 | 1.66 | 0.630 | 0.625 | 2.250 | 13.38 |

e – initial gross void ratio
$e_f$ – post-consolidation gross void ratio
$e_s$ – post-consolidation intergranular void ratio
$D_r$ – post-consolidation relative density

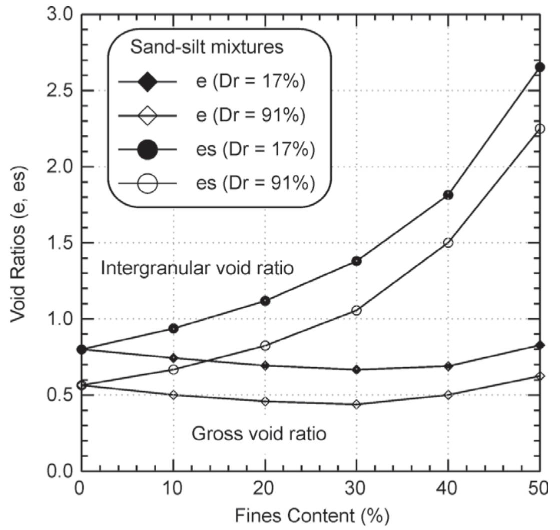


**Figure 3:** Variation of void ratios with fines content and relative density ($\sigma_3'$= 100 kPa).

## 3. MONOTONIC TEST RESULTS

### 3.1. Undrained compression loading tests

Figures 4 and 5 show the results of the undrained monotonic compression triaxial tests carried out for different fines contents ranging from 0 to 50% at 100 kPa mean confining pressure within two separate density ranges ($D_r$ = 17%, 91%). Generally, the increase in the amount of fines leads to a decrease in the deviatoric stress. This increase results from the role of the fines in increasing the contractancy phase of the sand-silt mixtures, leading to a reduction of the confining effective pressure and consequently to a decrease of the peak resistance of the mixtures as illustrated by Figures 4a and 5a. The stress path in the (p', q) plane clearly shows the role of the fines in the decrease of 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 SHEN et al. (1977) and TRONCOSO & VERDUGO (1985).

Table 2 presents a summary of the undrained monotonic compression triaxial test.

### 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 a significant strain and associated drop in resistance. ISHIHARA (1993) defined the 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 $\phi_s$ indicate the deviator stress ($\sigma_1'$- $\sigma_3'$), the effective mean principal stress ($\phi_1'$+ $2\phi_3'$)/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. Furthermore, the residual shear strength was calculated according to relationship (3).

Figure 6 shows the undrained residual shear strength versus the gross void ratio and fines content. It is clear that the undrained shear strength decreases linearly as the gross void ratio decreases and fines content increases for the loose and dense state of the specimen ($D_r$ = 17% and 91%) up to 30% fines content. Therefore, when decreasing the gross void ratio and increasing the fines content, the undrained residual shear strength also decreases. In this case the gross void ra-

