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Effect of physical parameters on static undrained resistance of sandy soil with low silt content

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ABSTRACT

Previous researches concerning the behavior of sand mixed with non-plastic fine show that the void ratio related to sand grains (e_c) plays a more important role in comparison with the total void ratio, where soil undrained 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 cannot 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 $(e_c)_{eq}$.

1. Introduction

It is 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 [1,2] 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 [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 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 [4], while others had dissenting opinion [5,6], but Yamamuro and Lade [7] 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 (Fig. 1). 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

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

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 increases due to the increase in fc that creates more distances between sand grains.

Polito [8] modified viewpoint of Yamamuro and Lade [7] 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. 2). Thevanayagam et al. [9] 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

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soil bearing skeleton (Fig. 3). 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. [9] 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. Therefore, Thevanayagam et al. [9] modified e_c and presented $(e_c)_{eq}$ as follows:

$$(e_c)_{eq} = (e+(1-b)\times fc)/(1-(1-b)\times fc) \quad 0<b<1 \tag{3}$$

where b is portion of the silt fines that contribute to the active inter-grain contacts. According to Thevanayagam et al. [9]; $b=0$ 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. b value is dependent on the grain characteristics and grain size disparity ratio $Rd=D/d$, 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. [10] confirmed Thevanagam's viewpoint by performing a set of test. They showed that for a given sand, $(e_f)_{eq}$ and $(e_c)_{eq}$ are appropriate quantities for controlling the behavior of silt–sand mixtures at the states of $FC > FC_{th}$ and $FC < FC_{th}$, respectively.

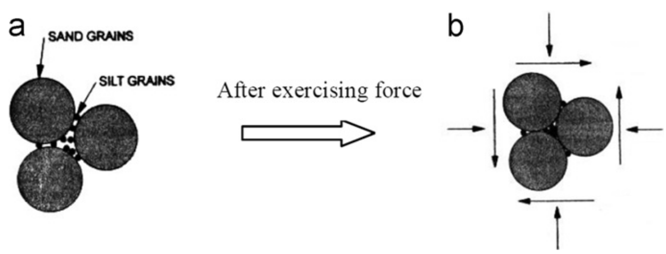

Fig. 1. Schematic diagram showing silty sands deposited in a loose state with larger grains separated by silt fine (after [7]).

Based on the 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 $(e_c)_{eq}$, 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 was studied. In the first stage, the rate of the changes of e, e_c and $(e_c)_{eq}$ against residual resistance of samples was 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 to study the generality of parameter $(e_c)_{eq}$ in describing the undrained behavior of silt sands.

The strength of granular material is most often referred to in terms of the friction angle. It has long been recognized [11,12] that an increase in the proportion of coarse material in an otherwise fine-grained granular soil can result in an increase in friction angle. Alternatively, when the voids within a coarse granular material are filled with fines, its friction angle is increased [13].

The value of the friction angle is function of the following:

- Particle size distribution (reducing with decrease in particle size).

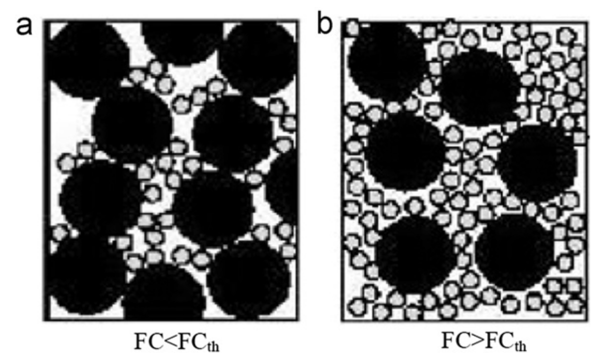

Fig. 3. Mixture of sand and silt intergranular classification (after [7]).

