EXPERIMENTAL STUDY OF UNDRAINED SHEAR STRENGTH OF SILTY SAND: EFFECT OF GRADING CHARACTERISTICS

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Résumé *: The liquefaction susceptibility of saturated medium sand-silt mixture samples is evaluated by monotonic undrained triaxial laboratory tests that were carried out on reconstituted specimens at various relative densities (Dr = 17 and 91%) and a constant confining pressure (σ3'= 100 kPa). The test results were used to conclude on the effect of grading characteristics and other parameters on the liquefaction resistance of the sand-silt mixtures. The test results indicate that the undrained shear strength at the peak can be correlated to the coefficient of uniformity (Cu) and the average diameter (D50). Indeed, they decrease linearly with the increase of the uniformity coefficient and decrease of the average diameter. It is found that a relationship between the static liquefaction resistance and any of the diameters (D10 or D50) and the coefficient of uniformity (Cu) would be more realistic than to build a relation between the coefficient of gradation (Cc) and the static liquefaction resistance.*

KEYWORDS: *liquefaction resistance, silty sand, fines content, coefficient of uniformity, gross void ratio.*

1. Introduction

Several earthquakes occurred in the region of Chlef located in northern Algeria last century. The most disastrous earthquake with Richter Magnitude, M_L = 7.2, corresponding to a Surface Wave Magnitude, Ms = 7.3 hit Chlef City and surrounding areas on October 10, 1980. This event inflicted important damages of varying extents to a large number of small to moderate size civil and hydraulic structures in the vicinity of the earthquake epicenter. Some of the distress was due to the liquefaction of saturated alluvium in foundation. Liquefaction associated ground deformations such as lateral spreading, flow failures, ground fissures and subsidence, sand boils, and slope failures were observed. The earthquake epicenter of the main shock was located 12 km in the east region of Chlef City (210 km west of Algiers) at latitude 36.143° N and longitude 1.413° E with a focal depth of about 10 km. The approximate duration of the quake was between 35 and 40 sec. The event, commonly referred to as the Chlef Earthquake, was among the most disastrous earthquakes that have affected the northern region of Algeria. 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 Chlef river banks great masses of sandy soils were ejected on to the ground surface level. A major damage to certain civil and hydraulic structures (earthdams, embankments, bridges, slopes and buildings) was caused by this earthquake.

The factors affecting the undrained shear (liquefaction resistance) strength of sands and silty sands under monotonic and cyclic loading conditions have been extensively studied by Zlatovic and Ishihara (1995), Lade and Yamamuro (1997), Thevanayagam et al (1997), Thevanayagam (1998), Yamamuro and Lade (1998), Amini and Qi (2000), Naeini (2001), Naeini and Baziar (2004), Sharafi and Baziar (2010), Belkhatir et al. (2010). The influence of several other parameters such as the confining pressure, the relative density, the degree of saturation, the sample preparation method, the overconsolidation ratio and the stress ratio are well understood. However, the influence of other parameters such as the fines content, the structure, size and shape of the grains are incomplete and requires further investigation Moreover, limited studies in the literature have substantially evaluated the influence of particle gradation alone Yilmaz et al. (2008). Chang et al. (1982) reported that cyclic liquefaction resistance of a clean sand was strongly affected by the mean size, D_{50}, and the uniformity coefficient, Cu, provided that $D_{50} \langle$ 0.23 mm. However, the individual influences of D_{50} and Cu were not isolated. Vaid et al. (1991) examined the influence of Cu by testing three clean sands with the same mineralogy, D_{50}. They concluded that the cyclic liquefaction resistance of clean sand increases with Cu at low relative density and the tendency was reversed at high relative density.