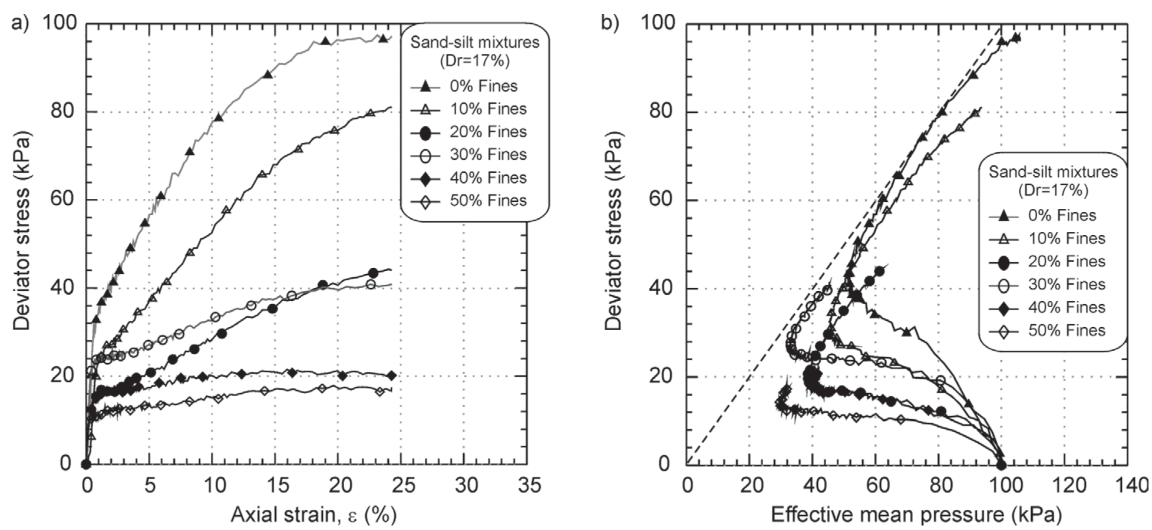


**Figure 4:** Undrained monotonic response of the sand-silt mixtures ($\sigma_3'$=100 kPa, $D_r$ = 17%).

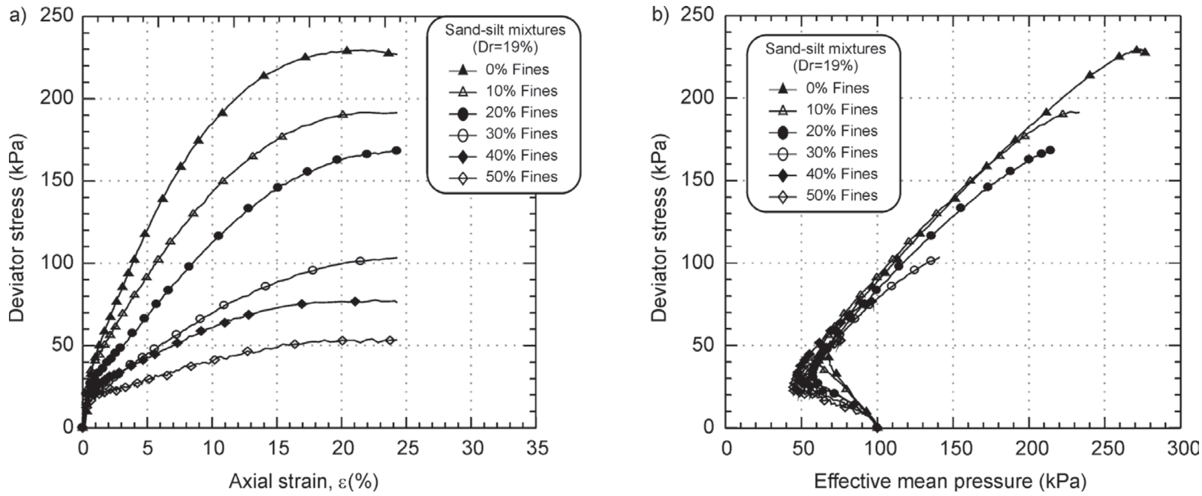


**Figure 5:** Undrained monotonic response of the sand-silt mixtures ($\sigma_3$'=100 kPa, $D_r$ = 91%).

tio does not represent the real behaviour of silty sandy soil in the range of 0–30% fines content. Indeed, the gross 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 shear strength continues to decrease almost linearly with increasing gross void ratio and the fines content for the two relative densities ($D_r$ = 17% and 91%)

Figure 7 illustrates the undrained residual shear strength versus the fines content. It can be concluded that the undrained residual shear strength of the sand-silt mixtures $S_{us}$ varies linearly for the two relative densities ($D_r$ = 17% and 91%) allowing it to be estimated in the field with no need for in-situ physical parameter measurements in the case of normally consolidated soils ($\sigma_v$'= $\sigma_c$'). 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 in order to evaluate the undrained residual shear strength which is a function of the fines content ($F_c$):

$$S_{us} = 17 - 0.17(F_c) \qquad \text{for } D_r = 17\%$$

$$S_{us} = 20 - 0.14(F_c) \qquad \text{for } D_r = 91\%$$

Figure 8 shows the undrained residual shear strength versus the intergranular void ratio. It seems that the variation of the undrained residual strength due to the amount of fines is related to the intergranular void ratio in the range 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 intergranular void ratio. By increasing the fines content in the range of 0–50%, the contact between sand grains decreases and therefore the intergranular 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 de-

