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EXPERIMENTAL STUDY OF MECHANICAL INSTABILITY OF SANDY SOILS

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ABSTRACT

The liquefaction is an important phenomenon of mechanical instability of sandy soil leading to a catastrophic failure; this mechanism has been constantly revised to include new parameters that may control the mechanical instability under static or dynamic loadings. Such instability may be manifested by severe damage or large displacement. Civil engineer considers the important parameter of the shear strength for better design of structures founded on soil vulnerable. The accurate determination of the critical or residual shear strength has been a major challenge to geotechnical engineering. Therefore, it is essential to determine the main parameters that may significantly control the shear strength and to provide a broad understanding of soil behavior under undrained conditions. In this work, the main objective is to experimentally analyze the mechanical behavior under undrained conditions reconstituted loose and medium dense specimens of terrigenous silica sand with different almond content. This present paper is an attempt to describe experimentally the mechanical behavior of soils with low plastic silty sand and represent the variation of the critical shear strength based on different parameters, the density state as well as the equivalent void ratio. The last characteristic seems rightly the vulnerability of the material to instability.

Keywords: sandy soils, liquefaction, void ratio, shears strength, density state, vulnerability.

1. INTRODUCTION

Sandy soils may be susceptible to mechanical instability due to static or dynamic loadings in undrained conditions. Facing such instability phenomena, it is important to characterize the mechanical behavior of this granular matter as an engineering construction material.

If the shear strength of a soil vulnerable to liquefaction drops below the existing initial static load, liquefaction phenomenon may occur and consequently lead to sudden catastrophic failure. Over the world, many dramatic cases of static liquefaction flow failure have been reported by Olson and Stark (2002 and 2003). Several liquefaction flow failures are triggered by either static or dynamic loadings. An accurate determination of the liquefied shear strength is of paramount importance for the design task of soil structures such as earth dams, shallow foundations, bridge supports, as well as an essential parameter for soil densification process in order to avoid such risk of soil instability manifested by failure or large mass displacement such as settlement.

A thorough understanding of soil liquefaction is continuously being revised to include new soil parameters controlling liquefaction phenomenon as long as new in-field and laboratory observations are revealed. Recent in-field post-liquefaction events had significantly oriented the research trend in investigating instability of natural sandy soil deposits. The effect of low plastic fines on mechanical behavior of sand matrix soil is not yet fully controlled. In this perspective, present challenge is to improve fundamental research on the mechanical behavior of soils prone to instability in order to obtain consensual framework to predict behavior of such soils.

Controversial laboratory research findings on the role of low plastic fines in sand matrix state; that such fines content may either increase (Georginnou V N *and al,* 1990; Zlatovic S and Ishihara K, 1997) or decrease (Yamamuro J A and Lade P V, 1998; Amini F and Qi G Z, 2000) liquefaction vulnerability. Other researchers revealed that fine content decrease liquefaction resistance until some limiting fine content value is reached (Koester JP *and al*, 1994; Lo S R *and al*, 2010). Such contributions tried to give a physical interpretation to the influence of fines onto liquefaction behavior of sand matrix soils. Even though, the well-known liquefaction susceptibility identification, the Chinese criteria (Wang W S, 1979) is largely applied in many codes of practices throughout the world, such rules are mainly based on in-field observations. This criterion has been reviewed many times and it still forms the basic principles and regulations for many codes of practices. Nevertheless, new recent evidences revealed deviation and inadequacy due to the presence of fines in sand matrix, essentially low plastic fines. Because of discrepancies in the research results in laboratory and in-field post-liquefaction events, many researchers agreed that such guidelines should be discontinued and should be reviewed in a general context (Prakash S and V K Puri, 2010).

In this work, an empirical approach for evaluating the undrained critical shear strength of silty sand using the equivalent void ratio is here attempted. Our main aim in this study is focused on analyzing the link between the

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contribution of the fine content and the soil matrix. To achieve this objective, a series of monotonic undrained triaxial tests were performed on reconstituted saturated sand-silt mixture specimens with fine fraction ranging from 0% to 50% with two combinations of initial relative density states (15% and 45%) at confining effective mean stress (100kPa).