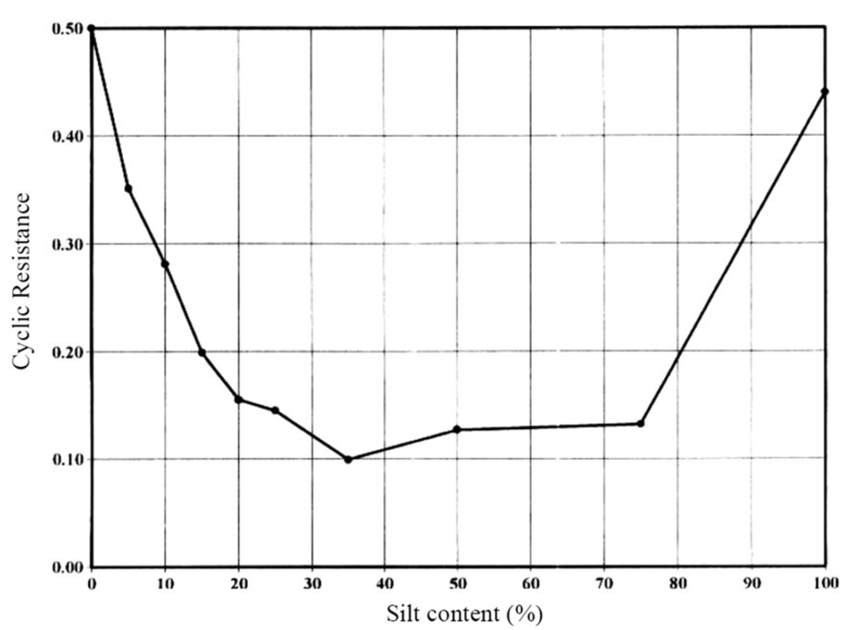

Fig. 2. Variation in cyclic resistance with silt content for Monterery sand at constant void ratio of 0.68 (after [8]).

- Particle shape and surface roughness (increasing with increase in angularity and surface roughness).
- Strength and specific gravity of individual particles.
- State of packing (increasing with increase in density).
- Applied stress level (decreasing with increase in stress, resulting in a curved strength envelope passing through the origin).

In the third and fourth stages, the effects of grain mechanical properties such as internal friction angle in critical state (Φ_{cs}) on the undrained resistance of various types of sand mixed with non-plastic fine were studied.

Table 1
Description of subgroup—sand samples used in the experiments.

Upper sieve (ASTM)	Lower sieve (ASTM)	Subgroup name
No. 10	No. 20	No. 10–No. 20
No. 10	No. 60	No. 10–No. 60
No. 10	No. 80	No. 10–No. 80
No. 10	No. 200	No. 10–No. 200
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

2. Materials tested

In this work, Babolsar coast and Malayer Shooshab river sands were used in experiments. The specific gravity of Shooshab and Babolsar sands was 2.67 and 2.7, respectively, according to ASTM D 854-02 [14]. 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 procedures

3.1. Sample preparation technique

Previous investigations have indicated that sample preparation methods affect the liquefaction behavior of soils [15,16] and thus the choice of a proper sample preparation technique is important in determining the liquefaction potential of sands. Current field sampling techniques are not readily able to produce high-quality undisturbed granular soil specimens for laboratory testing at an affordable cost. Accordingly, numerous sample reconstitution methods have been developed for use in the