2. Laboratory testing program

2.1. Materials properties

Silty sand samples were collected from liquefied layer of the deposit areas at a depth of 6.0 m (Figure 1) close to the Chlef earthquake epicenter (October 10th, 1980). Figure 2 shows craters of liquefied ground on banks of Chlef River. Figure 3 illustrates typical subsidence location of the liquefied soil and sample collection. The tests were conducted on the mixtures of Chlef sand and silt. Chlef sand was mixed with 0 to 50% silt to get different fines contents. The index properties of the sand, sand-silt mixtures and silt used in this laboratory research work are presented in Table 1. The grain size distribution curves of the tested sand-silt mixtures are shown in Figure 4. 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 given in Figure 5. According to this Figure the different indices decrease with the increase of the fines content until F_c = 30%, then, they increase with further fines content increase. We note that the void ratios of samples have effectively changed after saturation and consolidation phases. The variation of e_{max} versus e_{min} is illustrated in Figure 6. It can be seen from this Figure that the correlation between the minimum and maximum void ratios of the sand-silt mixture samples is quite similar to that of Cubrinovski and Ishihara (2002).

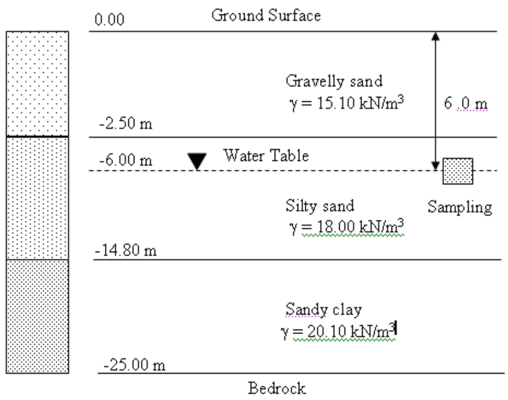
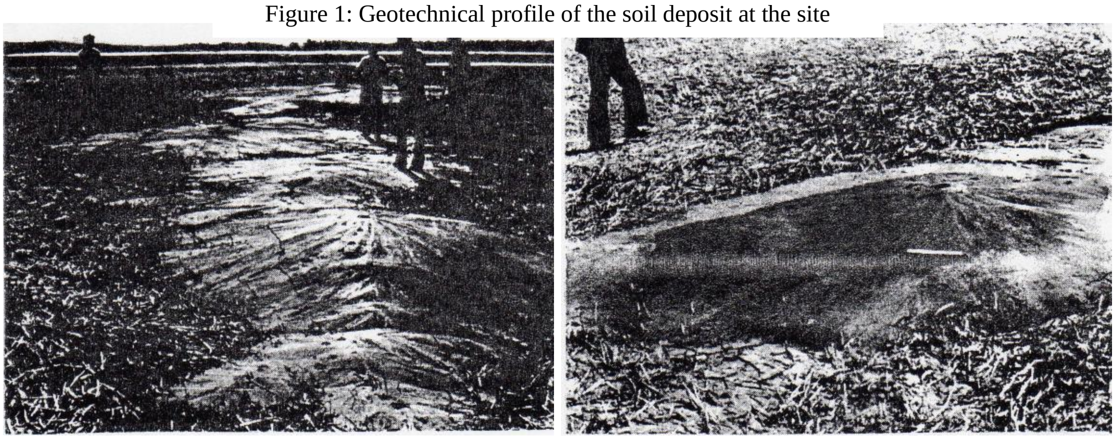


Figure 1: Geotechnical profile of the soil deposit at the site



(a) (b)