2. DESCRIPTION OF THE SITE AND LOCATION OF SAMPLING OF POINTS

The soil under study was collected from the coastal region of Oran in Algeria. This region is located on the north western part of Algeria bordering the Mediterranean Sea. As well, it is situated on the near field of the ruptured euro-african plate. Such region is vulnerable to an intense seismic activity. Other risk of collapse is greater for the city of Oran especially for the sea front is caused by the water trickling of Ain el Ruina. The sample is at the port of Oran on the same extension of this water trickling and mainly composed of silty sand.

As part of the development and expansion of the Oran dock, a water tank with a capacity of 600m^3 has been planned with the following characteristics:

- Height of 14 meters, diameter of 07 meters.
- GPS coordinates: 35 ° 42'25.16 "N 0 ° 38'28.82" W

General view of the region is shown on Figure-1.

Figure-1. Localization and view of the study region.

The studied soils consist predominantly of sandy soils with low silt content. This fraction of silt exists in variable proportions with low plasticity index.

Sampling points are shown on the geotechnical profile in Figure-2.

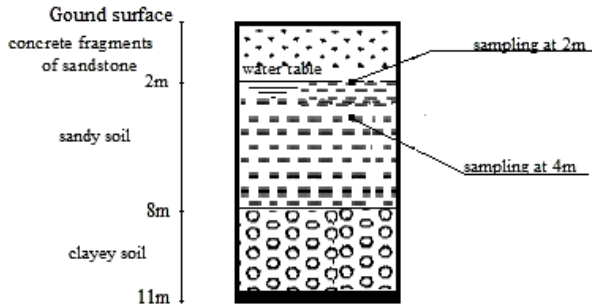

Figure-2. Soil geotechnical profile and position of samples.

3. EXPERIMENTAL STUDY

A series of undrained triaxial tests were conducted on reconstituted sand-silt mixtures specimens. Sandy soil was collected from coastal region of Oran (north western Algeria) at two different depths. Soil under study consists of terrigenous silica sand composed mainly of quartz and feldspar. In our experimental program, sand and fine particles were separated from each other and then mixed in a specified fraction to form specimen to test. Since most static liquefaction and earthquake induced liquefaction have occurred in silty sand and sandy silt, different silt contents ranging from 0% to 50% were mixed to sand of the Oran dock and then used in the tests. As well, liquefaction vulnerability is highly affected by the relative density of the soil (Yamamuro J A and Kelly M C, 2001; Maheshwari B K and Patel A K, 2010). In this work, loose and medium density states have been analyzed as well as the soil under consideration has the following geotechnical properties, as shown in Table-1.

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Table-1. Geotechnical properties of soil samples.

	$F_c(\%)$	G_s	e_{min}	e_{max}	D_{10}	D_{50}	D_{30}	D_{60}	C_u	C_c
Clean Sand	0	2,66	0,69	1,02	0,21	0,33	0,29	0,36	1,70	1,07
Silty Sand	5	2,66	0,65	0,99	0,18	-	0,28	0,36	1,98	1,19
Silty Sand	10	2,66	0,61	0,96	0,09	-	0,26	0,35	4,15	2,32
Silty Sand	15	2,66	0,52	0,93	0,01	-	0,25	0,35	31,36	16,08
Silty Sand	20	2,66	0,48	0,91	0,01	-	0,23	0,34	37,56	16,64
Silty Sand	30	2,65	0,36	0,88	0,01	-	0,09	0,32	40,13	3,30
Silty Sand	40	2,65	0,36	0,96	0,01	-	0,01	0,30	42,43	0,06
Silty Sand	50	2,65	0,40	1,15	0,01	-	0,01	0,26	42,67	0,07
Silty	100	2,64	0,77	1,76	0,00	0,01	0,01	0,01	2,25	1,36

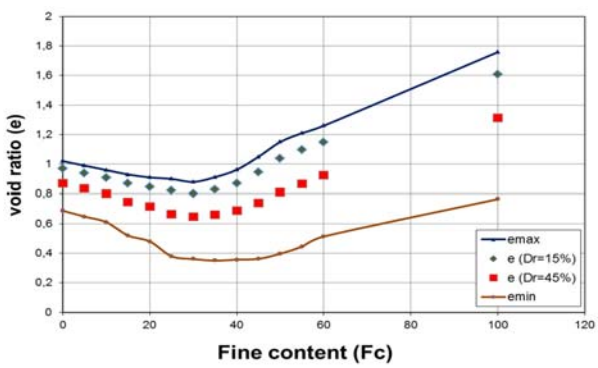

Figure-3. Limiting void ratios.

