Liquefaction potential of clean and silty sand

M. Bayat¹, E. Bayat²
¹- PhD Student, Department of Civil Engineering, Tehran University, Tehran, Iran
²- BSc Student, Department of Civil Engineering, Tafresh University, Tafresh, Iran

bayat.m@ut.ac.ir

Abstract
Soil liquefaction is one of the most interesting and complex phenomena studied in geotechnical earthquake engineering. The liquefaction resistance of a saturated fine to medium sand mixed with varying amounts of non-plastic fines was evaluated by laboratory cyclic triaxial tests at same relative densities and a constant confining pressure. The test results were used to conclude on the effect of low non-plastic contents (0 to 20%) and grading characteristics on the liquefaction resistance of the sand. The test results indicate that the undrained residual strength reduced with the increase of non-plastic fine content. Also, shear strength of gap graded sand mixed with low non-plastic fine content increases with decrease in effective size (D₅₀). In other words, in this state, we can use the D₅₀ as a parameter to control of silty sand’s undrained resistance. Besides, the undrained residual strength of pour sand specimens with same effective size increases due to increase of coefficient of uniformity (Cu).

Keywords: Liquefaction, Silty sand, Shear resistance, Grading characteristics, effective size

1. INTRODUCTION

Liquefaction of saturated granular soils during earthquakes has been one of the most important and challenging problems in the field of geotechnical earthquake engineering. During earthquakes, the ground shakes, causing cohesionless soils to lose their strength and behave like a liquid. This phenomenon is called soil liquefaction and occurs due to an increase in the excess pore water pressure and a corresponding decrease in the effective overburden stress in a soil deposit and will cause the settlement of buildings, landslides, the failures of earth dams, or other hazards.

Although more than 60 years that the focus of the researches has been on the phenomenon of liquefaction, but large part of these researches has been done on the clean sands, assuming that the behavior of clean sand can be generalized to the natural sands such as silt sand. In fact, most of these researchers believed that firstly plastic fines in sand lead to the increase of the undrained shear resistance (Ishihara and Koseki, 1989; Yasuda et al., 1994) and secondly existence of the silt in sand does not affect the sand residual resistance, because silt fines are similar to sand grains and do not have magnetic forces on their surfaces (Ishihara, 1993). Various tests on different sands have confirmed the first part of idea mentioned above which is based on shear resistance of sand improved by increased amount of plastic fines in sand. The reason behind this issue is due to the fact that liquefaction is so state that increase of pore water pressure causes grains to separate and also sand grains are suspended, but the existence of plastic fines in the soil with magnet forces in their surface causes to situation of almost constancy of grains. Hence, more amount of plastic fines in sand causes the improvement of the undrained shear resistance. However, researches have not confirmed the second part of idea mentioned above; these researches showed that the behavior of clean sand in comparison with sand–silt mixtures is completely different. Yet, there is a disagreement over this difference so that some believed silt in the sand reduces undrained resistance of sandy-soil mixtures (Chang et al., 1982), while others had dissenting opinion (Vaid, 1994; Tronsco and Verdugo, 1985), but Lade and Yamamuro (1997) obviously showed, having performing the tests, that the increase of silt in sand remarkably leads to the decrease in undrained resistance of sand–silt mixtures at constant total void ratio. They justified their reasoning by saying that when an amount of silt is added to sand, a part of silt will be placed in void within the grains, so this amount of silt does not have considerable effect on soil behavior. On the other hand, a part of silt that is placed in contact surfaces of sand grains leads to separation and sliding the grains during loading. This leads to the increase of soil compressibility and the decrease of soil undrained resistance. Polito
(1999) modified viewpoint of Lade and Yamamuro (1997) showing that the increase of silt in sand to the threshold value \( FC_{th} \approx 35\% \) reduces undrained resistance of silt sand, but after this the increase of silt improves undrained resistance at constant void ratio (Fig. 1). Thevanayagam et al. (2002) stated that silt fines have the main role in determining the behavior of soil at the state of FC>FC_{th}. In this case, silt fines are close to each other and sand grains break away. This means that silt fines play the main role in soil bearing skeleton. In this field, Thevayanagam and Martin (2002) offered a soil classification system based on contact density and the equivalent void ratios for sand mixed with fine content (FC) less than and more than a threshold value FC_{th}.

