# Liquefaction potential sand-silt mixtures under static loading

## El potencial de licuefacción de las mezclas arena-limo bajo Cargas estáticas


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**Abstract**
As a part of an on-going research program on the mechanical instability of granular soils in our laboratory, defined contents of sand and silt in soil mixtures are studied through the triaxial apparatus, full range of initial relative density states (loose, medium and dense) are experimented and analyzed. In this work, the investigated soil is collected from different depths of Kharouba coastal region in the province of Mostaganem (Lat: 35.96° N; Long: 0.1° E). It consists mainly of sand with a low percentage fraction of non-plastic silt, below 30%. The coastal region of Mostaganem (Kharouba) is very close to the harbor and it experiences a significant seismic activity, then it would be very susceptible to the phenomenon of liquefaction under static or dynamic loads, hence the importance and relevance of this study which introduces the identification parameters of this instability due to liquefaction. Undrained triaxial tests under monotonic loading are carried out in the laboratory on saturated reconstituted samples. These results are presented and analyzed. New parameters for assessing the influence of fines content and density state on the behavior of heterogeneous soils (sand-silt) will be introduced to the liquefaction. New correlations expressing the undrained critical shear with these new parameters will be deducted for full range of initial relative density (loose, medium and dense) for design purpose.



**Key words:** Sand, silt, static liquefaction, undrained, density, monotonic loadings.



**Resumen**
Como una parte de un programa de investigación en curso sobre la inestabilidad mecánica de suelos granulares en nuestro laboratorio, los contenidos definidos de arena y limo en mezclas de suelo se estudian a través del aparato triaxial. Rango completo de estados de densidad relativa inicial (suelto, medio y denso) son experimentados y analizados. En este trabajo, el suelo investigado es coleccionado desde diferentes profundidades de la región costera de Kharouba en la provincia de Mostaganem (Lat: 35.96° N; Long: 0.1° E). Está constituido principalmente por arena con una baja fracción porcentual de limo no plástico, debajo del 30%. La región costera de Mostaganem (Kharouba) está muy próxima del puerto y conoce una actividad sísmica significativa, entonces sería muy susceptible al fenómeno de licuefacción bajo cargas estáticas o dinámicas, de ahí la importancia y la pertinencia de este estudio cual introduce los parámetros de identificación de esta inestabilidad debido a la licuefacción. Las pruebas triaxiales no drenadas bajo las cargas monotónicas se llevan a cabo en el laboratorio sobre muestras saturadas reconstituidas. Estos resultados son presentados y analizados. Nuevos parámetros para evaluar la influencia contenido de los finos y el estado de densidad en el comportamiento de los suelos heterogéneos (arena-limo) serán introducidos a la licuefacción. Nuevas correlaciones que expresan la cizalladura crítica no drenada con estos nuevos parámetros serán deducidas para el rango completo de densidad relativa inicial (suelto, medio y denso) para propósito del diseño.



**Palabras clave:** Arena, limo, estática licuefacción, no drenada, densidad, cargas monotónicas.


## Introduction and literature review

Liquefaction is among the most active domains of geotechnical studies for decades. This phenomenon, known as loss of soil strength, can have disastrous consequences as a result of the spectacular and costly ruptures in terms of human lives and material damage. Initially, researches on liquefaction have been focused on clean uniforms sands, containing no fine (Seed & Idriss, 1971). However, several empirical evidence emerging from the observations as a result of liquefactions which have occurred, their summary results are shown in Table 1. The characteristics of liquefied soil clearly show the influence of fines on the evolution of this phenomenon.

After the Haichang (1975) and Tangshan (1976) earthquakes, (Wang, 1979) established criteria to identify the susceptibility of soils to liquefaction. The criteria proposed by (Wang, 1979) commonly known by Chinese criteria, are essentially based on four conditions which are:

- Fines content in clay particles (≤ 5 microns) ≤ 15%.
- Liquid limit ($W_L$) ≤ 35%.
- Natural water content($W_N$) ≥ 0.9 $W_L$ (saturated condition).
- Liquidity Index($I_L$) ≤ 0.75.

**Table 1.** Observations and results of liquefaction susceptibility of soils. Source: own elaboration.

| Year | Place | Sources | Characteristics of liquefied soils |
|---|---|---|---|
| 1907 | Wachusett (U.S) | (Olson, Stark, Walton, & Castro, 2000) | Approximately 5-10% silt |
| 1964 | Niigata (Japan) | (Kishida, 1969) | 70% fines and 10% of clay fraction |
| 1968 | Tokachi (Japan) | (Tohno & Yasuda, 1981) | 90% fines and 18% of clay fraction |
| 1974 | Alberta (Canada) | (Plewes, O'Neil, McRoberts, & Chan, 1989) | Static Liquefaction (10-15% fines content) |
| 1976 | Tangshan (China) | (W. Wang, 1979) | 20% of clay fraction |
| 1978 | Mochikoshi (Japan) | (Ishihara, Yasuda, & Yoshida, 1990) | Silty sand ($F_C$ ≈ 50% fines) |
| 1983 | Idaho (U. S) | (T L Youd, Harp, Keefer, & Wilson, 1985) | 70% fines and 20% of clay fraction |
| 1991 | Sullivan (Canada) | (Davies, Chin, & Dawson, 1998) | Static liquefaction of a silty sand ($F_C$> 50%) |
| 1993 | Hokkaido (Japan) | (Miura, Yagi, & Kawamura, 1995) | 48% fines and 18% of clay fraction |
| 1994 | Merriespruit (South Africa) | (Fourie, Blight, & Papageorgiou, 2001) | Static liquefaction of a silty sand ($F_C$> 50%) |
| 1999 | Chichi (Taiwan) | (Ku, Lee, & Wu, 2004) | Fines content of 36% to 53% |
| 2009 | Olancha (U. S) | (Holzer et al., 2010) | Fines content of 15% ± 8% |

Many probabilistic approaches exist (Villavicencio, Breul, Bacconnet, Fourie, & Espinace, 2016), but in practice, the implementation of such an approach is difficult. On the other hand, new evidence indicates insufficiency and a certain deviation from these Chinese criteria with respect to the presence of fines. Many questions may arise on these criteria and many researchers have pointed out the need to review them in a broader context (Prakash & Puri, 2010).

