Six[th] International Conference of
Seismology and Earthquake Engineering
16-18 May 2011 Tehran, Iran

AN EXPERIMENTAL STUDY CONCERNING THE EFFECTS OF NON PLASTIC FINE AND PHYSICAL PARAMETERS ON THE LIQUEFACTION RESISTANCE

M. Maleki[1], A. Ezzatkhah[2], M. Bayat[3], M. Mousivand[4]

[1] *Associate Professor, Department of Civil Engineering, Bu-Ali Sina University, Hamedan, Iran*
Maleki@basu.ac.ir
[2] *M.SC. Department of Civil Engineering, Bu-Ali Sina University, Hamedan, Iran*
Amin_isi@yahoo.com
[3] *M.SC. Student, Department of Civil Engineering, Bu-Ali Sina University, Hamedan, Iran*
M.Bayat@basu.ac.ir
[4] *M.SC. Student, Department of Civil Engineering, Bu-Ali Sina University, Hamedan, Iran*
Mohsen63m@yahoo.com

ABSTRACT

For sand mixed with non-plastic fine, the void ratio related to sand grains (e_c) plays a more important role in comparison with the total void ratio, where soil liquefaction resistance will be improved due to increase in FC at the constant e_c. In spite of this fact, the recent works indicate that e_c 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 $(e_c)_{eq}$ has been defined that takes into account the non-plastic fine participation ratio in the soil bearing skeleton. In the present work, the generality of the expression of $(e_c)_{eq}$ is verified. For this, a set of static undrained triaxial tests were performed. The results of tests indicate that, the undrained behavior of a given sand mixed with different percentages of non-plastic fine can be described by $(e_c)_{eq}$. But if the grading curves of sand change, we can not find a logic retention between $(e_c)_{eq}$ and undrained resistance of soil, unless the physical and mechanical characteristics of soil are well introduced in expression of $(e_c)_{eq}$.

Keywords: Liquefaction resistance, Silty sand, Equivalent void ratio, Static triaxial test.

1. INTRODUCTION

According to previous researches, the large part of researches concerning the liquefaction phenomena have been done on the clean sands, with assumption that the behavior of clean sand can be generalized to the natural sands such as silty sand. In fact, most of these researchers believed firstly that, plastic fines in sand lead to the increase of the undrained shear resistance [1-2] and secondly that existence of the silt in sand does not affect the sand residual resistance, because silt fines are similar to sand grains and don't have magnetic forces on their surfaces [3]. 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 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 [4], while others had dissenting opinion [5-6], but Yamamuro and Lade (1997) obviously showed, having performing the tests, that increase of silt in sand remarkably leads to the decreasing of undrained resistance of sand-silt mixtures at constant total void ratio [7]. 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. But 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 liquefaction resistance.

They suggested, therefore, that sand skeleton void ratio (e_c) probably can control undrained resistance of silt-sand mixtures. In fact, e_c represents the space within sand grains in the sand-silt mixture. This parameter is calculated by the following expression:

$$e_c = (e + fc)/(1 - fc) \tag{1}$$

Where e is the total void ratio and fc is the ratio of silt weight to total sample weight. According to this relationship, for a given total void ratio, the e_c increase due to the increase in fc that creates the more distances between sand grains.

Polito (1999) modified viewpoint of Yamamure and Lade (1997) showing that the increase of silt in sand to the threshold value ($FC_{th} \approx 35\%$) reduces undrained resistance of silt sand [8], but after this the increase of silt improves undrained resistance at constant void ratio (Fig. 1).

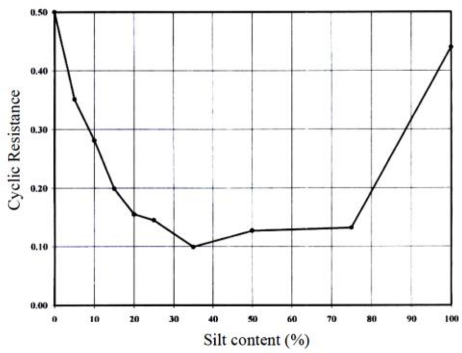

Figure 1. *Variation in cyclic resistance with silt content for Monterery sand at constant void ratio of 0.68 (after Polito, 1999).*