\[ \text{Figure 1. Variation in cyclic resistance with silt content for Monterrey sand at constant void ratio of 0.68} \quad (\text{Polito, 1999}) \]

On the other hand, the physical parameters 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), Della et al (2011), Djafar Henni et al (2011) and Missoum et al (2011). 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, grading characteristics, size and shape of the grains are incomplete and requires further investigation.

A summary of research results are presented; Lee and Fitton (1968) expressed that particle size has much more effect on cyclic resistance with respect to particle shape and particle size distribution curve. Seed and Peacock (1971) believed that for the same number of cycle (10 cycles), as effective size reduces from 1.0 mm to 0.1 mm, magnitude of stress ratio to cause liquefaction decreases. Also, in this field, some of researchers (Finn et al. (1970), Ishihara et al. (1975), Miura et al. (1994)) showed that as particle size increases, the cyclic resistance increases. Seed and Idriss (1971) reported that fine sand with D_{50} value around 0.08 mm is more susceptible to liquefaction. In this field, Castro and Poulos (1977) expressed that Liquefaction is very likely for uniform clean and loose sand. Also 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, C_{u}, provided that D_{50} < 0.23 mm. However, the individual influences of D_{50} and C_{u} were not isolated. Vaid et al. (1991) examined the influence of C_{u} by testing three clean sands with the same mineralogy D_{50} and they concluded that the cyclic liquefaction resistance of clean sand increases with C_{u} at low relative density and the tendency was reversed at high relative density. Yilmaz et al. (2008) expressed that a relationship between cyclic resistance and any of the size (i.e. D_{10}, D_{30} or D_{60}) would be more realistic than to build a relation between coefficient of uniformity or coefficient of curvature and the cyclic resistance.

Some of previous researches concerning the behavior of sand mixed with non-plastic fine show that the void ratio related to sand grains plays a more important role in comparison with the total void ratio, where soil underained resistance will be improved due to increase in FC at the constant the void ratio related to sand. In spite of this fact, the recent works indicate that the void ratio related to sand is unable to show perfectly the role of the non-plastic fines that are in voids between sand grains. For this reason, an equivalent void ratio (has been defined that takes into account the non-plastic fine participation ratio in the soil bearing skeleton. Missoum et al (2001) indicated that there are linear correlations relating the undrained residual shear strength of loose, medium dense, and dense (D_{r} = 12\%, 50\%, and 90\%) sand–silt mixtures to the equivalent void ratio.
The current study includes the results of a set of laboratory tests carried out on sand samples with the same relative densities and variation in the non-plastic fines contents ranging from 0 to 20% and consolidated at mean confining pressure of 300 kPa using static triaxial test apparatus, in order to study the influence of non-plastic fines and grading characteristics and any of the size (i.e. \(D_{10}\), \(D_{30}\) or \(D_{60}\)) on the undrained residual shear strength and liquefaction potential of sand specimens. Also Belkhatir et al. (2011) indicated that the undrained shear strength at the peak and the undrained residual strength can be correlated to \(C_u\) and \(D_{50}\). In other words, they decrease linearly with the increase of the uniformity coefficient and decrease of the average diameter and also they indicated that a relationship between the liquefaction resistance and any of the diameters (i.e. \(D_{10}\) or \(D_{50}\)) and \(C_u\) would be more realistic than to build a relation between \(C_c\) and the liquefaction resistance.