The grain size distribution curves for the collected sand and silt under study are shown separately in Figure-4. D_i corresponds to the soil diameter at which (i%) of the soil weight is finer. The plasticity index of the silt I_p is 6%. According to ASTM D2487-11(2011) classification, the sand under study is poorly graded (SP), while the silt is inorganic (ML) and of low plasticity.

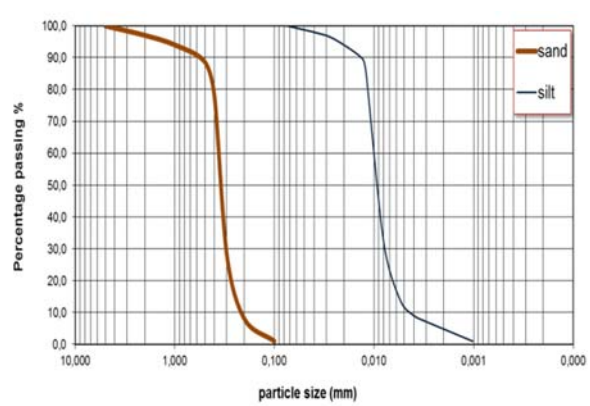

Figure-4. Grain size distribution curves.

e_{min} is the minimum void ratio (densest density state) and e_{max} is the maximum void ratio (loosest density state). From Table-1, it can be seen that the variation of both indices follows the same trend. The two indices decrease with the increase of the fines content till around (25% - 35%) then they increase after this value (Figure-3). The same observation is related by Missoum *and al*. (2013) for Chlef (Algeria) sandy soil.

3.1 Experimental tests

3.1.1. Preparation

The mechanical instability of soils may be strongly dependent on the sample preparation methods (Ladd R S, 1974; Mulilis J P, 1977). Several sample preparation methods have been elaborated for use in the laboratory such as dry or wet funnel pluviation, moist tamping and water sedimentation. Samples under testing should reproduce in-situ soil conditions. Therefore, the choice of an adequate sample preparation method is fundamental to determine the instability potential of sandy soils. The sedimentation deposition method tends to reproduce well field performance as shown by Vaid *and al* (1999).

3.1.2. Soil sample preparation and shearing

The studied samples are cylindrical in shape of 70mm in diameter D and 140mm in height H. According to the required relative density, mass of sand-silt mixture to be put inside the mould is determined previously. The relative density of reconstituted sample is defined as:

$$D_r = \frac{(e_{max} - e)}{(e_{max} - e_{min})} \quad (1)$$

Where, e is the global void ratio

Then, the samples have been purged by carbon dioxide gas for more than 30 minutes, and saturated by de-aired water. The saturation state is controlled by

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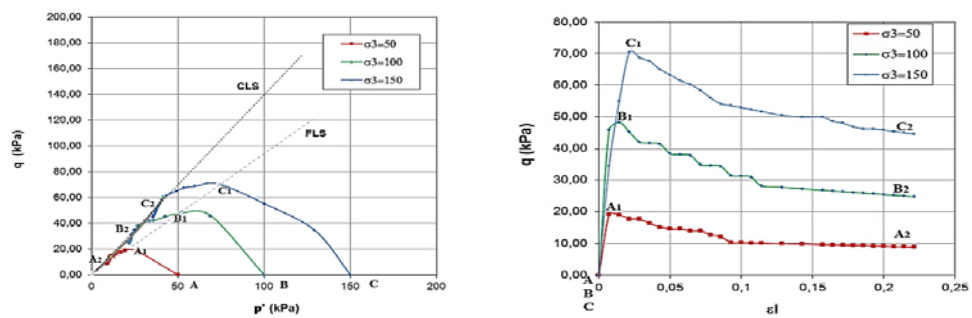
Skempton's pore water pressure parameter *B*. According to ASTM D 4767-02 (2004), samples can be considered fully saturated if B is at least or greater than 0.95. In all tests, this condition was fulfilled. In the objective to achieve full saturation state, a backpressure of 100 kPa has been applied during the test experiments.

All samples were isotropically consolidated at mean effective stress of 100 kPa, and then subjected to undrained monotonic triaxial loading. In the objective to stabilize the pore water pressure development throughout all the samples, a constant strain rate of 2.5% per hour was applied during all the tests. All the tests were carried up to 20 % axial strain.