Thevanayagam et al. (2002) stated that silt fines have the main role in determining the behavior of soil at the state of $FC > FC_{th}$ [9]. In this case, silt fines are close to each other and sand grains are break away. This means that silt fines play the main role in soil bearing skeleton. They suggested that undrained resistance of silt-sand mixtures can be described by parameter e_f that is the ratio of total void ratio to the soil content or:

$$e_f = e / fc \tag{2}$$

Thevanayagam et al. expressed that, at the state of $FC < FC_{th}$, although the e_c can show grain separation with the increase of silt accurately, e_c is unable to express the role of silt fines that are in the void within sand grains, as they do not facilitate grain separation and sliding [9]. Therefore, Thevanayagam et al. (2000) modified e_c and presented $(e_c)_{eq}$ with the following expression [10]:

$$\left(e_c\right)_{eq} = \left(e + (1-b)\times fc\right) / \left(1-(1-b)\times fc\right) \quad (3)$$

Where, *b* is portion of the silt fines that contribute to the active inter-grain contacts. According to Thevanayagam et al. (2002); *b*=0 would mean that none of the fine grains actively participates in supporting the coarse-grain skeleton; *b*=1 would mean that all of the fine grains actively participate in supporting the coarse grain skeleton.

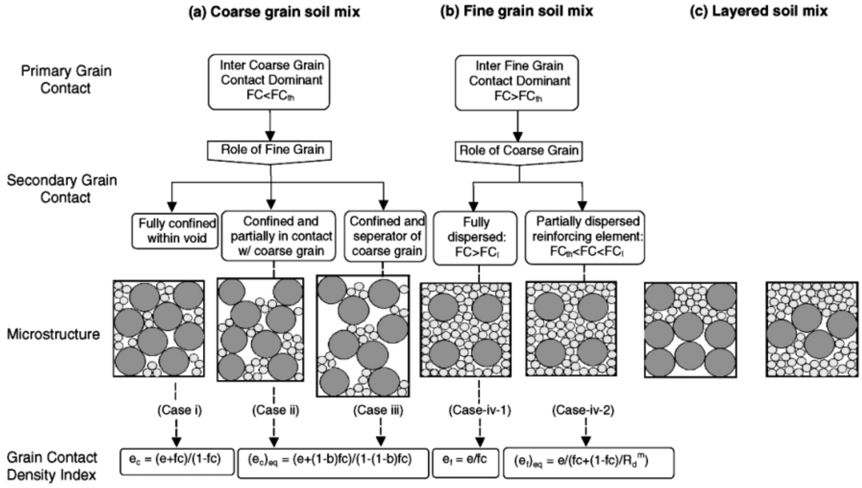

Figure 2. *Granular mix classification and contact density indices (after Thevanayagam 2002)*

b value is dependent on the grain characteristics and grain size disparity ratio $R_d=D/d$, where *D* is sand grains average diameter (D_{50}) and *d* is silt fine average diameter (d_{50}). They also modified e_f, and presented $(e_f)_{eq}$ for controlling the behavior of silt-sand mixtures at the states of $FC>FC_{th}$. Yang et al. (2004) confirmed Thevanagam's viewpoint by performing a set of test [11]. Thevayanagam et al. (2002) offered a soil classification system based on contact density and the equivalent void ratios; $(e_c)_{eq}$ and $(e_f)_{eq}$, respectively, for sand mixed with silt content (FC) less than and more than a threshold value FC_{th} (Fig. 2), [12].
Based on necessity of considering and specifying the silt sand behavior, in this work some series of undrained static triaxial tests were done for verifying the generality of $(e_c)_{eq}$, with focusing on how to calculate *b* factor and its eventual relation with the sand physical characteristics.
In the first stage, the effect of low percentages of silt fine, on behavior of sand were studied. In the first stage, the rate of the changes of *e*, e_c and $(e_c)_{eq}$ against residual resistance of samples were considered and compared with each other. In the second series, the samples were prepared with a constant silt of 20%. In fact the difference between samples was only in their grading curves. The purpose was the study on generality of parameter $(e_c)_{eq}$ in describing the undrained behavior of silt sands.