The role of the fine relative to the potential of soil liquefaction is a broadly investigated subject, but research results are controversial. According to the previous research works, the presence of fines can increase or decrease the susceptibility to liquefaction as shown in Table 2, which summarizes these conflicting results since the introduction in 1979 of Chinese criteria.

Precisely, the above obtained results show contradictory behaviors, therefore new concept is needed, by introducing some new parameters which identify the real soil behavior to mechanical instability the potential of liquefaction of soils. The site of the sample (Figure 1) of the soil under investigation, is essentially formed by quaternary deposits formed by a silty sand, results of emerged old beaches which covered a marl substratum Pliocene Highlighted by different polls performed in the site.

In this research program work, new geotechnical parameters are introduced in order to reflect the behaviors of sand-silt mixtures to the mechanical instability or liquefaction potential. Initially, a former research study was conducted on loose medium sand-silt mixtures with the following initial density states $D_r$ = 15%, $D_r$ = 50% (Bensoula et al., 2014). Since the state of density in-situ soil under study falls in the full range of density states (loose, medium and dense), it is more interesting to deduce new relationships that can encompass the all range density state. In order, to recheck the soil liquefaction tendency, the same guidelines of the previous undertaken work (Bensoula et al., 2014) were set and new general relationships are obtained to cover the entire initial density states (loose, medium and dense).

Table 2. Summary of findings on the relationships between the fine and susceptibility to liquefaction. Source: own elaboration.

| Relationship (when the fines content increases) | Year | Researchers | Research field / results / comments |
|---|---|---|---|
| More liquefiable | 1997 | (Lade & Yamamuro, 1997) | Presence of fines creates a very compressible structure |
| | 2009 | (Beroya, Aydin, & Katzenbach, 2009) | Clay minerals control the cyclic behavior |
| Less liquefiable | 2001 | (T Leslie Youd & Idriss, 2001) | Empirical graph used globally |
| | 2006 | (Gratchev, Sassa, & Fukuoka, 2006) | Demonstrate the influence of the plasticity |
| | 2010 | (Maheshwari & Patel, 2010) | Effect of silt content depends on the relative density |
| | 2013 | (Chen & Xiao, 2013) | Analysis after the 2008 earthquake |
| Transition more to less liquefiable | 2010 | (Abedi & Yasrobi, 2010) | Content of fines critical of 10% to 15% |
| | 2011 | (Baziar, Jafarian, Shahnazari, Movahed, & Tutunchian, 2011) | Transition fines from 10% to 15% |
| | 2012 | (Lade, 2012) | Initial location of fines is the main factor |
| | 2012 | (Y. L. Wang, Li, Sun, & Yuan, 2012) | Research only on the clay fraction |
| | 2014 | (Bensoula, Missoum, & Bendani, 2014) | Transition from 30% fines content (loose medium sand- silt mixtures) |

## Methodology

Two types of tests were carried out in this study, one in the site and the other one in the laboratory. For the essays made in the site, a set of geotechnical recognitions were made (Figure 1) including the nicked polls, the standard penetration tests, the pressiometric tests and the dynamic penetrometer essays.

Figure 1. Site of sampling studied. Source: Google Maps.


The samples collected in different depths were used in different tests in the laboratory to make a determination of the mechanical and physical features of the soil. The lithological cut of the site resulting from the nicked polls from 25 m of depth gave the composition of the soil which the results are grouped in the Table 3.

Table 3. Results of in-situ polls. Source: own elaboration.

| Soil nature | Sand | Marl (soil) | Marl (Rock) |
|---|---|---|---|
| Depth (m) | 0 - 8 | 8 - 18 | 18 - 25 |

The standard penetration tests (SPT) were made with parallel with the polls to identify the relative density, the harshness, the resistance and the deformation of the soil. The test was realized with an enforcement of the corer by a hammer drop which weighs 63.5 Kg with a height of a drop height of 76 cm. The pressiometric tests (MPT) were made in different depths in a zone to calculate by depths the limited pressure and the deformation modulus.

The dynamic penetration tests (CPT) consist to identify changes of the characteristics of the layers according of the depth and the dynamic stress. The dynamic penetration consists to sink in the soil, by beating with a quasi-continuous manner, a train of rods with an overflowing tip at one end. The number of blows of the hammer correspond to a given enforcement and noted as the measure of the penetration of the tip in the soil. The results of the different tests in-situ of our site are given in the Table 4.