3.2. Critical shear strength in the undrained study

At different levels of mean effective stresses the sand-silt mixture specimens are susceptible to liquefy after saturation. In Figure-5, we can see the mechanical behavior in undrained triaxial test. The samples are compressed isotropically at three level of effective mean stress and represented at Points A, B and C. During this process, at the positions A_1, B_1 and C_1 the soil sample reaches peak shear strength S_u (yield strength) Vaid Y P and Chern J C (1983), then the shear strength is reduced causing limited liquefaction or mechanical instability manifested by fabric disintegration, or liquefaction phenomena if shear strength falls drastically. The flow liquefaction susceptibility line (FLS) is represented by a dashed line in Figure-5(a).



(a) Stress paths (*q, p'*) **(b)** variation of *q* vs ε_l

Figure-5. Undrained triaxial compression test results of loose sand.

The points A_2, B_2 and C_2 define the shear strength at critical state S_{ucr}, in the framework of critical state soil mechanics theory, the following relationship can be written:

$$q_{CSL} = M.p'_{CSL} \quad (2)$$

With *M* is the slope of critical state line

In the case of triaxial tests and according to Schofield and Wroth (1968), the following expression can be written;

$$Sin\ \phi_S = \frac{3.M}{6+M} \quad (3)$$

Where: q_{csl}, p'_{csl} and Φ_s indicate the deviatoric stress (σ'_1-σ'_2), the effective mean principal stress $\frac{(\sigma'_1 + 2\sigma'_3)}{3}$, and the mobilized angle of inter-particle friction at the critical state, respectively. The critical shear strength may be evaluated by equation 4:

$$S_{ucr} = \left(\frac{q_{CSL}}{2}\right).\cos\phi_S \quad (4)$$

4. RESULTS

The results of the undrained monotonic compression triaxial tests carried out for different fines content ranging from 0 to 50 % of mean effective pressure of 100kPa within two separate density ranges (D_r =15 and 45 %) are shown on Figure-6. During the tests, the stress paths (*q, p'*) are recorded and represented graphically as well as the axial strain versus the deviatoric stress *q*.

It can be clearly noticed that an increase in the added fines from 0% to 30% leads to a decrease in the deviatoric stress *q*. This decrease comes 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 mean effective pressure and as well as a decrease in the peak deviatoric stress of the mixtures (Koester JP *and al*, 1994; Kokusho T *and al*, 2014). The stress path in the plane (*q, p'*) shows clearly the role of the fines in the decrease of both the average effective pressure and the maximum deviatoric stress (Figure-6c) and in this case, the effect of fines on the undrained behavior of the mixtures may be considered not participating in the force chain of the mixtures (Missoum H *and al*, 2013; Naeini, S A *and al*, 2004). Several laboratory studies reported the same observations and that the undrained shear resistance may be reduced with the increase of fine contents till a threshold value of fine content (Shen C K *and al*, 1977; Sladen J *and al*, 1985; Troncoso J H *and al*, 1985).