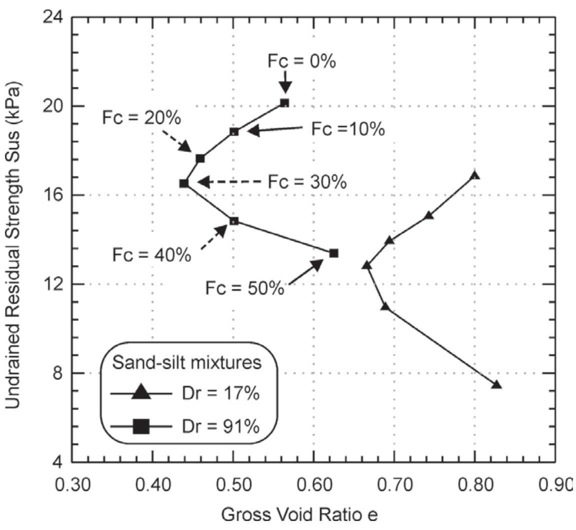


**Figure 6:** Variation of the undrained residual strength with gross void ratio and fines content ($\sigma_3$'=100 kPa).

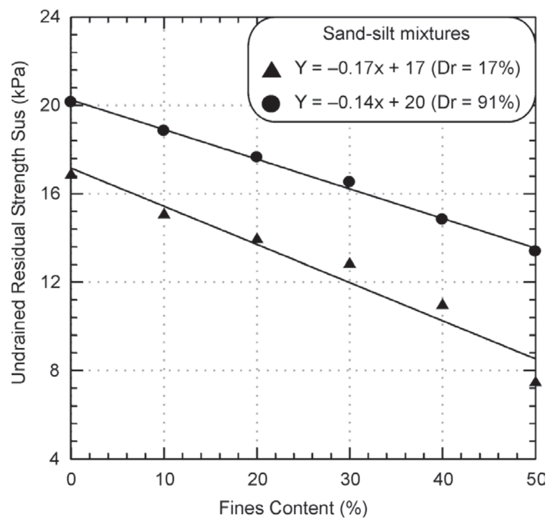


**Figure 7:** Undrained residual shear strength versus fines content ($\sigma_3$'=100 kPa).

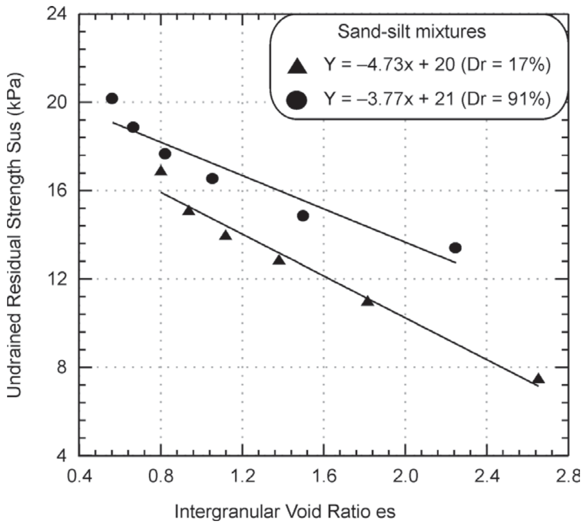


**Figure 8:** Undrained residual shear strength versus intergranular void ratio and fines content ($\sigma_3'$=100 kPa).

creases linearly with the increasing intergranular void ratio. In this case, the following expressions are suggested to evaluate the undrained residual shear strength which is a function of the intergranular void ratio ($e_s$), for the range of 0% to 50% fines content in normally consolidated undrained triaxial compression tests:

$S_{us}$= 20 – 4.73($e_s$) for $D_r$ = 17%

$S_{us}$= 21 – 3.77($e_s$) for $D_r$ = 91%

Figure 9 shows the variation of residual shear strength ($S_{us}$) with relative density ($D_r$) 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. THEVANAYAGAM et al. (1997) and SITHARAM et al. (2004) 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 intergranular void ratio on the undrained residual shear strength of silty sands at various relative densities ($D_r$ = 17% and 91%). 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 intergranular void ratio, but there is a significant increase in the undrained residual shear strength with increase in relative density. This aspect of the present study is in good agreement with experimental work reported by ISHIHARA (1993) on the Tia Juana silty sand, BAZIAR & DOBRY (1995) on silty sands retrieved from the Lower San Fernando Dam, and by NAEINI & BAZIAR (2004) on the Adebil sand with different fines contents.