**Table 4.** Results of in-situ tests. Source: own elaboration.

| Soil nature | Depth (m) | Rheological coefficient $\alpha$ | Pressiometric tests | | Dynamic penetration tests | Young modulus $E_{si}$ (MPa) |
|---|---|---|---|---|---|---|
| | | | Deformation modulus $E_m$ (MPa) | Limited pressure $P_l$ (MPa) | Number of blows $N_{SPT}$ | |
| Sand | 8.2 | 0.33 | 6.5 | 0.7 | 6 | 19.7 |
| Marl | 11.8 | 0.67 | 7 | 0.75 | 9 | 10.4 |

The soil elastic modulus (Young modulus) is calculated with correlation of results of the tests in-situ (pressiometric) according to the norm NF P94-262 with $E_{si}= E_m/\alpha$. The results of the mechanic tests made in the laboratory on undisturbed samples with a shear box test are given in the Table 5 providing the two essentially characteristics of the soil should be known, the cohesion $C_u$ and the angle of internal friction $\varphi$.

**Table 5.** The angle of internal friction and the cohesion according to the shear box test. Source: own elaboration.

| Soil nature | Cohesion (kPa) | Angle of internal friction (°) |
|---|---|---|
| Sand | 10 | 25 |
| Marl | 30 | 17 |

The grading analysis allows to give an idea about the distribution of grains depending of the size of the grains who formed the soil and the Atterberg limits allows to give an outline about the consistence of the soil. liquidity limit $W_L$%, plasticity limit $W_P$% and the plasticity index $I_p$%. The results obtained are summarized in the Table 6 below.

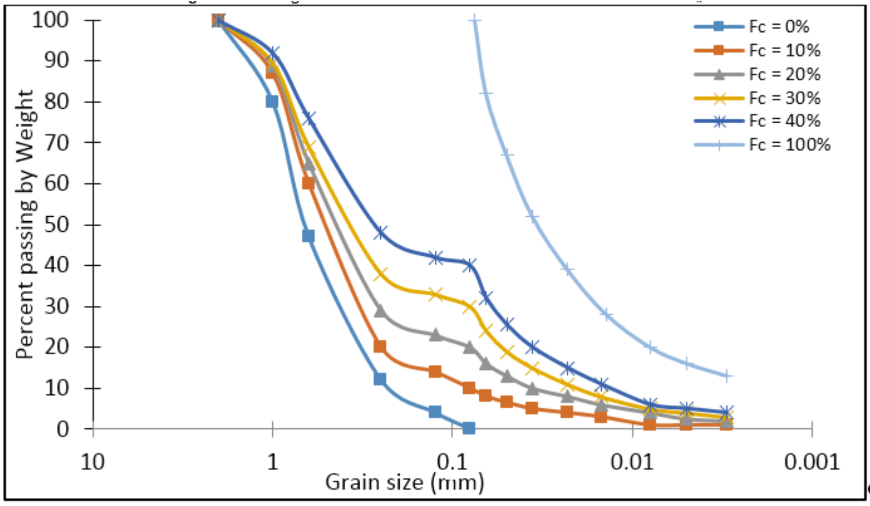
**Table 6.** Atterberg limits and grading of the studied soil. Source: own elaboration.

| Depth (m) | Atterberg limits | | | Grading (%) | |
|---|---|---|---|---|---|
| | $W_L$ (%) | $W_P$ (%) | $I_P$ (%) | 2.00 mm | 0.08 mm |
| 2 - 3 | 24.93 | 20.12 | 4.81 | 100 | 18.39 |
| 7 - 8 | - | - | - | 100 | 29.65 |
| 14 - 15 | - | - | - | 100 | 35.60 |
| 17 - 18 | - | - | - | 100 | 47.08 |

The naturel water content (W%) expresses the percentage of water in the soil studied, the values obtained varies around 7.54 and 13.85% and the degree of saturation ($S_r$%) who characterize the percentage of the gaps that can be occupied by the water runs from 96 to 99%.

In this study, the materials used are removed from different depths runs from 0 to 8 m from the site of samples. These samples showed that the content of fine silt do not exceed 30% (Table 6). Basing on the work of (Thevanayagam & Mohan, 2000) who consider that the matrix of a mixture of sand and fines is a combination of two sub-matrixes which are a major matrix and an another fine matrix, which there are a choice to power a part of the fines in the clean sand to form samples for the tests. The samples taken from the site of the study which are formed by a mixture of silt-sand are separated to form samples used for the three tests of this study. This samples contains fines from 0 to 40% with three different initial relative densities to be known, 15%, 50% and 95%. The density of the clean sand is 2.65 and of the silt is 2.70. For the limit of liquidity and plasticity, the values taken are assimilated to those found in the physical tests made on the samples taken between 2 and 3 m (Table 6), to be known $W_L$ is rounded up to 25% and $W_P$ to 20% so the plasticity index would be equal to $I_P$ = 5%. The Figure 2 below shows the different grading curves of reconstituted samples used in the study.

**Figure 2.** Grading curves of the materials used. Source: own elaboration.




The indications of the extreme gap are conversely proportional to the content of fine (Fc) from 0 to 30% but beyond of this values the trend is reversed. The Table 7 regroups all the geotechnical properties of the reconstituted samples used in this study.