2. MATERIALS AND METHODS

In this work, Babolsar coast and Malayer Shooshab river sands were used in experiments. The specific gravity of Shooshab and Babolsar sands were 2.67 and 2.7 respectively, according to ASTM [13]. Babolsar coast sand and Malayer Shooshab river sand were divided into triplet and septet different subgroups respectively. 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.

3. EXPERIMENTS PROCEDORES

3.1. Sample preparation technique

Previous investigations have indicated that sample preparation methods affect the liquefaction behavior of soils and thus the choice of a proper sample preparation technique is important in determining the liquefaction potential of sands [14], [15]. Among these methods, wet compaction technique has the advantage that it is relatively easy to control the global specimen density achieved, even for loose specimens [16]. In this work, wet compaction technique has been used for preparation of samples. All samples 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.

Table-1. *Description of subgroup – sand samples used in the experiments*

Upper sieve (ASTM)	Lower sieve(ASTM)	Subgroup name
No.10	No.80	No.10-No.80
No.40	No.200	No.40-No.200
No.60	No.200	No.60-No.200
No.80	No.200	No.80-No.200
No.40	No.80	No.40-No.80
No.200	No.400	Silt

3.2. Initial degree of saturation

For samples preparation, dry Sand and silt have been mixed with respect to the considered different weight ratios. To prevent the grains separation, while soil is pouring in the mould and for achieving to the desired density easily, for all other samples, the initial moisture percentage is fixed constant equal to 8%. The required amount of dry soil mass and water for each layer of samples have been determined exactly.

3.3. Final degree of saturation

After taking necessary measurements, the samples have been first subjected by CO_2 at least for 30 min and then saturated by de-aired water. The control of saturation degree is done by means of Skempton's pore pressure parameter *B*. According to JGS and ASTM, samples have been considered to be fully saturated if *B* is at least equal or greater than 0.95 [17], [18]. In this study, backpressure of 150 kPa has been applied during the tests to achieving the saturation state.

3.4. Consolidation and loading

After the saturation process, the samples were subjected to the confining stress for consolidation. During consolidation, the difference between confining pressure and backpressure has been arranged such that for each sample 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, silt content initial density and confining stress. In present work the effects different factor were studied for a given confining stress. The choice of 300kPa for confining stress as a mean value in geotechnical practice purposes was based on various research works existing in literature such as Ishihara [3]. After consolidation step and calculation of e_0 (e.g. void ratio after consolidation), axial load has been applied on sample in constant strain rate manner.

4. RESULTS OF TESTS

4.1. First series of tests

For the first series of tests, No40-No80 subgroup of Babolsar sand whose grain size distribution curve has been presented in Fig. 3a, 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 e_c in undrained behavior of silt sands.

To achieve this purpose, samples have been prepared at the state of constant e_c. In Table 2, total void ratio and the corresponding e_c for samples have been presented.

Table-2. *Summary of samples characteristics*

Silt percentage	*e*	e_c	Residual stress
0	0.991	0.991	0
5	0.904	1.004	35
10	0.808	1.009	190
15	0.743	1.050	258
15	0.712	1.014	-
20	0.629	1.036	-

Deviator stress (σ_1-σ_3) versus axial strain curves for different percentages of silt have been shown in Fig. 5. As shown, with constant e_c in all samples, the undrained residual resistance has been significantly improved due to increase of silt percentage. This is according to the result of Thevanayagam et al. saying the soil liquefaction resistance will be improved by increasing in silt content for a given constant e_c [9]. These results indicate consequently that e_c alone can not be capable of describing undrained behavior of sand mixed with non-plastic fine.