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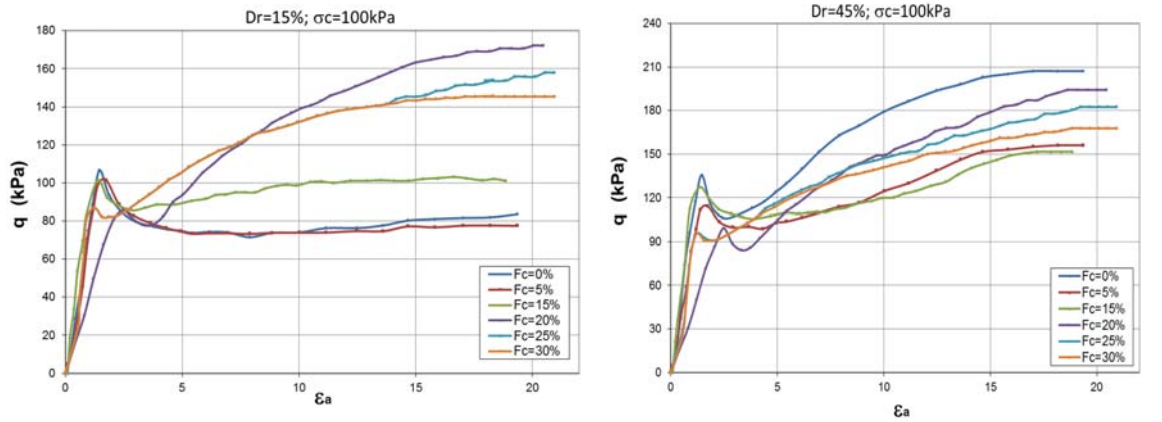
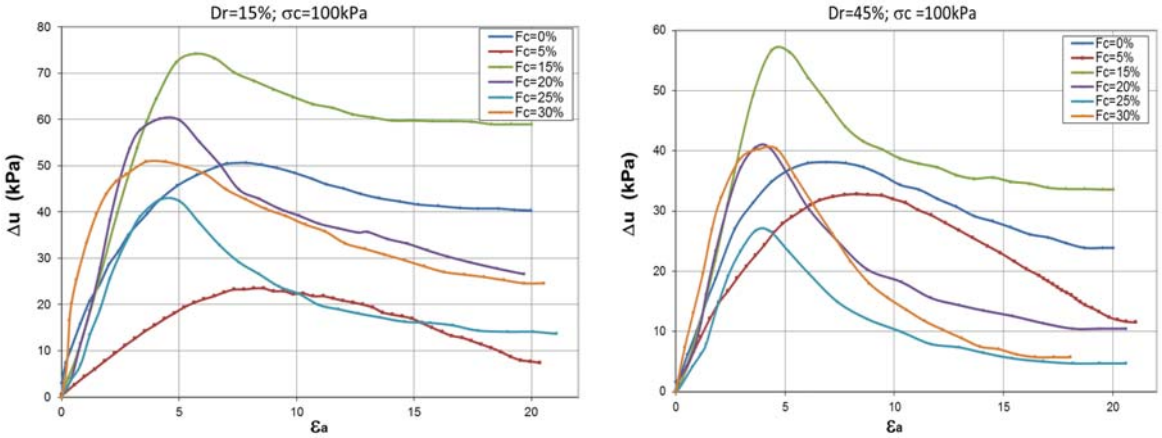
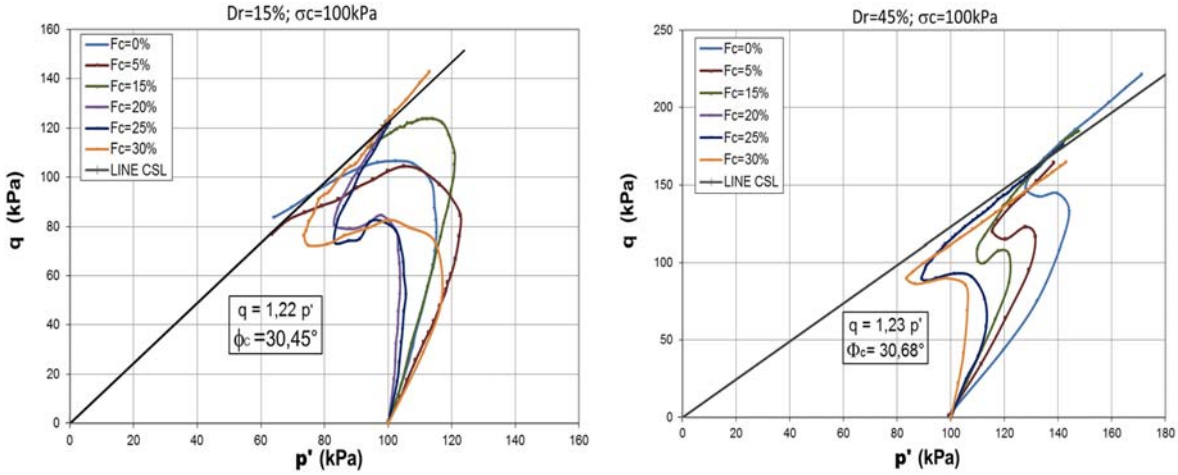


a) Evolution of deviatoric stress vs axial strain



b) Excess pore water pressure vs axial strain ($\Delta u, \varepsilon_a$)



c) Deviatoric stress vs mean effective pressure (q, p')

Figure-6. Undrained triaxial compression response for both relative densities.

From the Figure-6, it can be noticed that specimens show the same behavior under the same initial relative density and the same confining effective mean stress. The first phase of the stress paths represents the attainment of a peak deviatoric stress at small axial strain which is followed by a loss of shear strength until residual or critical shear strength is attained with no further decrease in strength. This type of behavior may be

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interpreted by a strain softening. The fast decline in shear strength and consequent generation in pore water pressure may be associated to the soil fabric disintegration. This process can be manifested by a limited liquefaction.