**Table 7.** Geotechnical properties of experienced sand-silt mixtures. Source: own elaboration.

| Properties | Clean Sand ($F_c$ = 0%) | Silt-Sand ($F_c$ = 10%) | Silt-Sand ($F_c$=20%) | Silt-Sand ($F_c$=30%) | Silt-Sand ($F_c$ = 40%) | Silt ($F_c$= 100%) |
|---|---|---|---|---|---|---|
| $F_c$ (%) | 0 | 10 | 20 | 30 | 40 | 100 |
| $G_s$(g/cm$^3$) | 2.650 | 2.663 | 2.671 | 2.683 | 2.696 | 2.700 |
| $e_{min}$ | 0.507 | 0.485 | 0.429 | 0.397 | 0.456 | 0.754 |
| $e_{max}$ | 0.857 | 0.801 | 0.763 | 0.714 | 0.756 | 1.433 |
| $D_{10}$ (mm) | 0.342 | 0.128 | 0.046 | 0.019 | 0.011 | 0.0013 |
| $D_{30}$ (mm) | 0.571 | 0.288 | 0.207 | 0.132 | 0.087 | 0.015 |
| $D_{50}$ (mm) | 0.781 | 0.467 | 0.398 | 0.297 | 0.225 | 0.029 |
| $D_{60}$ (mm) | 0.815 | 0.598 | 0.512 | 0.446 | 0.327 | 0.041 |
| $C_u$ | 2.38 | 4.67 | 11.13 | 23.47 | 29.73 | 31.54 |
| $C_c$ | 1.17 | 1.08 | 1.82 | 2.06 | 2.10 | 4.22 |

The Figure 3 show the triaxial device used for the realization of the tests named "AUTOTRIAX 29-WF4632". It allows the determination of effective stress and stress paths with a pressure cellular runs to 3500 KPa and a counter-pressure runs to 1000 KPa.

The preparation of the sample is a very important step in the study of the susceptibility of the liquefaction of soils because it affects considerably the results found (Ladd, 1974), (Mulilis, Arulanandan, Mitchell, Chan, & Seed, 1977). That's why, the dry discharge method was used in this experimentation i.e. by aerial way by putting down in the soil in a cylindrical mold with a high of 144 mm and a diameter of 70 mm using a funnel. when the molds are prepared, it should be a saturation of samples using the carbon dioxide flow (Lade & Duncan, 1973). The quality of the saturation is measured with the coefficient of Skempton (B) which equal $\Delta u/\Delta\sigma$ knowing that one sample is considered like is completely saturated if this coefficient B is more than 90%.

After consolidating the samples by increasing the pressure in the cell and in inside of the sample at the same time by generators of hydraulic pressure, the micro-bubbles of the interstitial gas which are situated in the grains squeeze by applying a counter pressure which improve the quality of the saturation. Therefore, the applied pressures are maintained in the cell and the sample until the volumes are stabilized. A regular augmentation is given to the generator of hydraulic pressure which is connected to cell with a value of 100 KPa that's allows to measure the change of the interstitial pressure $\Delta u$ in the generator of hydraulic pressure connected to the sample.

**Figure 3.** Used automatic triaxial device 29-WF4632. Source: own elaboration.

## Results and discussion

Throughout the automatic triaxial test, the software provides total control of the triaxial press, the automatic regulation of pressure, the opening and closing of the pressure lines and the continuous measurement of the volume variation to automatically perform the different steps of the test. During the tests the stress paths in the plan (p', q) are registered and represented on the graph.

The Figures from 4 to 6 represent the results of the undrained tests to different values of the content of fines ($F_c$ = 0% to $F_c$ = 40%) and different relative densities ($D_r$ = 15%, $D_r$ = 50% and $D_r$ = 95%) with an initial confining pressure of 100 KPa.

Basing on the developed theory by (Vaid & Chern, 1983) concerning the behavior of saturated sand samples in undrained triaxial tests, we noticed in the Figure 4 to 6 that in the case which $F_c$ = 40%, the mixture have a dilatant behavior and it does not develop with a contraction stage and the critical stress deviator ($q_{cr}$) increase continuously in opposite to other cases which $F_c$ varies from 0 to 30%.

**Figure 5.** Stress path in the plan (p', q) with Dr = 50%. Source: own elaboration.

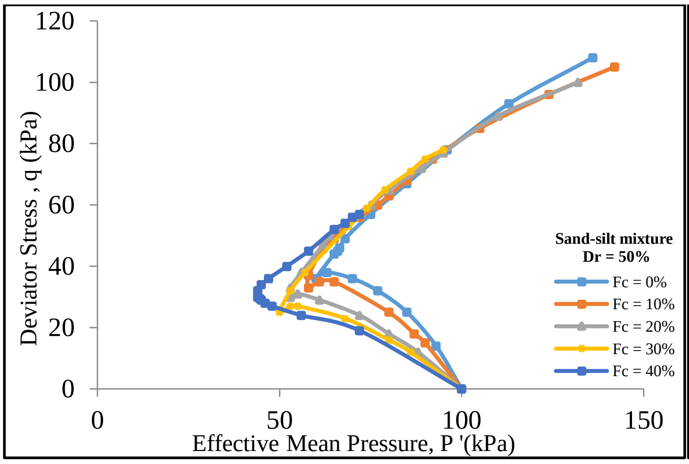
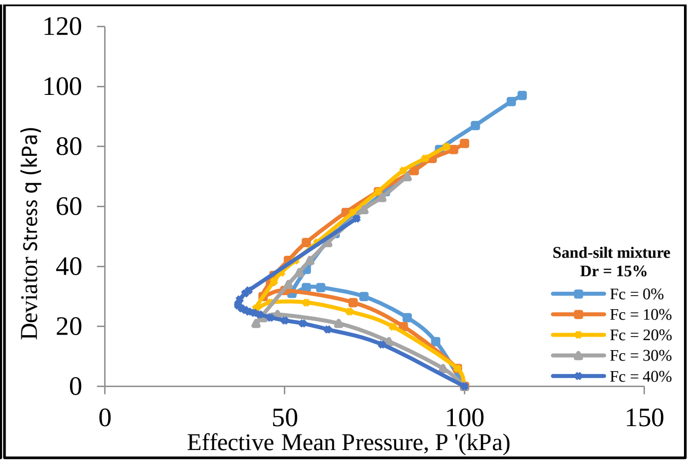



**Figure 4.** Stress path in the plan (p', q) with Dr = 15%. Source: own elaboration.




Another observation to note is that the stage of contraction is almost absent when the initial relative density increase like we clearly see in the Figure 6.