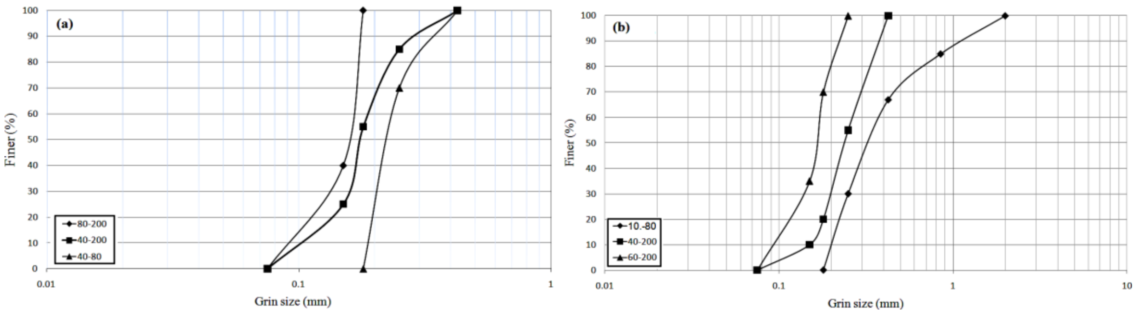

Figure 3. *Grain size distribution curves of sand for first and second series of tests; (a) Babolsar sand; (b) Shooshab sand*

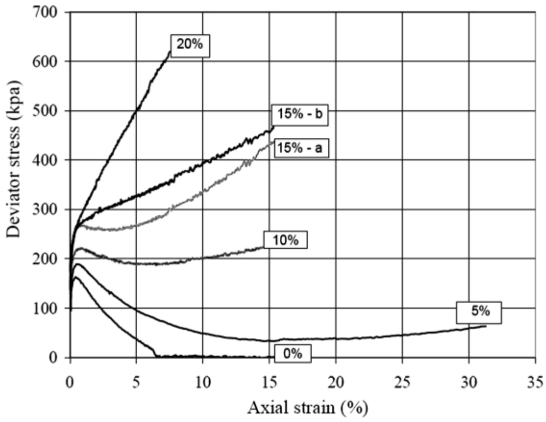

Figure 4. *Stress-strain relationship of samples from Babolsar sand, with the same e_c*

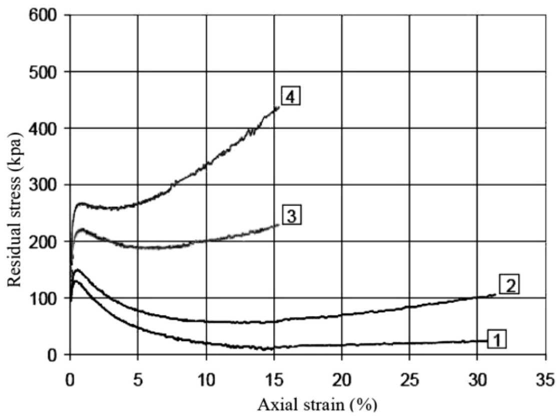

Figure 5. *Stress-strain relationship of samples from Babolsar sand, with non-zero residual resistance*

In the second stage the role of parameter $(e_c)_{eq}$ proposed by Thevanayagam et al. [10], has been reviewed. For this a set of undrained triaxial tests were performed on Babolsar coast sand mixed with different percentages of silt (See Table 3). Stress-strain curves for samples with non-zero residual resistance have been shown in Fig. 5. For calculating the parameter $(e_c)_{eq}$ corresponding to each sample, it is necessary to consider a value for parameter *b* in equation 3.
To achieve this purpose, two samples of Babolsar sand with different silt percentages (0 and 10%) and with similar total void ratio have been prepared. Stress-strain curves of these two

samples have presented in Fig. 6. As shown in this figure, the curves have been overlapped reasonably. With assumption that $(e_c)_{eq}$ can control undrained behavior of silty sand at the state of $FC<FC_{th}$, based on equation 3, the $(e_c)_{eq}$ values of both samples was equaled. Then, according to the following operations, a value 0.6 is obtained for factor *b*.

$$\frac{0.991+(1-b)\times 0}{1-(1-b)\times 0}=\frac{0.991+(1-b)\times 0.1}{1-(1-b)\times 0.1} \tag{4}$$

Table-3. *Summary of samples characteristic exposed in fig. 6and 7*

Test number	**Silt percentage**	***e***	e_c	$(e_c)_{eq}$	**Residual Stress (kPa)**
5	0	0.991	0.991	0.991	0
6	10	0.911	1.123	0.991	0
1	20	0.828	1.285	0.987	13
2	20	0.751	1.189	0.903	57
3	10	0.808	1.009	0.883	190
4	15	0.743	1.050	0.854	257