Within the range of initial relative densities under this study, the peak deviatoric stress increases with the increase in confining effective mean stress for all reconstituted specimens under investigation.

The samples exhibit dilation when the fine fraction is over 30 % Wang W S (1979), there is a continuous increase in the deviatoric stress without a loss in shear strength. When the fine fraction is over 30%, samples exhibit dilation, there is a continuous increase in the deviatoric stress without a loss in shear strength, and this increase is due to the role of the fine to increase the dilatancy in the soil mixtures. This shows that above 30% of fine particles contribute to the consolidation of the sample and reverse its behavior, it does not give instability or liquefaction state. Similar conclusions are reported by Thevanayagam and Mohan (2000); Bobei and Wanatowski (2009); Rahman and Lo (2012). Similar results were observed with the analysis of several recent earthquakes, Northridge (1994), Kocaeli (1999) and Chi-Chi (1999); soil liquefaction existed with more than 15% of fine particles content. The same observations were noted in the report of Kramer and Seed (1988) and Fourie and Tshabalala (2005).

4.1 Correlation between critical undrained shear resistance (S_{ucr}) and equivalent void ratio (e^*):

The equivalent void ratio (e^*) can be defined by equation (5). That is defined beyond a certain value of F_c (the threshold value); the fines of silt are directly involved in the sandy soil strength (Thevanayagam S *and al*, 2002). This parameter affects substantially the shear strength, it can be determined by:

$$e^* = \frac{(e + \alpha \ Fc)}{(1 - \alpha \ Fc)} \tag{5}$$

Where e^* is the equivalent void ratio and α represents the silt fraction that participates in the soil resistance. When α =1, the equivalent granular void ratio is reduced to the intergranular void ratio.

The obtained results from the tests show two distinct stress paths tendencies. Loose samples show amplified contractancy phase, while medium dense samples exhibit a contractive phase followed by a dilative phase in undrained conditions. The test results are summarized in Table-2.

Table-2. Experimental results for undrained tests.

Materials	*Fc* (%)	Initial relative density *Dr* (%)	Relative density after consolidation *Dr* (%)	Initial void ratio *e*	Equivalent void ratio *e**	Equivalent relative density *Dr** (%)	S_{ucr}/σ_c
Clean sand	0%	15	16,71	0,97	0,97	15,00	0,39
		45	45,91	0,87	0,87	45,00	0,51
Siltysand	5%	15	17,05	0,94	1,03	-3,01	0,37
		45	46,30	0,84	0,92	30,10	0,48
Siltysand	15%	15	15,76	0,91	1,08	-18,06	0,37
		45	46,98	0,80	0,97	15,05	0,43
Siltysand	20%	15	17,36	0,85	1,16	-42,14	0,37
		45	45,96	0,72	1,01	3,01	0,35
Siltysand	25%	15	16,72	0,82	1,19	-51,17	0,35
		45	46,05	0,68	1,02	0,00	0,35
Siltysand	30%	15	16,08	0,80	1,22	-60,20	0,34
		45	46,14	0,65	1,03	-3,01	0,34
Siltysand	40%	15	16,14	0,87	1,40	-114,37	-
		45	48,52	0,69	1,17	-45,15	-
Siltysand	50%	15	24,40	1,04	1,69	-201,66	-
		45	47,64	0,81	1,39	-111,36	-

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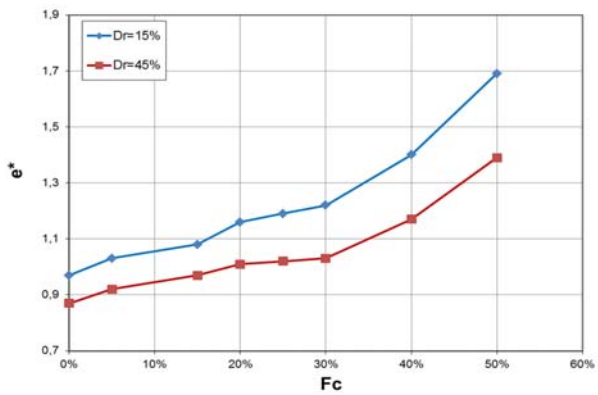

Figure-7. Variation of equivalent void ratio (e^*) versus fines content (F_c) (With D_r 15% and 45% at σ_c=100 kPa).