Figure 2: Craters of liquefied soil on banks of the Chlef River

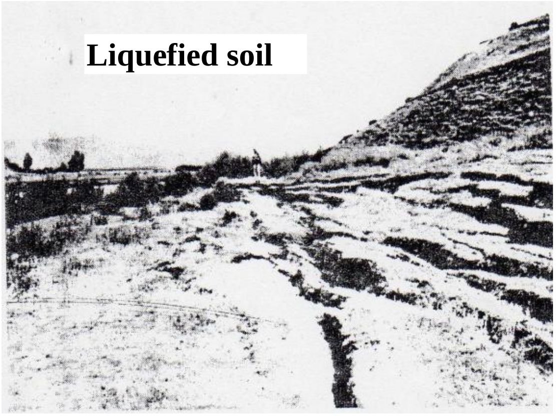


Figure 3: Subsidence of the Chlef River banks due to liquefaction

Table 1: Index properties of sand-silt mixtures

Material	Fc (%)	G_S	D_{10} (mm)	D_{50} (mm)	C_u	e_{min}	e_{max}	I_p (%)
Sand	0	2.680	0.22	0.68	3.36	0.535	0.854	-
Silty Sand	10	2.682	0.08	0.50	7.75	0.472	0.798	-
	20	2.684	0.038	0.43	15.26	0.431	0.748	-
	30	2.686	0.022	0.37	23.18	0.412	0.718	-
	40	2.688	0.015	0.29	27.33	0.478	0.732	-
	50	2.690	0.011	0.08	28.18	0.600	0.874	-
	60	2.692	-	-	-	0.657	1.007	-
Silt	100	2.70	-	-	-	0.72	1.420	5.0

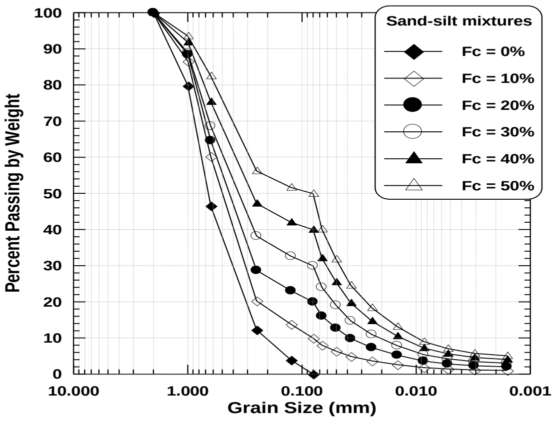


Figure 4: Grain size distribution curves of tested materials

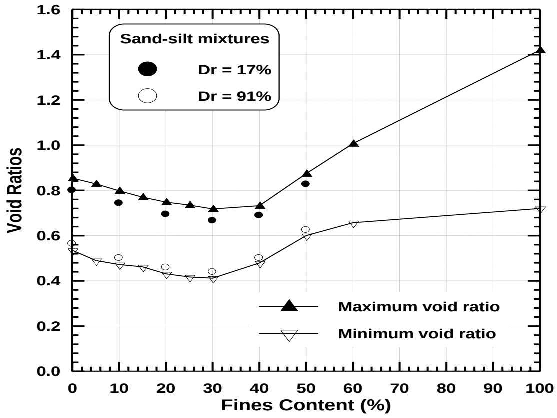


Figure 5: Void ratios Index of the sand-silt mixtures versus fines content (σ_3'=100 kPa)

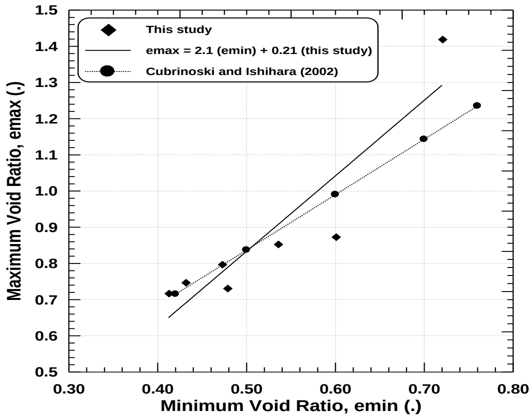


Figure 6: Maximum void ratio versus minimum void ratio of the sand-silt mixtures

2.2. Sample preparation

The dimensions of 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. All samples were prepared using seven layers. The resulting height to diameter ratio of 1 is kept constant. All samples were prepared by first estimating the dry weights of sand and silt needed for a desired proportion into the loose, medium dense and dense state (D_r = 12, 53 and 90%) using 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 Ladd (1978). After the specimen has been formed, the specimen cap is placed and sealed with O-rings, and a partial vacuum of 15 to 25 kPa is applied to the specimen to reduce the disturbances.

2.3. Sample saturation

Saturation was performed by purging the dry specimen with carbon dioxide for approximately 20 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.

2.4. Sample consolidation

When samples were fully saturated, they were subjected to consolidation. During consolidation the difference between all-around pressure and back pressure was set so that for each sample the effective consolidation pressure was fixed as 100 kPa.

2.5. Shear loading.

All undrained triaxial tests for this study were carried out at a constant strain rate of 0.167% per minute, 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.