$(e_c)_{eq}$ values of the samples in Figs .5 and 6, with respect to obtained b factor, were calculated. The calculated values of $(e_c)_{eq}$ accompanied with *e* and e_c for samples have been presented in Table 3. According to Fig. 7, there is a very good correlation between undrained residual resistance and $(e_c)_{eq}$. It is clear that, the section of correlation curve below e_c axis has not physical meaning for Triaxial compression test. In conclusion, the obtained results confirm the works of Thevanayagam et al. [10], in that if a given sand is mixed with different percentages of silt, $(e_c)_{eq}$ can predict the liquefaction resistance at the state of $FC<FC_{th}$. The question which remains is whether $(e_c)_{eq}$ can be considered as a general quantity for controlling undrained resistance of silt sand at state of $FC<FC_{th}$ the answer to which is the main purpose of second series of tests.

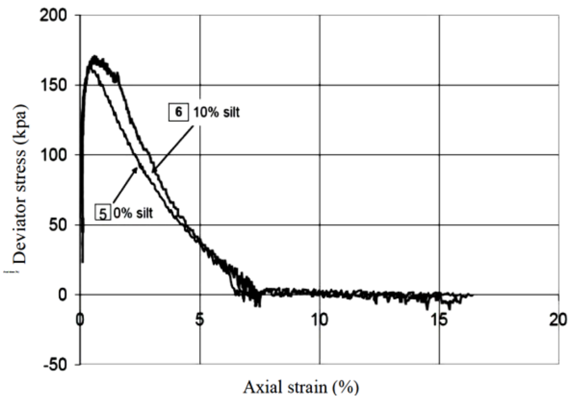

Figure 6. *Stress-strain relationship of two samples from Babolsar sand, with the same behavior*

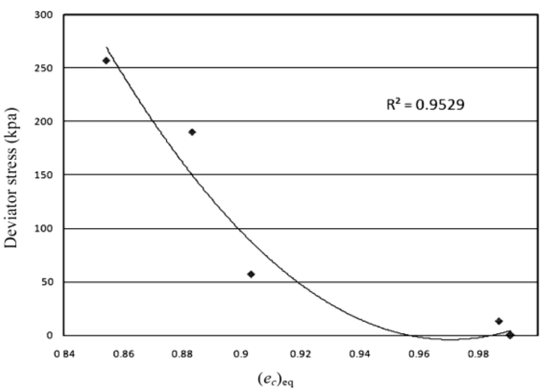

Figure 7. *Scatter diagram: residual stress versus* $(e_c)_{eq}$

4.2. Second series of tests

Three types of subgroups of Babolsar sand (No.40-No.200, No.40-No.80 and No.80-No.200) have been used in the second series of tests. Their grading curves have been shown in Fig. 4a. In this stage, all of the samples have same silt content equal to 20 percent. Stress-strain curves for these samples are shown in Fig .8 and also samples characteristics are shown in Table 4. Factor *b* for No.40-No.200 and No.80-No200 subgroups are calculated based on *b* factor obtained of No.40-No.80 subgroup and the fact that *b* is dependent on R_d ratio. It should be noted that b has an inverse relation with R_d. On the other hand, one kind of silt was used for all samples. So, we assume there is a linear relation between *b* and D_{50}. According to the work of Thevanayagam et al. [9], it is expected that samples with the same $(e_c)_{eq}$ values, have approximately the same behavior at the state of $FC<FC_{th}$. The generality of this idea is verified by the results shown in Fig. 9. The sample with $(e_c)_{eq}=0.82$ has residual resistance equal to 50kPa and the sample with $(e_c)_{eq}=1.02$ in strain equal to 8% yields to the complete liquefaction. But two samples (No.40-No.200 and No.40-No.80) have not the same values of $(e_c)_{eq}$ while their behaviors are similar. It shows that b factor, besides of R_d depends on the other factors and therefore this method is not suitable for calculation of b factor.