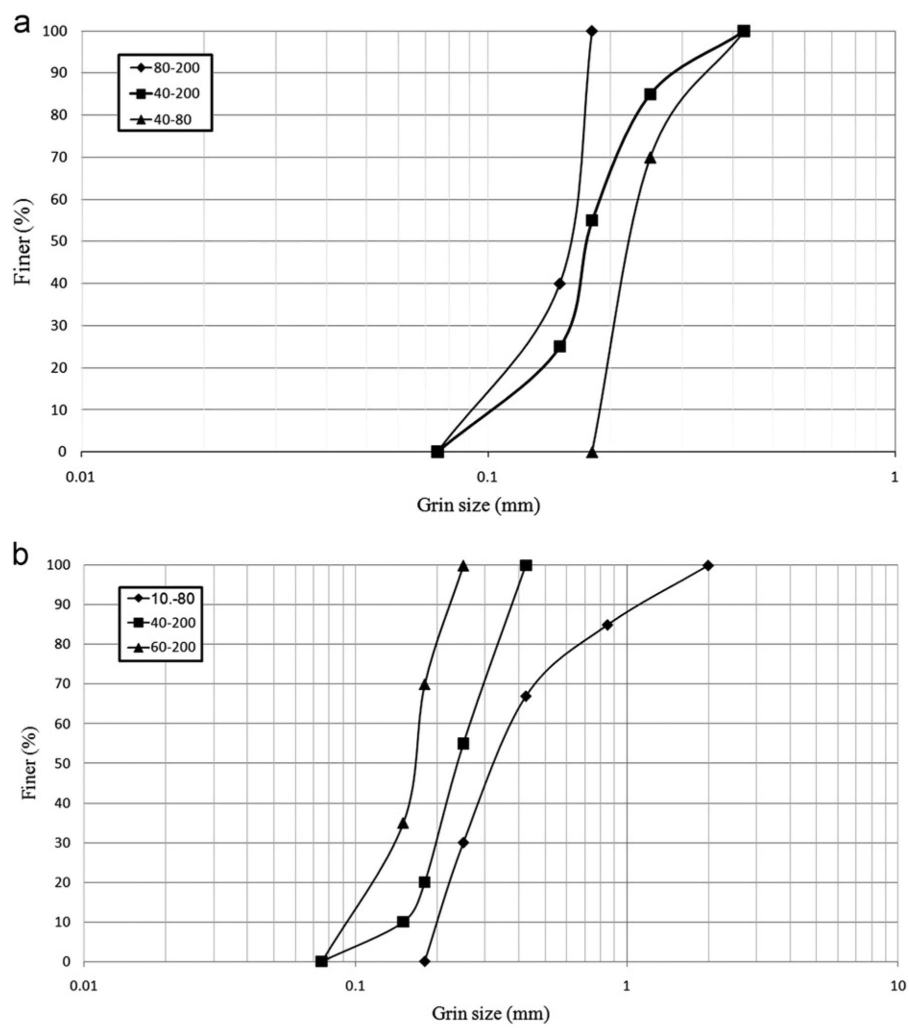

Fig. 4. Grain size distribution curves of sand for first and second series of tests: (a) Babolsar sand and (b) Shooshab sand.

laboratory. 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 [17]. 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, and hence height to diameter ratio of 2 is kept constant.

3.2. Initial degree of saturation

For sample preparation, dry sand and silt have 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 dry soil mass and water for each layer of samples has been determined exactly.

3.3. Final degree of saturation

After doing necessary measurements, the samples have been first subjected by CO_2 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 [18] and ASTM D 4767-92 [19], samples have been considered to be fully saturated if B is at least equal to or greater than 0.95. In this study, backpressure of 150 kPa has been applied during the tests to achieve 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 the present work the effects of different factors were studied for a given confining stress. 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 [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, No. 40–No. 80 subgroup of Babolsar sand whose grain size distribution curve has been presented in Fig. 4a was selected. Then, this soil was mixed with different

Table 2
Summary of samples characteristics.

Silt percentage	e	e_s	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	–

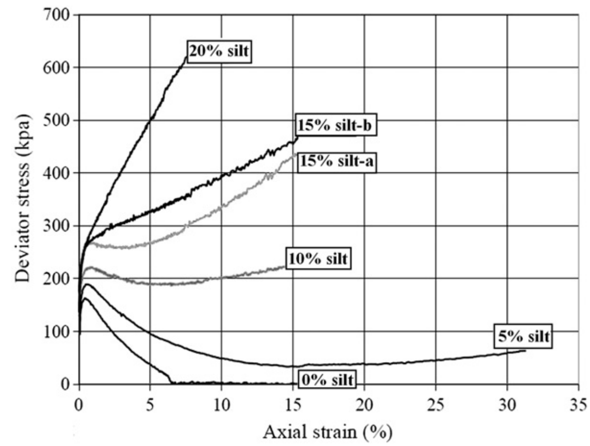

Fig. 5. Stress–strain relationship of samples from Babolsar sand with the same e_c.

Table 3
Summary of samples characteristic exposed in Figs. 6 and 7.

Test number	Silt percentage	e	e_s	$(e_c)_{eq}$	Residual stress
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

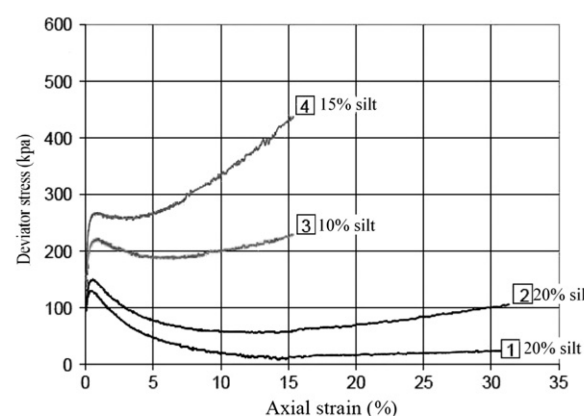

Fig. 6. Stress–strain relationship of samples from Babolsar sand with non-zero residual resistance.