Figure 7 shows the variation of the gross void ratio versus the effective diameter and fines content at the initial relative density (D_r = 17%, 91%). As could be seen in this Figure, the initial fabric void ratio (e) decreases almost linearly with the decrease of the effective diameter and increase of the fines content until the value of 30% and then it increases significantly with the decrease of the effective diameter and increase of the fines content.

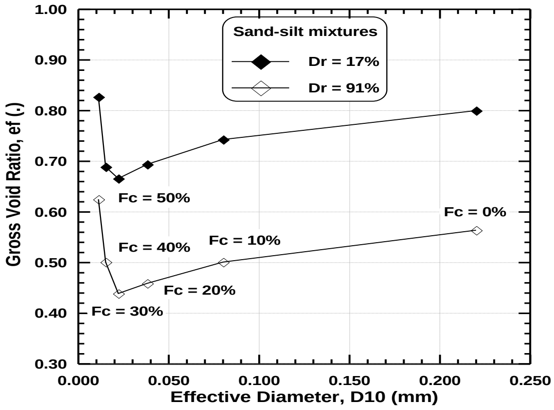


Figure 7: Gross void ratio versus effective diameter and fines content (σ_3'=100 kPa)

Figure 8 shows the variation of the gross void ratio versus the average diameter and fines content at the initial relative density (D_r = 17%, 91%). The results indicate that the initial gross void ratio (e) decreases almost linearly with the decrease of the average diameter and increase of the fines content until the value of 30% and then it increases with the same manner with the decrease of the average diameter and increase of the fines content.

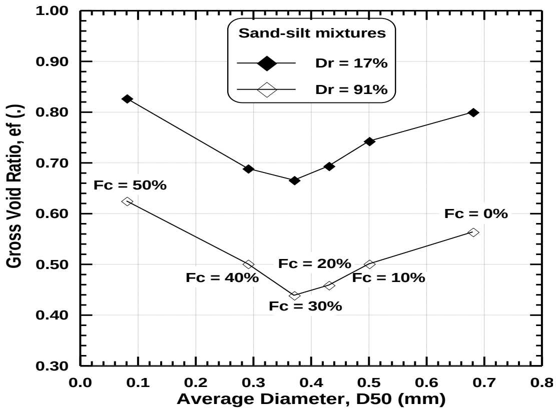


Figure 8: Gross void ratio versus average diameter and fines content (σ_3'=100 kPa)

Figure 9 shows the variation of the initial gross void ratio versus the coefficient of uniformity and fines content at the initial relative density (Dr = 17%, 91%). As can be seen from this Figure, the initial gross void ratio (e) decreases moderately with the increase of the coefficient of uniformity and fines content until the value of 30% and then it increases significantly from 40% to 50% fines content for both densities.

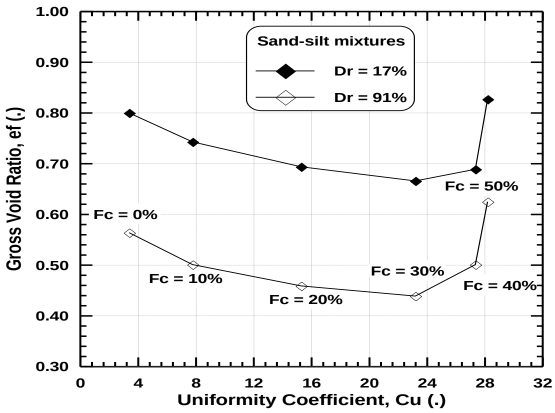
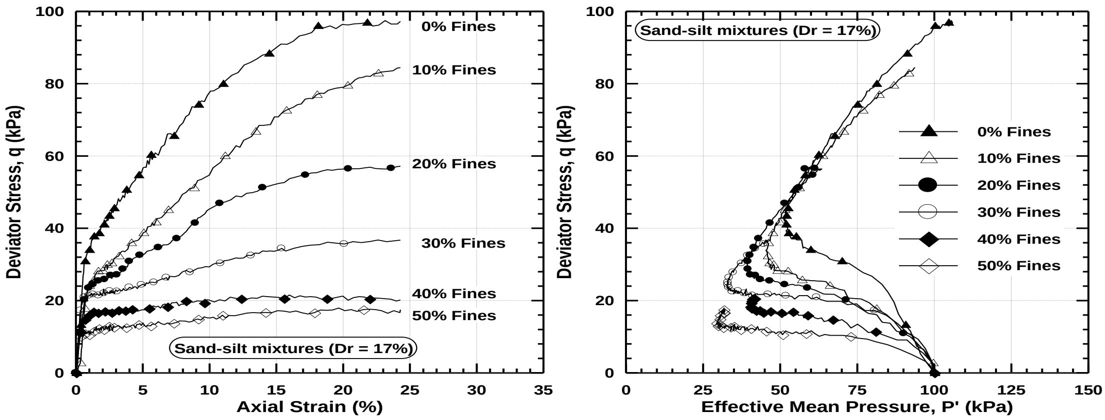