The parameter (e^*) is calculated from the correlation between soil properties (Ni Q *and al*, 2004; Yang S L *and al*, 2006). Rahman *and al*, (2008) analyze the experimental data of McGeary (1961), concluding that this parameter depends on both (F_c) and the size ratio (r). "r" is defined by the ratio of D_{50} (the size of fine particles of silt at a fraction of 50%) and D_{10} (the particle size at a fraction of 10% of clean sand):

$$r = \frac{D_{50(fines)}}{D_{10(clean-sand)}} \tag{6}$$

The correlation was proposed as follows:

$$\alpha = 1 - \left[1 - \exp(-\frac{0,3}{k} \cdot \frac{F_c}{F_{thre}})\right](r \cdot \frac{F_{thre}}{F_c})^r \tag{7}$$

Where: $k = 1 - r^{0.25}$ and F_{thre} is threshold fines content, which is characterized by the predominance of fines-controlled behavior. And, the value of F_{thre} is obtained by the localization of the state at which behavior is reversed with further increases in fines content. In this test, this transition point is located at 30% fines content (Figure-7). r and k were calculated to be 0, 0422 and 0, 546, respectively.

The variations in the equivalent void ratios versus fines content for the initial relative densities (D_r = 15% and 45%) are shown in Figure-8.

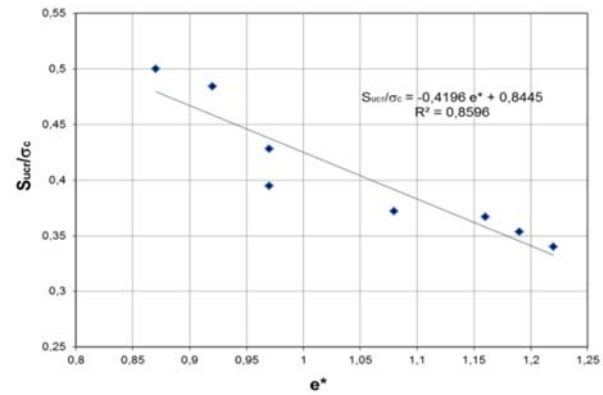

Figure-8. Variation in the undrained critical shear strength versus equivalent void ratio (e^*) (with D_r 15% and 45% at σ_c=100 kPa).

The result of S_{ucr}/σ_c= Fct (e^*) shows the variation of the undrained critical shear strength according to the undrained equivalent ratio for both initial relative densities jointly (Dr =15% and 45%). It is obvious to note that undrained critical shear strength decreases linearly with the increase of equivalent void ratio. Finally, the global void ratio does not give a direct idea about the real behavior of sandy soil with content 0-30% of fine particles; however the equivalent void ratio gives a better characterization of the mechanical condition of silty soil. For the soil under investigation the following relationship is obtained:

S_{ucr} / σ_c = - 0, 42 e^* + 0, 84

4.2 Correlation between critical undrained shear resistance (S_{ucr}) and equivalent relative density (D_r*):

The equivalent intergranular void ratio (e^*) may be represented as an essential representative parameter, consequently the equivalent relative density can be defined as follows (Shenthan T, 2005; Thevanayagam S *and al*, 2003; Thevanayagam S *and al*, 2002):

$$D_r^* = \frac{(e_{\max,cs} - e^*)}{(e_{\max,cs} - e_{\min,cs})} \cdot 100 \tag{8}$$

Where: $e_{max,cs}$ is the maximum void ratio of clean sand and $e_{min,cs}$ is the minimum void ratio of clean sand. This relative index may be defined by equivalent granular state of sandy soils with two extreme density states of clean sand.

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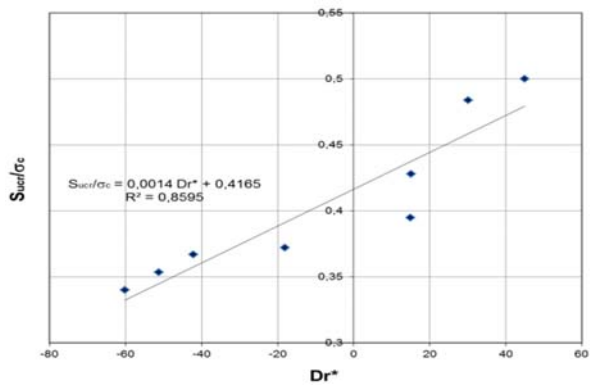

Figure-9. Variation in the undrained critical shear strength versus equivalent relative density (with D_r 15% and 45% at σ_c =100 kPa).