The Table 8 summarize all the results of deviators critical stress ($q_{cr}$), the medium critical stress ($p'_{cr}$),the slop of the line of critical state (M) and the angle of intergranular friction ($\phi_s$).

**Figure 6.** Stress path in the plan (p', q) with Dr = 95%. Source: own elaboration.

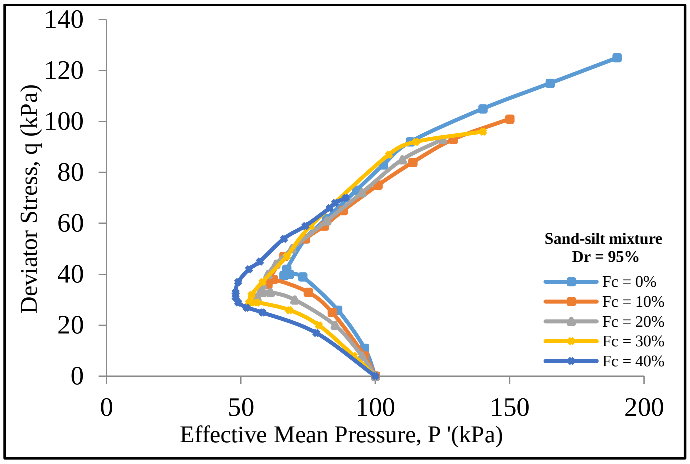



**Table 8.** Test results for different initial relative densities of sand-silt mixtures. Source: own elaboration.

| | $D_r$(%) | $q_{cr}$ (kPa) | $P'_{cr}$(kPa) | $\phi_s$ (°) | M |
|---|---|---|---|---|---|
| Clean Sand | 15 | 34.50 | 57.86 | 21.62 | 0.84 |
| ($F_c$ = 0%) | 50 | 38.69 | 63.11 | | |
| | 95 | 40.88 | 71.38 | | |
| Silt-Sand | 15 | 32.08 | 50.48 | 21.38 | 0.83 |
| ($F_c$ = 10%) | 50 | 35.84 | 63.13 | | |
| | 95 | 38.13 | 62.26 | | |
| Silt-Sand | 15 | 28.54 | 51.67 | 21.14 | 0.82 |
| ($F_c$ = 20%) | 50 | 31.08 | 55.10 | | |
| | 95 | 33.12 | 60.54 | | |
| Silt-Sand | 15 | 24.09 | 48.13 | 20.67 | 0.80 |
| ($F_c$= 30%) | 50 | 27.32 | 54.17 | | |
| | 95 | 29.56 | 55.26 | | |
| Silt-Sand | 15 | 27.52 | 37.13 | 20.19 | 0.78 |
| ($F_c$= 40%) | 50 | 30.85 | 44.22 | | |
| | 95 | 33.36 | 48.09 | | |

The undrained shear strength $S_u$ (the yield strength) reaches a peak in undrained conditions with a constant volume because the effective mean stress decreases at the same time when the pore water pressure develop.

The liquefaction is more pronounced when the applied shear stress of the soil exceeds this peak shear strength like a result to the static or dynamic loading. So from this point we can talk about an unstable regime which is developing in a sample and having a sudden failure with a large deformation until the critical state is attainted on the critical state line (CSL). In undrained situations, the shear strength at steady state or the critical shear strength, the following relationship (1) can be written:

$$q_s = M.p'_s \quad (1)$$

According to (Schofield & Wroth, 1968) and triaxial tests, the relation (2) is written like below:

$$\sin \emptyset_s = (3.M)/(6 + M) \quad (2)$$

From that we can say that the critical shear strength $S_{ucr}$ is defined by the following equation:

$$S_{ucr} = (q_s/2).\cos\emptyset_s \quad (3)$$

The Table 9 summarizes the whole results of the calculation of the undrained critical shear strength ($S_{ucr}$) of different constant in fines and the initial relative density of the studied soil.

Table 9. Undrained monotonic test results for different initial relative densities of sand-silt mixtures Source: own elaboration.