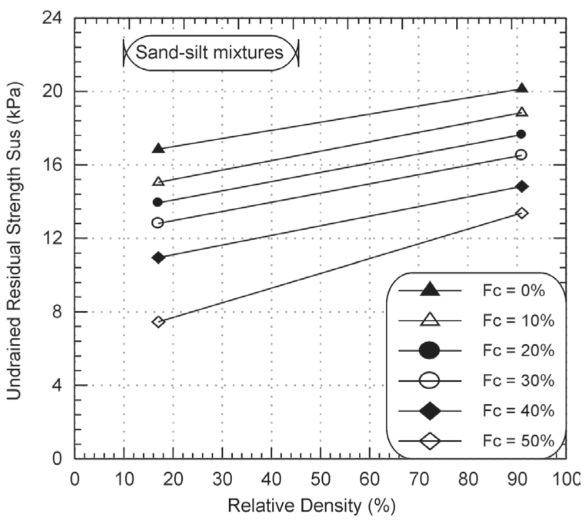


**Figure 9:** Undrained residual shear strength versus relative density and fines content ($\sigma_3'$=100 kPa).

## 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 contents ranging from 0–40% 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. Throughout the test program, a frequency of 0.5 Hz was used. The first test included three alternating cyclic tests on clean sand samples ($F_c$= 0%) with a relative density of 53% and a confining initial pressure of 100 kPa. The loading amplitudes of the cycles ($q_m$) were 30, 50 and 70 kPa. The tests of the second series were undertaken on the 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 considerably affects the liquefaction of the samples. Figure 10 illustrates the results of the test carried out on clean sand samples ($F_c$ = 0%) with loading amplitude of 30 kPa. It is clear from the Figure that the pore water pressure increases during the cycles resulting 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 (Figure 10). For the test with $q_m$ = 30 kPa and a fines content of 10%, an important increase in the pore water pressure during 27$^{th}$ cycle was observed, with a significant development of the axial strain (2.5%) leading to the liquefaction of the sand-silt mixture sample (Figure 11).

The test with loading amplitude of 30 kPa and a fines content of 40%, showed an important increase in the pore water pressure during the 3$^{rd}$ cycle, with a significant development of the axial strain leading to the liquefaction of the sample at the 4$^{th}$ cycle (Figure 12). 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.

### 4.2. Effect of fines on the liquefaction potential

Figures 13a and 13b show the variation of the cyclic stress ratio (CSR= $q_{max}/2\sigma_c$') and the cyclic liquefaction resistance