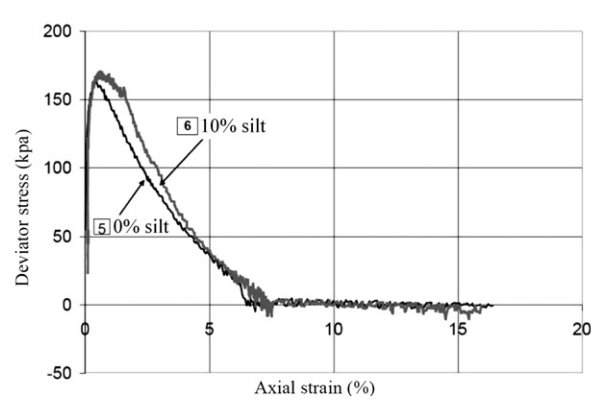

Fig. 7. Stress–strain relationship of two samples from Babolsar sand with the same behavior.

percentages of silt (0%, 5%, 10%, 15% and 20%). In this stage, the purpose is to verify different ideas mentioned in Section 1 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. Deviator stress $(\sigma_1-\sigma_3)$ 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. [9] saying the soil undrained resistance will be improved by the increase in silt content for a given constant e_c. These results indicate consequently that e_c alone cannot be capable of describing undrained behavior of sand mixed with non-plastic fine. In the second stage, the role of parameter $(e_c)_{eq}$ proposed by Thevanayagam [20] 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. 6. To calculate the parameter $(e_c)_{eq}$ corresponding to each sample, it is necessary to consider a value for parameter b in Eq. (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 are presented in Fig. 7. As shown in this figure, the curves have been overlapped reasonably. Assuming that $(e_c)_{eq}$ can control undrained behavior of silty sand at the state of $FC<FC_{th}$, based on Eq. (3), the $(e_c)_{eq}$ values of both samples became equal. Then, according to the following operations, the value 0.6 is obtained for factor b:

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

$(e_c)_{eq}$ values of the samples in Figs. 6 and 7, with respect to obtained b factor, were calculated. The calculated values of $(e_c)_{eq}$ accompanied by e and e_c for samples have been presented in Table 3. According to Fig. 8, 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 does not have physical meaning for Triaxial compression test. In conclusion, the obtained results confirm the works of Thevanayagam [20], in that if a given sand is mixed with different percentages of silt, $(e_c)_{eq}$ can predict the undrained 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.

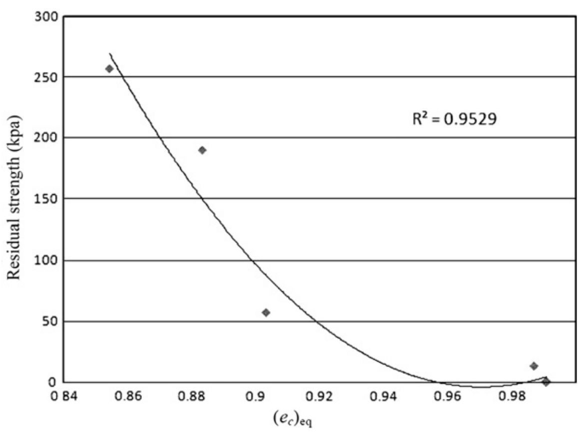

Fig. 8. Scatter diagram: residual stress versus $(e_c)_{eq}$.