The result of S_{ucr}/σ_c= Fct (D_r*) shows the variation of the undrained critical shear strength versus both equivalent relative densities jointly (with D_r = 15% and 45%). It notes that undrained critical shear strength increases linearly with equivalent relative density.

The following correlation of normalized undrained shear strengths and equivalent relative density is obtained:

$S_{ucr}/\sigma_c = 0,0014\ D_r^* + 0,4165$

4.3 Correlation between peak pore water pressure variation and equivalent void ratio (*e**)

In the same work, the relationship between pore pressure and the equivalent void ratio shows the peak pore water pressure increases directly with the equivalent void ratio (Figure-10) inversely to the variation in the undrained critical shear strength versus equivalent void ratio, similar research on the same subject have been accomplished by Maheshwari B K and Patel A K (2010).

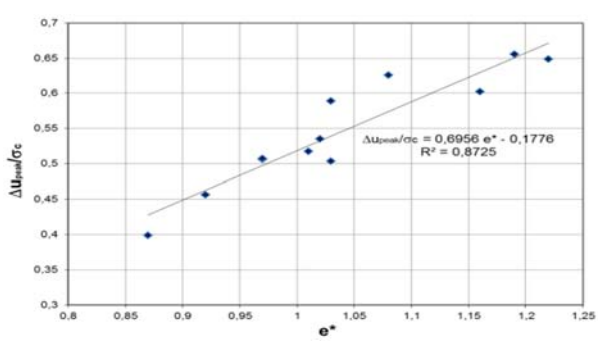

Figure-10. Variation of peak pore water pressure versus equivalent void ratio (with D_r 15% and 45% at σ_c=100kPa).

The following correlation between normalized peak pore water pressure and equivalent void ratio is obtained:

$\Delta u_{peak}/\sigma_c = 0,6956\ e^* - 0,1776$

4.4 Correlation between peak pore water pressure variation and equivalent relative density (D_r*)

The pore water pressure decreases with the increase of equivalent relative density as shown in Figure-11.

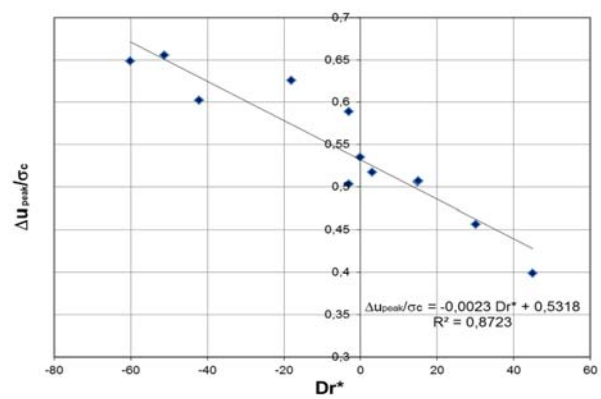

Figure-11. Variation of peak pore water pressure versus equivalent relative density (withD_r 15% and 45% at σ_c=100 kPa).

The following correlation of peak pore water pressure and equivalent relative density is obtained:

$\Delta u_{peak}/\sigma_c = -0,0023\ D_r^* + 0,5318$

5. CONCLUSIONS

A series of monotonic undrained triaxial tests were performed on sand-silt mixtures with two different relative densities loose and medium dense (15% and 45%) at initial effective confining stress of 100 kPa.

From the obtained results, it can be shown that the behavior of samples depends primarily on the fines content (Fines content are varied from 0% to 50%). Lower fine content till around 30% shows a contractive phase behavior followed a dilatancy phase. Beyond 30% of fines content, only a dilative behavior can be observed. The role of fine content may be adequately interpreted by the equivalent void ratio or the equivalent density.

For the soil under investigation, empirical approaches are obtained showing the link between the critical shear strength and the equivalent void ratio. It can be seen that the normalized critical shear strength decreases with the increase of the equivalent void ratio in linear manner. A reverse tendency may be noticed for the variation of the normalized peak pore water pressure against the equivalent void ratio.

Finally, the equivalent void ratio can be used as an essential parameter to low plastic silty sandy soils regarding liquefaction susceptibility.

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