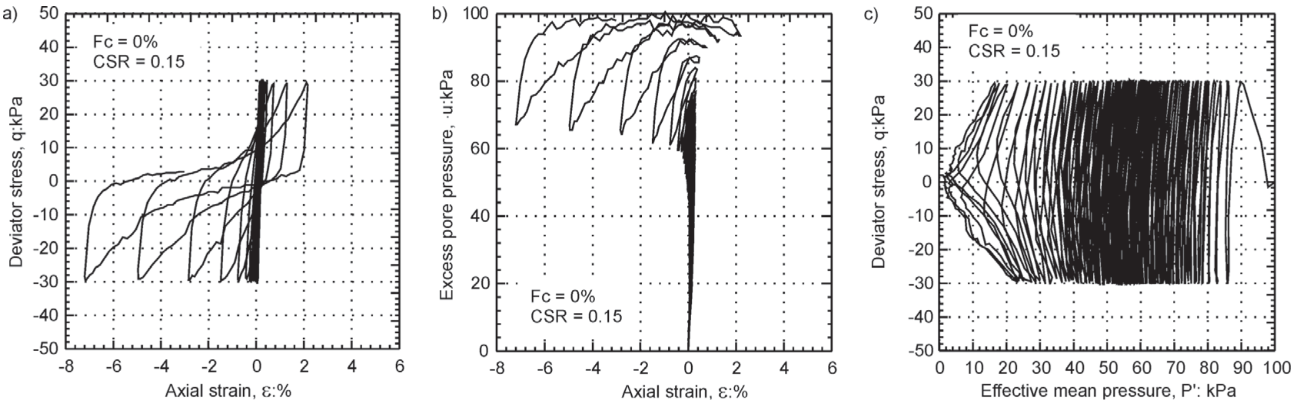


**Figure 10:** Undrained cyclic response of clean sand ($e_f = 0.685$, $e_s = 0.685$, $q_m = 30$ kPa, $D_r = 53\%$, $\sigma_3' = 100$ kPa).

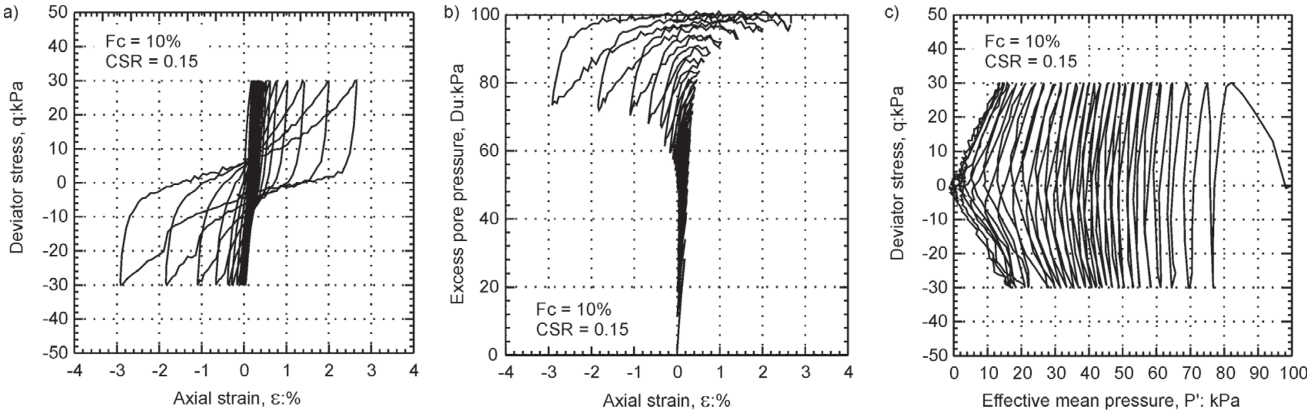


**Figure 11:** Undrained cyclic response of sand-silt mixture ($e_f = 0.625$, $e_s = 0.806$, $q_m = 30$ kPa, $D_r = 53\%$, $\sigma_3' = 100$ kPa).

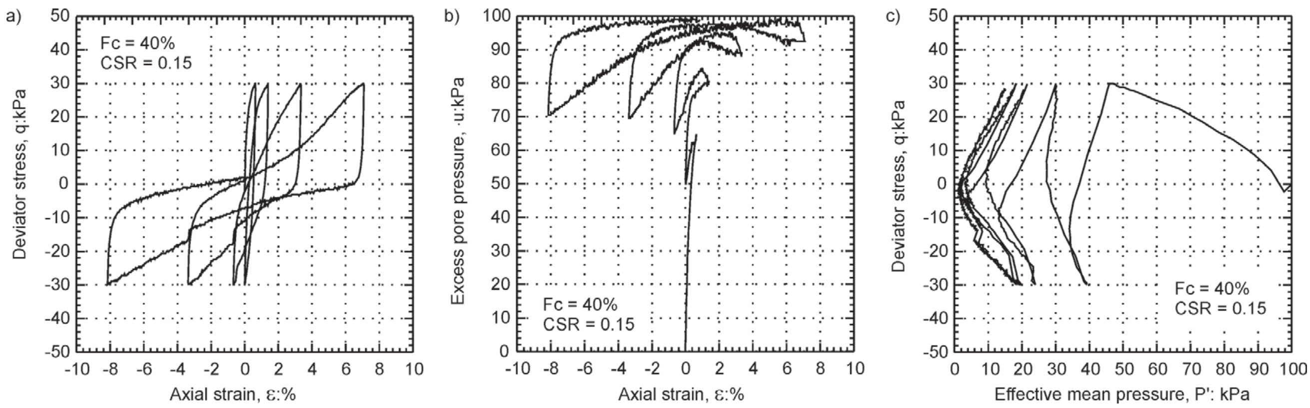


**Figure 12:** Undrained cyclic response on sand-silt mixture ($e_f = 0.597$, $e_s = 1.662$, $q_m = 30$ kPa, $D_r = 53\%$, $\sigma_3' = 100$ kPa).