| | $F_c$(%) | e | e* | $e^*_{min}$ | $e^*_{max}$ | $D_r$(%) | $D^*_r$(%) | M | $S_{ucr}/\sigma_c$ |
|---|---|---|---|---|---|---|---|---|---|
| Clean Sand $F_c$ = 0% | 0% | 0.805 | 0.805 | 0.507 | 0.857 | 15 | 15 | 0.84 | 0.1604 |
| | | 0.695 | 0.695 | | | 50 | 50 | | 0.1798 |
| | | 0.567 | 0.567 | | | 95 | 95 | | 0.1900 |
| Silt-Sand $F_c$= 10% | 10% | 0.754 | 0.904 | 0.612 | 0.956 | 15 | -16.26 | 0.83 | 0.1494 |
| | | 0.635 | 0.775 | | | 50 | 27.98 | | 0.1669 |
| | | 0.505 | 0.634 | | | 95 | 72.22 | | 0.1775 |
| Silt-Sand $F_c$= 20% | 20% | 0.713 | 0.977 | 0.649 | 1.035 | 15 | - 41.34 | 0.82 | 0.1331 |
| | | 0.590 | 0.835 | | | 50 | 6.56 | | 0.1449 |
| | | 0.463 | 0.688 | | | 95 | 54.47 | | 0.1545 |
| Silt-Sand $F_c$= 30% | 30% | 0.666 | 1.021 | 0.694 | 1.079 | 15 | - 63.66 | 0.80 | 0.1127 |
| | | 0.565 | 0.898 | | | 50 | -14.62 | | 0.1278 |
| | | 0.443 | 0.750 | | | 95 | 34.41 | | 0.1383 |
| Silt-Sand $F_c$= 40% | 40% | 0.711 | 1.169 | 0.846 | 1.226 | 15 | - 99.38 | 0.78 | 0.1291 |
| | | 0.605 | 1.035 | | | 50 | - 54.37 | | 0.1448 |
| | | 0.503 | 0.906 | | | 95 | -9.37 | | 0.1566 |

## Results and discussion

Clearly the behavior of clean sand is different from sand-silt mixtures. As research findings are controversial on the influence of fines on susceptibility to liquefaction, as there are some who have concluded that the introduction of fine sand in the matrix reduces the undrained shear (Chang, Yeh, & Kaufman, 1982), while others said the opposite (Troncoso & Verdugo, 1985; Vaid, 1994).

Therefore, on the basis of these divergent results presented in the literature, the fines content cannot alone provide a unified trend of soil behavior. Structural stability based on a soil susceptible to liquefaction depends on the shear resistance of the soil in post-liquefaction. Soil strength during the transition phase has a major influence in the design of engineering structures (Ishihara, 1993).

Therefore, it is important to clarify the main parameters that significantly influence the shear strength and derive some principles in the process of design.

The notion the ratio intergranular voids ($e_s$) is introduced by (Lade & Yamamuro, 1997) who show that this index controls the undrained shear sand-silt mixtures.

Thevanayagam & Mohan (2000) gave the ration intergranular voids formula which is written as follows:

$$e_s = (e + F_c)/(1 - F_c) \quad (4)$$

The Figure 7 shows the influence of the critical shear resistance according to the ratio of intergranular voids ($e_s$) while the Figure 8 shows the evolution of the ratio of intergranular voids ($e_s$) a function of the content of the fine ($F_c$).

The concept of the index of intergranular voids suggests that the fine fill in the voids formed between the grains of sand and silt-sand behavior mixture should be governed by the index of intergranular voids instead of empty global index. However according to Thevanayagam & Mohan (2000), when the empty intergranular maximal index exceeds the one of the clean sand, the fines constitute a dominant structure and prevent the contact between the grains of sand and consequently it controls the shear resistance while the grains of sand become secondary.

Zlatović & Ishihara (1995) and Pitman, Robertson, & Sego (1994) found that the thin particles have a contact with the sand grains to give a constant which runs from 5% to 25%, from this Thevanayagam, Shenthan, Mohan, & Liang (2002) introduced a new parameter called the equivalent intergranular voids noted (e*) which allows a fraction of fines to contribute to the resistance of the scrawny chain of the soil beginning from the limit level of the value of ($F_c$). The value of (e*) is defined by the following equation:

$$e^* = (e + \alpha.F_c)/(1 - \alpha.F_c) \quad (5)$$

where $\alpha$ is the parameter which determines the fraction of fines which participates in resistance in the soil skeleton.

Generally, (e*) is obtained from the correlation between the ground assessment properties and their values after analysis (Ni, Tan, Dasari, & Hight, 2004; Yang, Lacasse, & Sandven, 2005).

Rahman, Lo, & Gnanendran (2008) concluded that the index of intergranular voids depends on the ratio (r) grain size (sand and silt) and fines content ($F_c$).

**Figure 8.** Evolution of the intergranular voids ratio (es) as a function of the content of the fine (Fc). Source: own elaboration.

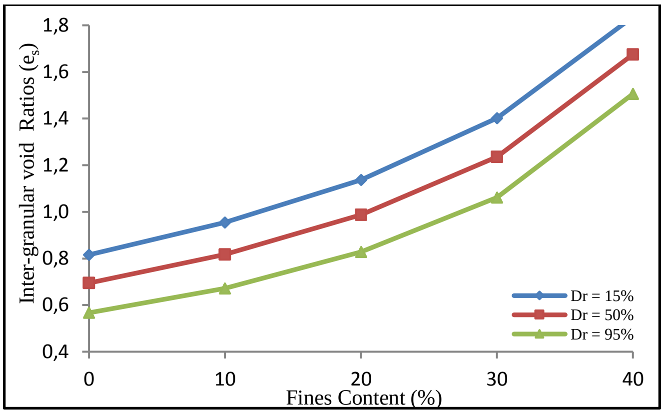
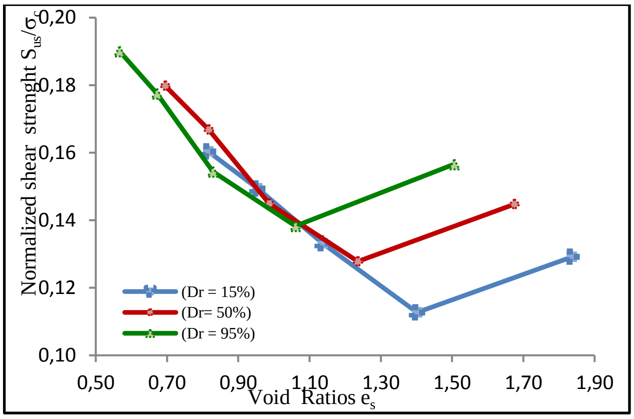



**Figure 7.** Influence of the critical shear resistance according to the ratio of intergranular voids (es). Source: own elaboration.