Figure 9: Gross void ratio versus uniformity coefficient and fines content
(σ_3'=100 kPa)

3. Monotonic tests results

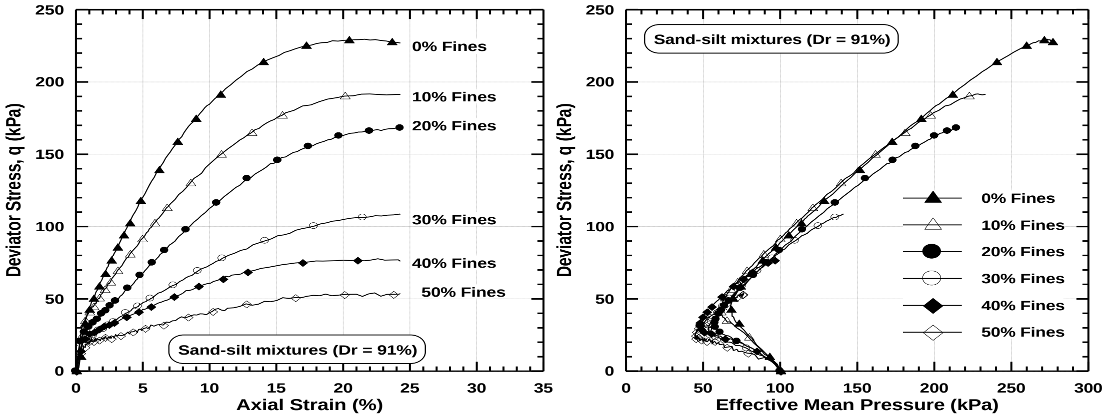
3.1. Undrained compression loading tests

Figures 10 and 11 illustrate the undrained monotonic compression triaxial test results carried out under an initial confining pressure of 100 kPa for different fines contents ranging from 0 to 50% and two initial relative densities (Dr = 17%, 91%). As can be seen from the Figures the increase of the amount of fines induces a decrease of the sand-silt mixture liquefaction resistance This decrease results from the role of the fines to increase 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 it is illustrated by Figures 10a and 11a. The stress path in the (p', q) plane shows clearly the role of the fines in the decrease of the average effective pressure and the maximum deviatoric stress (Figures 10b and 11b). 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 and Verdugo (1985).



(a) (b)

Figure 10: Undrained monotonic response of the sand-silt mixtures
(σ_3'=100 kPa, Dr = 17%)



(a) (b)

Figure 11: Undrained monotonic response of the sand-silt mixtures
(σ_3'=100 kPa, D_r = 91%)

Table 2 presents the summary of the undrained monotonic compression triaxial tests