2. Materials tested

Mixtures of sand and silt or clean sand were used in this study, that grading curve of used silt is shown in Fig. 2. The specific gravity of used sand was 2.67, according to ASTM D 854-02 (2002). The sand was divided into thirteen different subgroups. Each subgroup is obtained using two sets of sieves (e.g. upper one and lower one). Sand grains finer than the upper sieve are also filtered by the appropriate lower sieve. Finally, the amount of sand grains retained on the lower sieve is collected and named. The description of subgroups is presented in Table 1.

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<th>Lower sieve (ASTM)</th>
<th>Subgroup Name</th>
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<tr>
<td>No.200</td>
<td>No.400</td>
<td>Silt</td>
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Figure 2. Grain size distribution curves of used silt

3. EXPERIMENTS PROCEDURES

3.1. SAMPLE PREPARATION TECHNIQUE

In this work, wet compaction technique has been used for preparation of specimens. All the specimens have been constructed with the diameter of 70 mm and the height of 140 mm, using seven layers with a constant thickness of 20 mm for each layer, so height to diameter ratio of 2 is kept constant. This length to diameter ratio of 2 selected in order to minimize the effects due to end platens of the apparatus and to reduce the likelihood of buckling during testing.

3.2. INITIAL DEGREE OF SATURATION
For sample preparation, dry sand has been mixed with respect to the considered different weight ratios. To prevent the grains separation, while soil is being poured in the mould and to achieve the desired density easily, for all other samples, the initial moisture percentage is kept constant equal to 8%. The required amount of mixture of dry soil and water for each layer of sample has been determined exactly.

3.3 Final degree of saturation

The saturation is an important stage in the experimental procedure because the response of the sample under undrained loading depends on its quality. To get a good degree of saturation, the technique of carbon dioxide elaborated by Lade and Duncan (1973) was used. After doing necessary measurements, the specimens have been first subjected by CO₂ at least for 30 min and then saturated them by de-aired water. The control of saturation degree is done by means of Skempton’s pore pressure parameter B. According to JGS 0523 (2000) and ASTM D 4767-92 (2002), specimens have been considered to be fully saturated if B is at least equal to or greater than 0.95. In this study, backpressure of 300 kPa has been applied during the tests to achieve the saturation state.

3.4. Consolidation and loading

After the saturation process, the specimens were subjected to the confining stress for consolidation. During consolidation, the difference between confining pressure and backpressure has been arranged such that for each specimen the effective consolidation pressure was fixed equal to 300 kPa. There are the numerous factors affecting the undrained behavior of silty sand, such as sand grading characteristics, fine content, initial density and confining stress. In present work the effects of different factors were studied for a given initial density. The choice of 300 kPa for confining stress as a mean value in geotechnical practice purposes was based on various research works existing in literature such as Ishihara (1993). After consolidation step and calculation of e₀ (e.g. void ratio after consolidation), axial load has been applied on specimen in constant strain rate manner.

4. Results of tests

4.1. First series of tests

For the first series of tests, No.10-No.200 subgroup whose grain size distribution curve has been presented in Fig. 3 was selected. Then, this soil was mixed with different percentages of silt (0, 5, 10, 15 and 20%). In this stage, the purpose is to verify different ideas mentioned in introduction section concerning the role of silt content (FC) in undrained behavior of silty sands. To achieve this purpose, samples have been prepared at the state of constant e. In Table 2, total void ratio and the grading characteristics of samples have been presented. Deviator stress (σ₁-σ₃) versus axial strain curves for different percentages of silt have been shown in Fig. 4. As shown in this figure, with constant e in all samples, the undrained residual resistance has been significantly improved due to increase of silt percentage.
4.2. SECOND SERIES OF TESTS

Three types of subgroups of sand (No.40–No.60, No.20–No.80 and No.10–No.200) with the same D₅₀ have been used in the second series of tests. Their grading curves have been shown in Fig. 5. Stress-strain curves for these samples are shown in Fig. 6 and also samples characteristics are shown in Table 2. As shown in results, residual stress increases due to increase of Cu for the pore sand samples with the same D₅₀. Also, added silt fine (equal to 15%) to the subgroup of No.10–No.200 leads to decrease of residual stress. Note that increase of value of Cu increased residual stress only for pore sand samples in this series of tests. In other words, Cu can be control of the shear strength of pore sand with various grading curves.