(CLR) with the number of cycles ($N_c$) and fines content. According to ISHIHARA (1993), the cyclic liquefaction resistance (CLR) is defined as the cyclic stress ratio leading to liquefaction for 15 cycles. It was observed that the liquefaction potential of the sand-silt mixture decreases with further increase in the fines content. These results confirm those found during monotonic tests showing that the increase in the fines content leads to an increase in the contractancy phase. Consequently, the increase of the contractancy phase induced a reduction in the liquefaction potential with the increase in fines content. For the mixtures of Chlef sand-silt, the presence of fine grained soils increases the phase of contractancy resulting in a significant decline of the liquefaction potential. It should be noted that for the studied amplitude ($q_m = 30$ kPa), the increase in fines content leads to an acceleration of the liquefaction. Figure 13c shows the cycles of loading till the liquefaction versus the fines content. We notice that the liquefaction resistance decreases

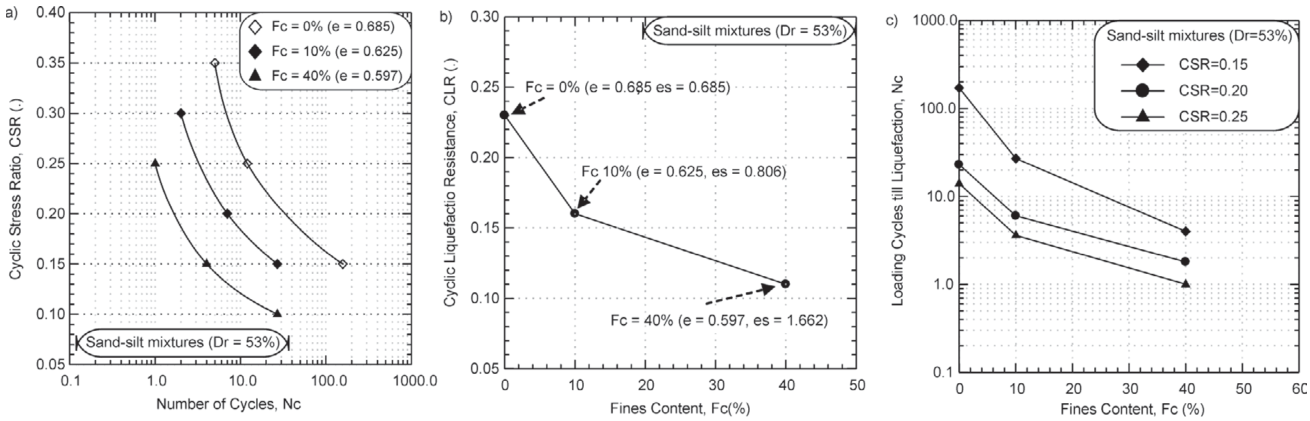


**Figure 13:** Effect of fines on the liquefaction potential of the sand-silt mixtures ($\sigma_3'$ = 100 kPa, $D_r$ = 53%).

with the increase in the fines content. 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).

### 4.3. Effect of the Relative Density

Undrained cyclic tests were performed on Chlef sand–silt mixtures (Fc = 5%) for two relative densities ($D_r$ = 17% and 62%). For each density, we have varied the loading amplitude in order to draw the liquefaction potential curve. The tests were carried out for amplitudes $q_m$ = 30, 50 and 70 kPa. Liquefaction was reached quickly for the higher amplitudes: after 2 and 3 cycles respectively for loading amplitudes of $q_m$ = 70 and 50 kPa. Liquefaction under loading amplitude $q_m$ = 30 kPa and relative density $D_r$ = 17% was reached at 24 cycles. For the tests carried out with a relative density of $D_r$ = 62% and for the same loading amplitudes, liquefaction occurred at 5, 13 and 167 cycles.