The ratio (r) of grain size is defined by the following equation (6):

$$r = D_{50(fine)} / D_{10(sand)} \quad (6)$$

The correlation is given by the following equation:

$$\alpha = 1 - [\{1 - \exp([-0{,}3(F_c/F_{thre})/k)]\}](rF_{thre}/F_c)^r \quad (7)$$

where $k = 1 - r^{0,25}$ and $F_{thre}$ is the threshold of fines in the mixture or the transient content of fines which characterizes the behavior of the predominance of fines.

Thus, the value of $F_{thre}$ is defined by the point where the tendency of behavior is reversed with increased fines content. In this study and according to the results of the tests, this transition point is located when the fine content is 30% in each of three applied initial relative densities, when r = 0.100 and k = 0.438.

The variation of the equivalent voids content (e*) according to the fines content ($F_c$) is illustrate on the Figure 9, while the Figure 10 shows the variation of undrained critical shear resistance with respect to the equivalent voids ratios.

**Figure 10.** Variation of undrained critical shear strength with respect to the equivalent voids ratios Source: own elaboration. Source: own elaboration.

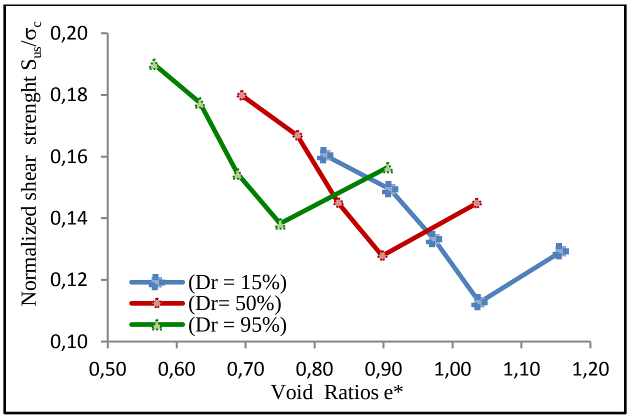
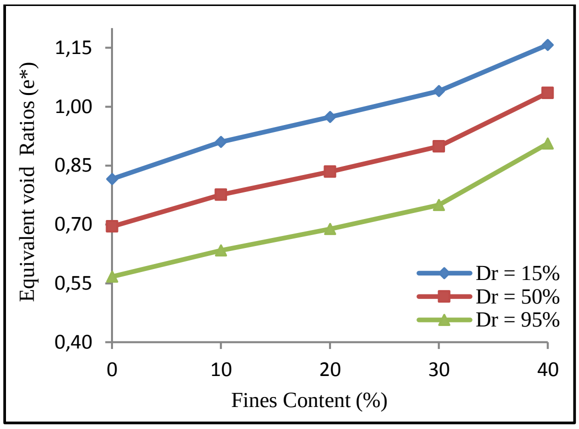



**Figure 9.** Variation of equivalent voids ratios (e*) function of the fines content(Fc). Source: own elaboration.



This means that by reducing the overall void index and increasing fines content, the critical shear strength also decreases. So it is obvious to conclude that the overall empty index does not represent the true behavior of sandy loam soils with fines content varies from 0 - 30%.

A linear correlation is illustrated in the Figure 11 to provide an equation which link the undrained critical shear strength and the index equivalent intergranular voids this equation is written which is written as follows:

$$\frac{S_{ucr}}{\sigma_c} = -0.143e^* + 0.267 \tag{8}$$

The index equivalent intergranular voids is an important parameter to describe the behavior of the sandy loam soils but it is possible to use other parameters like the equivalent relative density ($D_r^*$) which is a consequence obtained from the introduction of the equivalent voids ratios (e*) as the initial relative densities must decrease. The equivalent relative density is defined as follows (Thevanayagam et al., 2002; Thevanayagam, Shenthan, & Kanagalingam, 2003) and Shenthan (2005):

$$D_r^* = \left[\left(e_{max,cs} - e^*\right)/\left(e_{max,cs} - e_{min,cs}\right)\right]x100 \tag{9}$$

**Figure 11.** Correlation of undrained critical shear strength a function of an equivalent voids ratios (e*). Source: own elaboration.

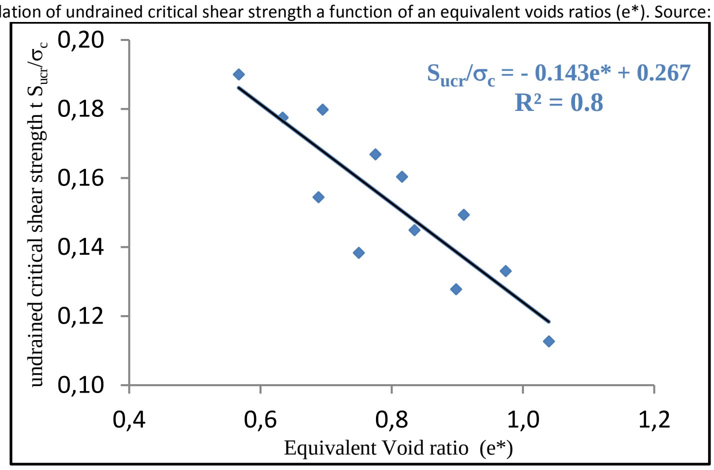



Figure 12 illustrates the results of mixtures used in different initial relative densities using the formula (9). A correlation is deducted to link the equivalent intergranular voids and the equivalent relative density when the expression is defined by the equation (10) below:

$$D_r^* = -313.4\ e^* + 267.7 \tag{10}$$

**Figure 13.** Correlation of undrained critical shear strength with respect to equivalent densities relative ($D_r^*$). Source: own elaboration.