![Figure 5. Grain size distribution curves of used subgroups of pore sand in second series of tests](image)

![Figure 6. Stress-strain relationship of used samples in second series of tests](image)

4.3. THIRD SERIES OF TESTS

In the third series of tests, four types of subgroups (No.40–No.200, No.60–No.200, No.80–No.200 and No.100–No.200) have been used whose grading curves are shown in Fig. 7. Samples were prepared with the same silt content equal to 15 percent and at the constant total void ratio. Also, samples characteristics have been presented in Table 2. Difference among samples is in their grading of pore sand at this stage, on the other hand, difference among samples is greatest grain size is various in the samples. Results of the third series of tests have been shown in Fig. 8. No.40–No.200 subgroup sample with a non-uniform grains in comparison with the three other samples has the lowest liquefaction resistance and No.100–No.200 subgroup sample with finer and uniform range of grains size has the highest liquefaction resistance. The reason behind this is that No.100–No.200 subgroup sample has more void within sand grains than No.40–No.200 subgroup; therefore, for a given silt fine filling, the grains contact surfaces will be greater that leads to the increasing of undrained resistance. In fact, for sand with non-uniform grains size, least values of silt content lead to sand grains separation, but for fine sand with uniform grading, it requires more silt for filling void within the grains. So, the sand with non-uniform grains to yield grains separation requires more silt in comparison with sand with uniform grading.

![Figure 7. Grain size distribution curves of used subgroups of pore sand in thirds series of tests](image)

![Figure 8. Stress-strain relationship of used samples in thirds series of tests](image)
4.4. FOURTH SERIES OF TESTS

In this stage, three types of subgroups of Shooshab sand (No.10-No.20, No.10-No.40, No.10-No.60 and No.10-No.80) with the grading curves presented in Fig. 9 have been used. The samples were prepared with the same percent of silt equal to 15 and the constant total void ratio state. The samples characteristics have been presented in Table 2. The basic difference between the subgroups prepared in this stage is in their fines grain size. Results of fourth series of tests are shown in Fig. 10. As it is shown in this figure, No.10–No.80 subgroup sample with smaller and non-uniform sand grains in comparison with other samples has the highest residual resistance and No.10–No.20 subgroup sample with greater and uniform sand has the lowest residual resistance.

![Figure 9. Grain size distribution curves of used subgroups of pour sand in fourth series of tests](image)

![Figure 10. Stress-strain relationship of used samples in fourth series of tests](image)

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<th>D50</th>
<th>D60</th>
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<th>Cu</th>
<th>e</th>
<th>Silt Content (%)</th>
<th>Soil Type (unified system)</th>
<th>Residual Stress (kPa)</th>
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5. CONCLUSIONS

A series of consolidated undrained static triaxial tests were carried out on sand with low non-plastic fine contents at mean confining pressures of 300 kPa. The effect of variation in the non-plastic fine content and
grading characteristics on shear resistance of sand were studied. In the light of the experimental evidence, the following conclusions can be drawn:

1. Increase of silt content in sand to a threshold value (according to previous studies nearly 35%) decrease of shear resistance. On the other hand, a part of silt that is placed in contact surfaces of sand grains leads to separation and sliding the grains during loading. This leads to the increase of soil compressibility and the decrease of soil undrained resistance. For in this state, FC (percent of silt content) is an effective parameter to control of shear resistance of sand. Its means that increase of FC causes a decrease in residual stress.

2. In the sand samples with the same D50, Cu is an effective parameter to control shear resistance of pour sand. In other words, increase of Cu increases shear resistance of pour sand samples.

3. Decrease of D50 of gap graded sand mixed with low silt content increases shear resistance. In other words, in this state, the approach of sand grain size to non-plastic fine causes an increase of shear resistance of graded sand mixed with low silt content.

6. REFERENCES