Because results are not limited to specific sand, similar tests were performed on samples of Shooshab sand with different grading curves (Fig. 3b). Samples characteristic have been presented in Table 4. Stress-strain curves of these samples are shown in Fig. 9. According to these results, samples with smaller sand (No.60-No.200), silt percent of 20 and less $(e_c)_{eq}$ have most resistance. While sample with larger sand (No.10-No.80), similar to the other samples with same silt percentage but larger value of $(e_c)_{eq}$, has least resistance. This is according to the work of Thevanayagam et al. [9]. Note that, there is very little difference between value of $(e_c)_{eq}$, while samples have completely different behavior to each other.

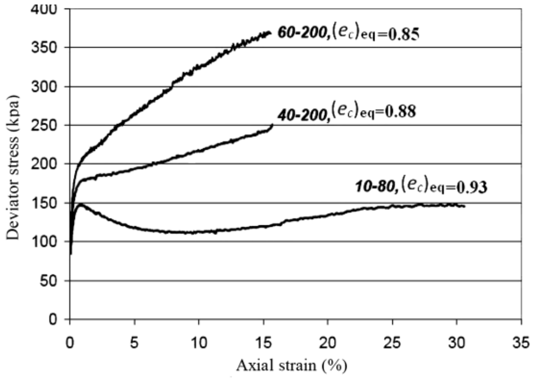

Figure 8. Stress-strain relationship of three samples from Shooshab sand, with the different grain size distribution curves and $(e_c)_{eq}$

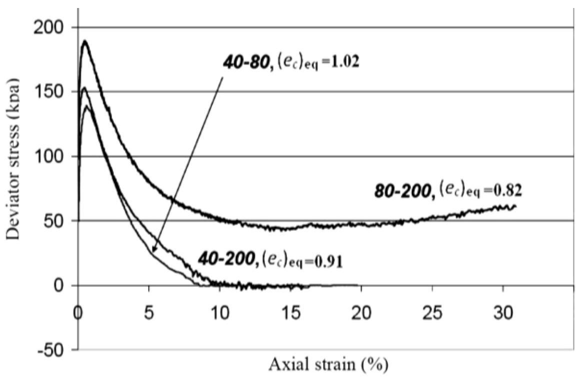

Figure 9. The difference behavior among of three samples from Babolsar sand, with the different grains size distribution curves and $(e_c)_{eq}$

Table-4. *Properties of samples*

Tests series	Kind of sand	kind of subgroup	D_{10}	D_{30}	D_{50}	D_{60}	C_C	C_u	e	b	fc	$(e_c)_{eq}$	Φ_{cs}	Residual Stress (KPa)
Second	Babolsar	40-80	0.19	0.21	0.22	0.23	1.21	1.01	0.86	0.60	0.2	1.02	27.3	0
		40-200	0.11	0.16	0.17	0.19	1.73	1.22	0.84	0.78	0.2	0.93	28.03	0
		80-200	0.09	0.13	0.16	0.17	1.89	1.10	0.78	0.83	0.2	0.84	33.4	45
Second	Shooshab	10-80	0.2	0.25	0.31	0.37	1.85	0.84	0.71	0.43	0.2	0.93	34.54	112
		40-200	0.15	0.2	0.23	0.26	1.73	1.03	0.72	0.57	0.2	0.88	35	—
		60-200	0.10	0.13	0.16	0.17	1.7	0.99	0.79	0.83	0.2	0.85	36.95	—

5. CONCLUSIONS

In this study, results of undrained static triaxial tests prove that if the type of material (sand) does not change, and the silt percentage changes $(e_c)_{eq}$ can control undrained residual resistance of samples at the state of $FC<FC_{th}$. This means that the sample with less value of $(e_c)_{eq}$ has most liquefaction resistance compared to the sample with more value of $(e_c)_{eq}$, regardless their silt percentages. But results of this study indicate that, for $(e_c)_{eq}$ to become a general quantity to control the liquefaction resistance of silty sand, it must be redefined. In fact according to the results of present paper, if we would like $(e_c)_{eq}$ as a general quantity to control undrained resistance of silty sand in state of $FC<FC_{th}$, it would require b factor in $(e_c)_{eq}$ equation which is

related to not only physical parameters (such as identification and grading parameters) but also to the mechanical parameters specifically internal friction angle at critical state.

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