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A laboratory study on the pore pressure generation model for Firouzkooh silty sands using hollow torsional test

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Abstract

This paper discusses the applicability of a simple model to predict pore water pressure generation in non-plastic silty soil during cyclic loading. Several Stress-controlled cyclic hollow torsional tests were conducted to directly measure excess pore water pressure generation at different levels of cyclic stress ratios (CSR) for the specimens prepared with different silt contents (SC=0% to 100%). The soil specimens were tested under three different confining pressures ($\sigma'3$= 60, 120, 240 kPa) at a constant relative density (Dr=60%), with different silt contents. Results of these tests were used to investigate the behavior of silty sands under undrained cyclic hollow torsional loading conditions. In general, beneficial effects of the silt were observed in the form of a decrease in excess pore water pressure and an increase in the volumetric strain. Modified model for pore water pressure generation model based on the test results are also presented in this paper. Comparison of the proposed pore pressure build up model with seed's model indicates the advantage of proposed model for soil with large amount of silt.

Keywords: Liquefaction; Hollow torsional cylinder test; Firouzkooh silty sand; Cyclic stress ratio; Pore water pressure

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. The 1964 Niigata Earthquake caused dramatic damages due to liquefaction, and thus led to a significant acceleration in the liquefaction research. The generation of pore pressures in soils during cyclic loadings, such as earthquakes or pile driving induced loadings, has been studied for many years [1-3]. In particular, the cyclic behavior of fine-grained soils has received considerable attention after the 1999 Kocaeli earthquake in Turkey, in which these soils exhibited different behavior than previously considered by researchers. Initial research efforts [4-7] focused mostly on investigations of clean sands and the factors that most affected the liquefaction resistance of these soils. However, most natural and artificial (e.g., hydraulic fills) sand deposits contain some silts, and most sites, experienced liquefaction during previous earthquakes, have contained some percentage of silts [7-9]. Therefore, liquefaction investigations over the last decade have focused more on the influence of silts.

Previous laboratory liquefaction studies, concerning on the effect of silts on liquefaction susceptibility, have not yet reached a consensus.

The pore pressure generation model introduced by Seed et al. [2] for sands has been tested by many researchers [10-11] and has showed satisfactory agreement. On the other hand, pore water pressure generation characteristic of silty soils, when containing a small amount of clayey particles, show much different trend from that of sands [12-15]. The behavior of non-plastic silty soils with various amounts of silts is not clear. It should be noticed that misinterpretation of the pore pressure generation rate may result in underestimating the liquefaction-induced hazards. Therefore, the pore pressure pattern of non-plastic silty soils should be determined carefully so that the liquefaction mitigation programs such as sand/stone columns can be used more precisely. While most previous research efforts have focused on clean sands, yet sand deposits with silts are more commonly found in nature. This research presents an investigation on the effect of silt content on excess pore water pressure generation in silty sands.

2. Literature review

The literature regarding the effect of silts on pore water pressure generation and cyclic resistance reveals that there is no consensus among the researchers as to how silts affect the cyclic resistance

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and that there is little information about the generation of excess pore water pressure in liquefiable silty soils. Dobry [13] indicated that pore water pressure generation as a function of shear strain and number of loading cycles for sands represented a wide range. He obtained these data from strain-controlled, undrained triaxial tests on seven different sands. Various specimen preparation techniques were used to reconstitute the sand specimens at relative densities ranging from 20% to 80%. Singh [14] studied the effect of non-plastic fines on the cyclic resistance of silty sand specimens reconstituted at a constant relative density of 50%. The results of the stress-controlled, cyclic triaxial tests showed a decrease of about 25% in cyclic resistance for 20% FC, followed by a slight increase for larger FCs. At 100% fines, the cyclic resistance was still smaller than that of clean sand. Also Singh [14] showed that for a given relative density sands containing 10, 20 or 30% of silt by weight have lesser resistance to liquefaction than clean sand. Singh explained that the loss of resistance appears to be consistent when results are compared in terms of void ratio. Erten and Maher [15] showed that there is little pore water pressure generation in silty soils if the strain level is less than the threshold value of the order of 0.01% that is similar to those observed for sand. The pore pressure generation is increased with the increase of silt content up to 30%, if compared at the same void ratio. Amini and Qi [16] indicated that the liquefaction resistance of silty sands decreased as the confining pressure increased and the increase in silt content caused the liquefaction resistance of silty sands to increase. They reported an increase in cyclic resistance with increasing silt content at a constant overall void ratio. Polito and Martin [17] used Monterey #0/30 sand and non-plastic silt, and measured the cyclic resistance from stress-controlled, cyclic triaxial tests for specimens with silt contents ranging from 0% (clean sand) to 100% (pure silt). The soil specimens were prepared at a constant overall void ratio of 0.68, which corresponds to 61% relative density of the clean sand. In above works, void ratio, and relative density or intergrain void ratios were used as controlling parameters to characterize the behavior of silty sands. In this study, the test results are analyzed based on the constant relative density. The previously published studies on the effect of silt content on cyclic liquefaction resistance do not present a unified picture [18-22]. The laboratory studies that focused on specimens at a constant overall void ratio generally showed a decrease in liquefaction resistance with increasing silt content, while those that focused on specimens at a constant sand skeleton void ratio. showed an increase in resistance or a constant resistance [18-22]. Derakhshandi et al. [23] investigated the effect of plastic fines on the pore pressure generation characteristics of saturated sands. They indicated that the pore pressure response, as characterized by strain-controlled testing, of sand-clay mixtures can be explained by the relative values of the sand-skeleton void ratio and the maximum void ratio of the clean sand. For sand-skeleton void ratios smaller than the maximum void ratio of the clean sand, the sand matrix dominates and the soil response is sand-like. When the sand-skeleton void ratio is larger than the maximum void ratio of the clean sand, the fines matrix dominates the soil structure and the soil responds like clay. Polito et al. [24], discussed the applicability of two simple models for predicting pore water pressure generation in non-plastic silty soil during cyclic loading. The first model was developed by Seed et al. in the 1970s and relates the generated pore pressure to the cycle ratio, which is the ratio of the number of applied cycles of loading to the number of cycles required to cause liquefaction. The second model was the Green-Mitchell-Polito model proposed by Green et al. [25, 31], which relates pore pressure generation to the energy dissipated within the soil. Those pore pressure generation models differ significantly for soils containing less and greater than 35% fines, respectively, consistent with the limiting fines content concept. Hazirbaba and Rahtje [26] studied the effect of fines content on excess pore water pressure generation in sands and silty sands. They performed a complete and comprehensive review on the research on the effects of fine-grained non-plastic cyclic resistance by various researchers. They showed that effects of fines content were observed in the form of a decrease in excess pore water pressure and an increase in the threshold strain. Direct observations from field case histories [18] have indicated an increase in liquefaction resistance with increasing silt content. The main issue with most of these tests is that they focused on the shear stresses required to cause liquefaction. Because silt content affects the stiffness of the soil, stress-controlled tests induce different levels of cyclic stress ratio (CSR) in specimens as the silt content changes. The goal of this study is to shed light on the conflicting results reported in the literature regarding the effect of silt content on the excess pore water pressure through the use of stress-controlled tests and to advance the current findings. In addition, in this paper a modified model based on hollow torsional test results for pore water pressure generation is presented.