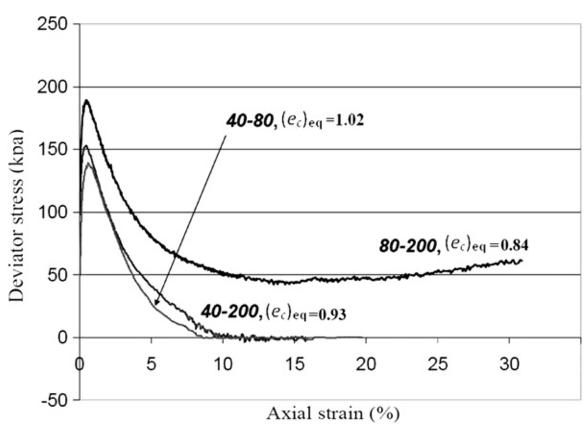

Fig. 9. The difference behavior among of three samples from Babolsar sand, with the different grains size distribution curves and $(e_c)_{eq}$.

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%. Stress–strain curves for these samples are shown in Fig. 9 and also samples characteristics are shown in Table 4. Factor b for No. 40–No. 200 and No. 80–No. 200 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

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	–
Third	Shooshab	40–200	0.12	0.18	0.22	0.25	2.08	1.08	0.86	0.6	0.15	0.99	34.85	283
		60–200	0.11	0.16	0.20	0.21	1.91	1.11	0.86	0.66	0.15	0.97	35.73	330
		80–200	0.09	0.11	0.13	0.15	1.67	0.90	0.86	1.02	0.15	0.86	38.21	405
Fourth	Shooshab	10–20	0.97	1.20	1.43	1.55	1.60	0.96	0.81	0.09	0.15	1.10	34	130
		10–60	0.36	0.52	0.66	0.77	2.14	0.98	0.81	0.20	0.15	1.06	34.41	198
		10–200	0.19	0.27	0.34	0.41	2.16	0.94	0.81	0.39	0.15	0.99	36.31	350

and D_{50} (average grain diameter of sand). According to the work of Thevanayagam et al. [9] and Thevanayagam and Martin [21], 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 50 kPa 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.

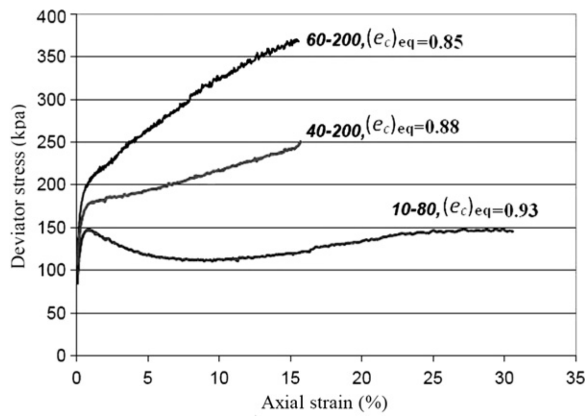

Fig. 10. Stress–strain relationship of three samples from Shooshab sand, with the different grain size distribution curves and $(e_c)_{eq}$.

Because results are not limited to specific sand, similar tests were performed on samples of Shooshab sand with different grading curves (Fig. 4b). Samples characteristic have been presented in Table 4. Stress–strain curves of these samples are shown in Fig. 10. According to these results, samples with smaller sand (No. 60–No. 200), silt 20% and less $(e_c)_{eq}$ have the greatest 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.

4.3. Third series of tests

In the third series of tests, three types of subgroups of Shooshab sand (No. 40–No. 200, No. 60–No. 200 and No. 80–No. 200) have been used whose grading curves are shown in Fig. 11a. Samples were prepared with the same silt content equal to 15% and at the constant total void ratio. Samples characteristics have been presented in Table 4. Factor b was calculated based on factor b corresponding to No. 40–No. 80 subgroup of Babolsar sand and to the fact that b is dependent on R_d ratio. Difference among samples is in their grading at this stage. Results of the third series of tests have been shown in Fig. 12. No. 40–No. 200 subgroup sample with a non-uniform grains in comparison with the two