Figure 14 summarizes all the test results. Figure 14a illustrates the influence of relative density on the liquefaction potential of Chlef sand-silt mixture ($F_c$ = 5%). It shows clearly that the increase in the relative density leads to an increase in the liquefaction resistance of the Chlef sand-silt mixture. Figure 14b shows the influence of the gross void ratio on the liquefaction resistance of the sand-silt mixtures. This figure shows clearly that liquefaction resistance decreases with the increase in the gross void ratio and the loading amplitude. We 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.

## 5. CONCLUSION

A series of undrained monotonic and cyclic triaxial tests were carried out on silty sand collected from liquefied sites on the 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 triaxial tests performed with two relative densities ($D_r$ = 17% and 91%) showed a contractive behaviour of the sand-silt mixtures samples at the initial confining pressure in the gross void ratio range tested. The undrained residual shear strength decreases as the gross void ratio decreases and the fines content increases up to 30%.

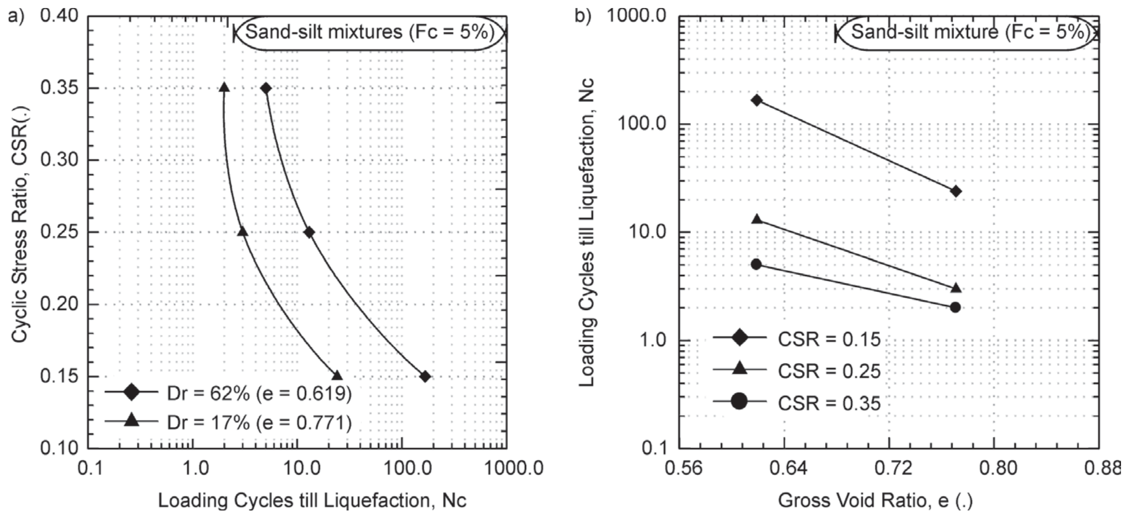


**Figure 14:** Effect of relative density on the liquefaction potential of Chlef sand-silt mixture ($\sigma_3'$=100 kPa, Fc =5%).

Beyond this, it decreases with increasing gross void ratio and fines content. The undrained residual shear strength decreases linearly with increase in the fines content and the intergranular 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 triaxial tests show that the liquefaction resistance of the sand-silt mixtures decreases with the increase of the fines content, and the samples sheared with higher level loading (CSR= 0.25) are more vulnerable to liquefaction than those sheared with a 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 gross void ratio and the loading amplitude. We note that the reduction in the liquefaction resistance of the sand-silt mixture becomes very marked for the smaller cyclic stress ratios CSR = 0.15 and 0.25.

These results confirm those found during monotonic tests showing that the increase in the fines content leads to an increase of the contractancy phase inducing a reduction in the liquefaction potential when the fines content ($F_c$) increases. For the sand-silt mixtures, the presence of fines increases the phase of contractancy resulting in a significant decline of the liquefaction potential. It should be noted that for the studied amplitude ($q_m$= 30 kPa), the increase in the fines content leads to an acceleration of liquefaction.

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