Table 2: Summary of monotonic compression triaxial tests results

Test No	Material	F_C(%)	D_r(%)	e	e_f	e_{sf}	Q_{peak}(kPa)
1	Sand	0	17	0.815	0.800	0.800	97.50
2			91	0.567	0.564	0.564	229.60
3	Silty sand	10	17	0.760	0.743	0.937	84.50
4			91	0.505	0.501	0.668	191.70
5		20	17	0.710	0.694	1.118	57.20
6			91	0.463	0.459	0.824	168.30
7		30	17	0.680	0.666	1.380	36.80
8			91	0.443	0.439	1.056	108.60
9		40	17	0.700	0.689	1.815	20.80
10			91	0.503	0.501	1.502	77.70
11		50	17	0.840	0.827	2.654	17.90
12			91	0.630	0.625	2.250	54.10

e : initial gross void ratio
e_f : post-consolidation gross void ratio
e_{sf} : post-consolidation intergranular void ratio
Dr : post-consolidation relative density
Q_{peak} : Peak strength

4. Effect of grading characteristics on the peak strength

4.1. The effective diameter D_{10}

Figure 12 illustrates the variation of the peak strength (Q_{peak}) with the effective diameter (D_{10}) and fines content. It is clear from this that the peak strength decreases almost linearly as the effective diameter decreases and fines content increases for the loose and dense state of the specimen (Dr = 17% and 91%) up to 20% for the dense samples and 10% for the loose samples followed by an important decline of the sand-silt mixture strength. We notice that the undrained shear strength at the peak converges towards the same values for smaller effective diameter values for both relative densities.

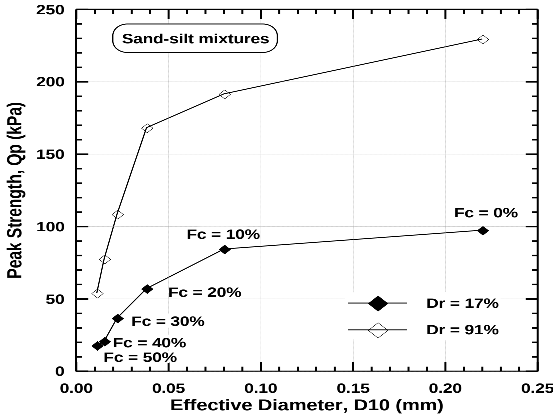


Figure 12: Peak strength versus effective diameter and fines content
(σ_3'=100 kPa)

4.2. The average diameter D_{50}

Figure 13 shows the variation of the undrained shear strength at the peak with the average diameter and fines content. It can be seen from this Figure that the undrained shear strength (Q_{peak}) of the sand-silt mixtures decreases linearly with the decrease of the average diameter (D_{10}) and increase of the fines content and converges towards a unique value of the peak strength for higher fines content for the two initial relative densities (D_r = 17% and 91%). 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 shear strength at the peak which is a function of the average diameter (D_{50}):

Q_{peak}= -7 + 152(D_{50}) for D_r = 17%
Q_{peak}= 11 + 325(D_{50}) for D_r = 91%

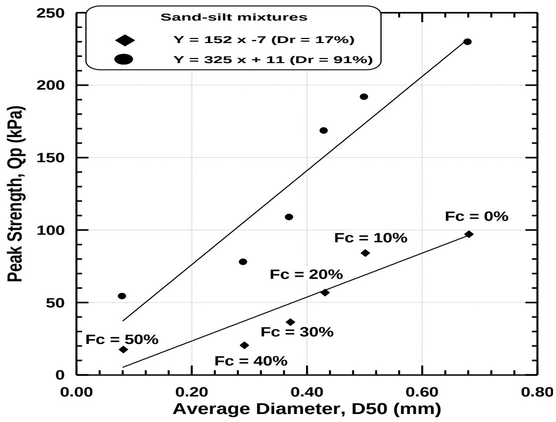


Figure 13: Peak strength versus average diameter and fines content
(σ_3'=100 kPa)

4.3. The coefficient of uniformity

Figure 14 shows the undrained shear strength at the peak (Q_{peak}) versus the uniformity coefficient (Cu). It is clear from this Figure that the peak strength decreases in a linear manner as the coefficient of uniformity and fines content increase. It seems that the decrease of the undrained strength at the peak (Q_{peak}) due to the amount of fines is related to contractant behaviour of the sand-silt mixture samples. Moreover, the undrained shear strength range for both relative densities decreases with the increase of the uniformity coefficient and fines content. 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 shear strength at the peak which is a function of the coefficient of uniformity (Cu):