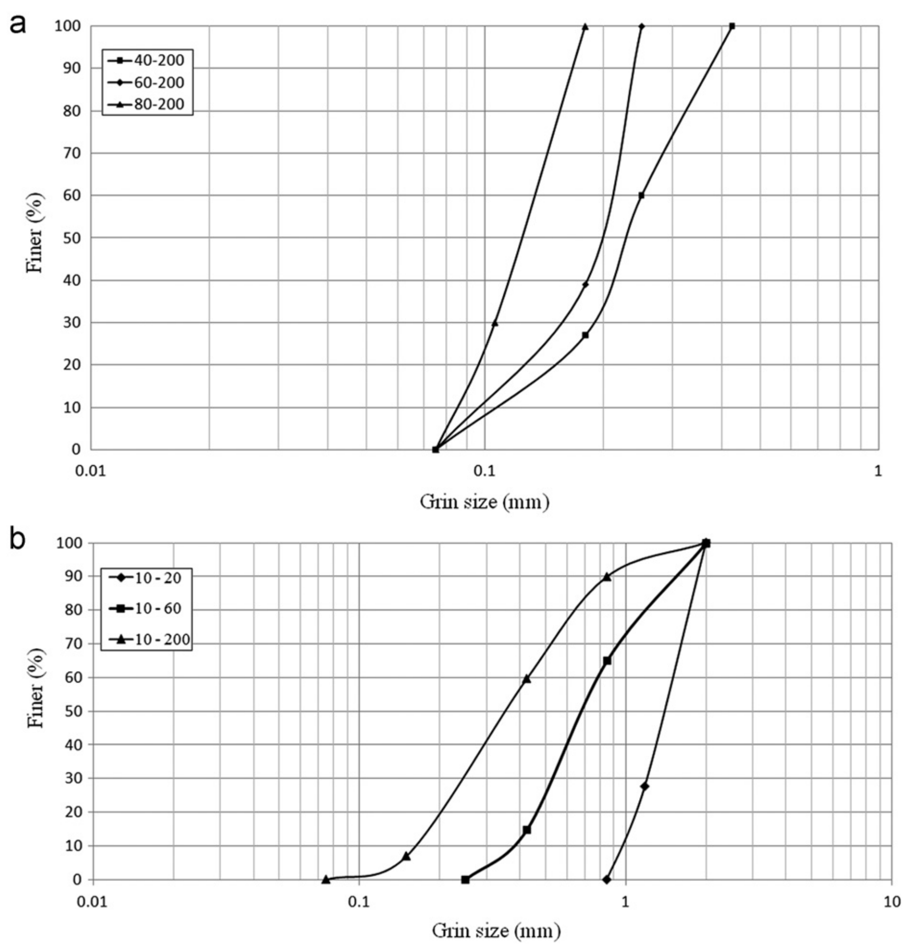

Fig. 11. Grain size distribution curves of sand for: (a) third series of tests and (b) fourth series of tests.

other samples has the lowest ***undrained resistance and No. 80–No. 200 subgroup sample with finer and uniform range of grains size has the highest undrained resistance. The reason behind this is that No. 80–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 (Fig. 13). In fact, for sand with non-uniform grains size, least values of silt content lead to sand grains separation, but for 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. There is very little difference between the values of $(e_c)_{eq}$, while behavior of samples is different from one another.

4.4. Fourth series of tests

In this stage, three types of subgroups of Shooshab sand (No. 10–No. 20, No. 10–No. 60 and No. 10–No. 200) with the grading curves presented in Fig. 11b 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 4. The basic difference between the samples prepared in this stage is in their grading. Results of fourth series of tests are shown in Fig. 14. As it is shown in this figure, No. 10–No. 200 subgroup sample with smaller and non-uniform sand grains in comparison with other samples has the highest undrained resistance and No. 10–No. 20 subgroup sample with greater and uniform

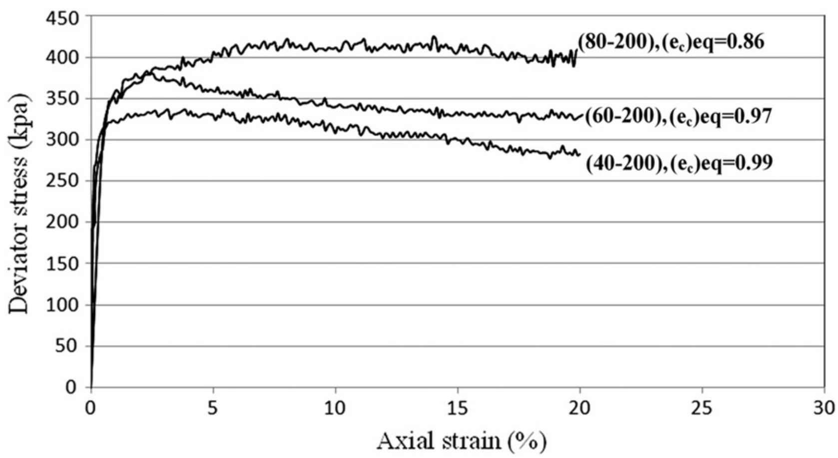

Fig. 12. Stress–strain relationship of three samples from Shooshab sand with the different grain size distribution curves and same *e*.