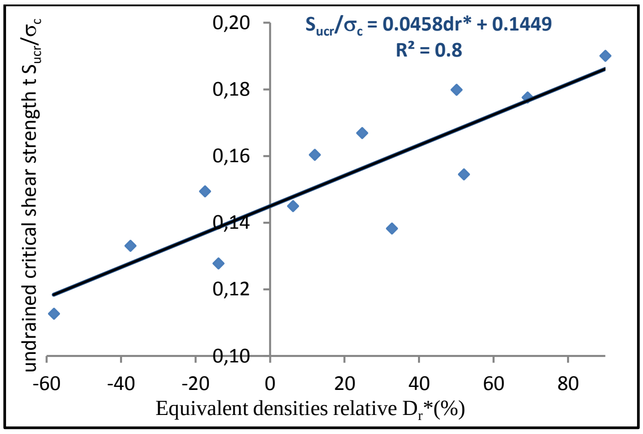
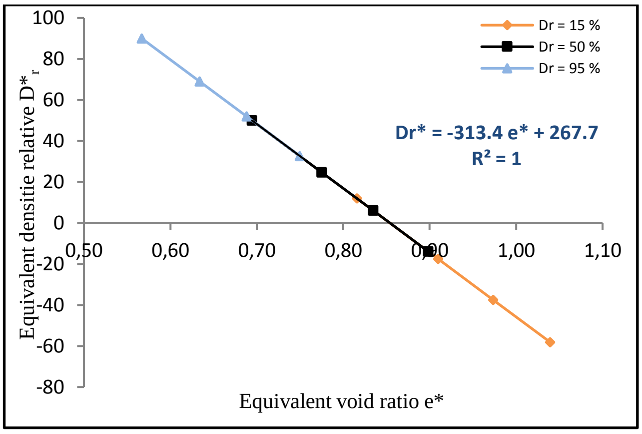



**Figure 12.** Variation of equivalent relative densities ($D_r^*$) with respect to equivalent void ratio (e*). Source: own elaboration.




A decrease in the equivalent relative density is noticed when the fines content increases and negative valuesfor loose soil as the equivalent void ratio is greater than the maximum void ratio of clean sand.

The variation of the critical shear resistance undrained in relation to the equivalent relative density to the different initial relative densities ($D_r$ = 15%, 50% and 95%) is illustrated in the figure 13. The undrained critical shear resistance thinks that when the equivalent relative density increase when the contents in fine are ≤ 30%.

In the interval 0% - 30% fine content, equivalent intergranular voids ratio and the equivalent relative density become the main parameters for characterizing the susceptibility to liquefaction of these soils.

As shown in Figure 13, a linear correlation that links the undrained critical shear strength with the equivalent relative density is expressed in eq. (11) below:

$$\mathrm{S_{ucr}}/\sigma_c = 0.0458\mathrm{d_r^*} + 0.1449 \qquad (101)$$

Where $\mathrm{d_r^*}$ is the equivalent relative density expressed in decimal ($\mathrm{d_r^*} = \mathrm{D_r^*}/100$ ).

Like we just said before the behavior of the loam-sandy soil developing liquefaction phenomenon or mechanical instability when the content in fines varies from 0% to 30% can be defined by the equivalent relative density and by the equivalent intergranular voids.

These obtained results justify the earlier results (Bensoula et al., 2014) for both loose and medium density states and the same tendency is clearly observed for dense soils ($D_r$ = 95%).

## Conclusions

The results show that the undrained critical shear strength is greatly affected by the content of fines contained in the sand matrix at also by the initial relative densities. Participation of fine silt in soil behavior depends on the fine fraction present in this soil as well as its density state.

The obtained results in this work, justify the earlier results for both loose and medium density states and the same tendency is clearly observed for dense soils ($D_r$ = 95%).

When the initial relative density is maintained constant the liquefaction resistance decreases as the fines content increases to a threshold which is in this study of 30% independently of the initial density state. The undrained critical shear strength decreases linearly with increasing equivalent intergranular voids ratios, whereas it increases with the increase of the equivalent relative density, this behavior is valid only for an $F_c$ fines content less than or equal to 30%. In the range of 0-30% by fines fraction and through the two correlations, the equivalent intergranular voids ratios and the equivalent relative density may be key parameters for characterizing the susceptibility of soil instability.

The soil under study is vulnerable to the phenomenon of liquefaction when its content of fines varies from 0% to 30%, this may be defined by an association of one of the two parameters, the equivalent relative density or the equivalent intergranular void. A specific correlation between critical shear strength and these two latter parameters are obtained. When tested, the soil of study has exceeded the threshold by the Chinese criteria and test results clearly show that this soil is likely to liquefy a fine content up to 30%. This coincides with the recent observations made under real case of static liquefaction reported by many researchers, allowing to conclude that the phenomenon of liquefaction appears in sandy soils with fines content less than or equal to 30%.

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