Q_{peak}= 108 – 3.19(Cu) for D_r = 17%
Q_{peak}= 253 -6.53(Cu) for D_r = 91%

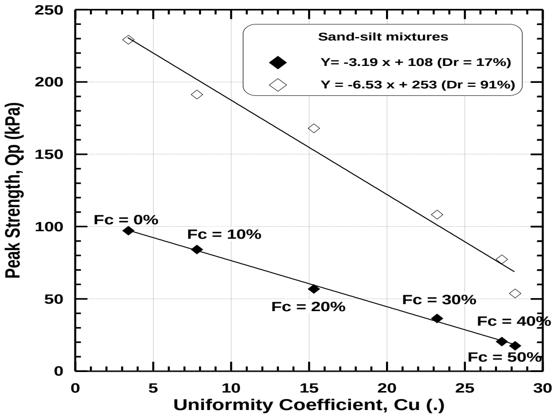


Figure 14: Peak strength versus uniformity coefficient and fines content (σ_3'=100 kPa)

4.4. The relative density

Figure 15 shows the variation of the undrained shear strength at the peak (Q_{peak}) with the initial relative density (D_r) at various uniformity coefficients. It is clear from this Figure that an increase in the relative density results in an increase in the peak strength at a given coefficient of uniformity. Thevanayagam et al. (1997) and Sitharam et al. (2004) report similar behaviour of increasing undrained shear strength with increasing relative density. It can be concluded from the results of this laboratory investigation that there is a significant decrease in the undrained shear strength at the peak with the increase of the coefficient of uniformity for both relative densities, but there is a significant increase in the undrained shear strength at the peak with the increase of the relative density. Moreover, the slope of the peak strength line is very pronounced for smaller uniformity coefficients (Cu = 3.36, 7.75 and 15.26) compared to higher uniformity coefficients (Cu = 23.18, 27.33 and 28.18). The aspect of the present research confirms the experimental work tendency reported by Ishihara (1993) on Tia Juana silty sand, Baziar and Dobry (1995) on silty sands retrieved from the Lower San Fernando Dam, and by Naeini and Baziar (2004) on Adebil sand with different fines contents.

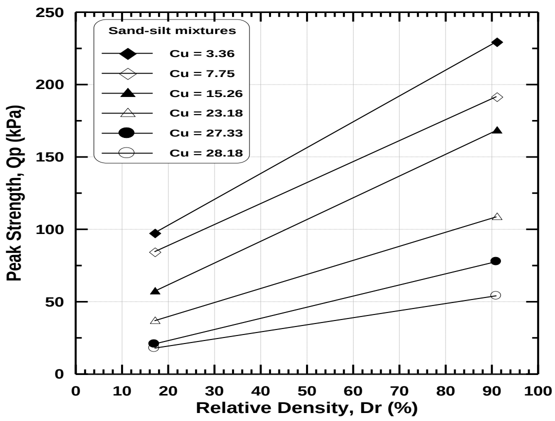


Figure 15: Peak strength versus initial relative density at various uniformity coefficients

(σ_3'=100 kPa)

5. Conclusion

A series of undrained monotonic triaxial tests were carried out on sand-silt mixture samples retrieved from liquefied sites at Chlef River banks (Algeria). The effect of grading characteristics and other parameters was studied. In the light of the experimental evidence, the following concluding remarks can be drawn:

Undrained monotonic triaxial compression tests performed with two relative densities (Dr = 17% and 91%) showed a contractive behaviour of the sand-silt mixture samples subjected to the initial confining pressure in the gross void ratio range tested. The undrained shear strength at the peak decreases as the coefficient of uniformity increases and the average diameter decreases and fines content increases up to

50%. The peak strength decreases linearly with the increase of the coefficient of uniformity and decrease of the average diameter. Moreover, the relative density influences significantly the undrained shear strength at the peak. It can be noticed that an increase in the relative density resulted in an increase of the peak strength. The slope of the peak strength line is very pronounced for smaller uniformity coefficients (Cu = 3.36, 7.75 and 15.26) compared to higher uniformity coefficients (Cu = 23.18, 27.33 and 28.18).

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