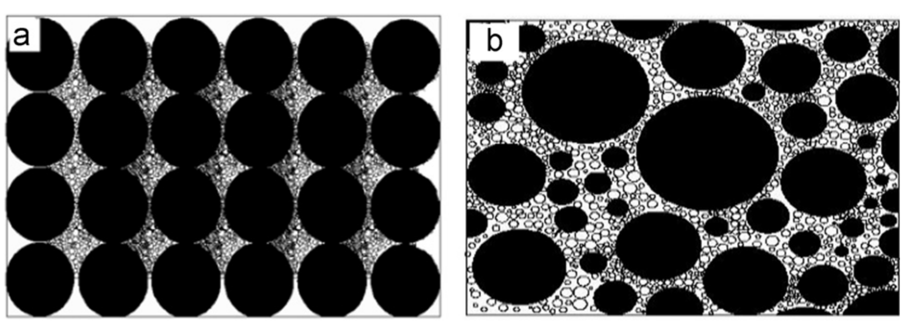

Fig. 13. Sand–soil mixtures with the different grading: (a) sample with uniform sand grains and (b) sample with non-uniform sand grains.

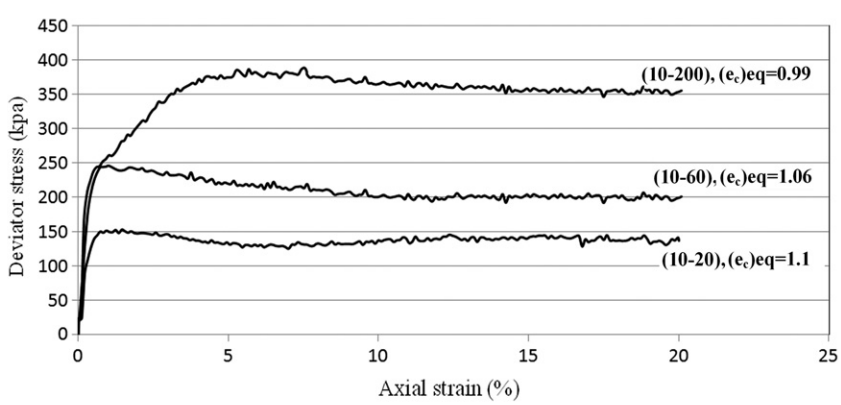

Fig. 14. Stress–strain relationship of three samples from Shooshab sand with the different grain size distribution curves and same *e*.

sand has the lowest undrained resistance. As it was mentioned in the third series of tests, the results of present stage also indicate that for the small variations of $(e_c)_{eq}$ there is a significant difference between stress–strain curves. For example, No .40–No. 200 and No. 10–No. 200 samples with the same value of $(e_c)_{eq}$ present different behaviors. This difference is 50 kPa at the peak state and 70 kPa at the critical state. These results show, to have a more general form for $(e_c)_{eq}$, it is necessary to introduce the effect of invariant natural characteristics (that do not change due to density, confining stress and loading rate) of soil in its expression. An eventual correlation can be deduced between b parameter and physical (grading curve characteristics for example) or mechanical parameters (critical state internal friction angle particularly) of soil. It should be noted that critical state internal friction angle is independent from density and confining stress. On the other hand, Φ_{cr} depend on type (mineralogy) and physical characteristics such as grading and shape of particles of soil. There is certain correlation relationship between Φ_{cr} and physical characteristics of soil, after Favre [22].

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 the most undrained 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 undrained 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 to define b factor in $(e_c)_{eq}$ equation in a way to be related not only to physical parameters (such as identification and grading parameters) but also to the mechanical parameters that take into account the inter-grain frictional properties of the soil. In this regard the internal friction angle at critical state that is generally independent of density and confining pressure is proposed as a mechanical parameter; however its relation with b factor should be investigated as a future study.

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