3. Material description, sample preparation and procedures

The natural soil used in the present experimental investigation was taken from city of Firouzkooh north of Iran. To accomplish the objectives of this study, sands with varying silt content ranged from 0 to 100% (percent passing No. 200 sieve) were used. The soil properties, used for this study are presented in Table 1 and the grain size distribution curves for the soil samples are also shown in Fig. 1. Specimens with five different silt contents were prepared for the hollow torsional

Table 1. Physical Properties of Tested Materials.

No	Name	(USCS)	G_s	γ_d(gr/cm^3)	e_{max}	e_{min}	e	D_{50}	C_c	C_u
1	F0	SP	2.67	1.574	0.87	0.58	0.696	0.361	0.899	2.44
2	F15	SM	2.67	1.692	0.83	0.41	0.578	0.351	9.093	30.46
3	F30	SM	2.67	1.74	0.854	0.319	0.533	0.308	1.522	48.92
4	F60	SM	2.68	1.56	1.259	0.36	0.72	0.05	0.033	30.431
5	F100	ML	2.68	1.321	1.88	0.46	1.028	0.021	1.65	11.54

tests. The variations of maximum and minimum void ratios for samples with different silt content are also revealed in Fig. 2. A series of 60 tests were conducted on samples with the same relative density of 60% and the confining pressures of 60, 120 and 240 kPa and silt content of 15%, 30%, 60%, and 100%. The hollow torsional shear apparatus, used in this research, were provided by International Institute of Earthquake Engineering and Seismology (IIEES) of Iran. The hollow cylindrical samples had external and internal diameters of 100 and 50mm respectively and the height of 100mm. The under compaction method was implemented to prepare uniform samples [27]. The required parameters for specimen preparation, such as water content and percentage of under compaction values were selected as 9% and 4%, respectively. After preparation of the sample, the saturation process is started. All the specimens were isotropically consolidated under three effective confining pressures (σ'3) of 60, 120, 240 kPa. The intensity of the cyclic torsional load was varied in such a way to produce a wide range of cyclic stress ratios (CSR $=\iota_c/\sigma'3$) and corresponding number of cycles, Nl, applied to the specimens in order to cause initial liquefaction. Frequency of cyclic loading was 0.1 Hz and the numbers of cycles varied in the range of 0.17 to 900 cycles. It should be noted that, the initial liquefaction was assumed to occur when the excess pore water pressure became equal to the initial consolidation stress, σ'3 of the specimen (r_u=1) or double amplitude of 7.5% shear strain achieved. Cyclic hollow torsional tests were conducted on specimens with five different silt content (0, 15, 30, 60, 100%), three effective confining pressures (σ'3 = 60, 120, 240 kPa) and relative density of 60% (Dr = 60%). Continuous records of the excess pore water pressure, i.e., Δu, and cyclic torque, T, as well as the cyclic shear stress, ιc, were obtained during the cyclic torsional loading. Each test was continued until the initial liquefaction occurred. Post-liquefaction pore pressure dissipation tests were initiated immediately following the cyclic loading. The bottom end of the specimen was connected to a pressure controlled volume measuring burette (in the pressure panel). The top end of the specimen was connected to a pore pressure transducer with no drainage allowed from this end. This setup simulated a one-way drainage condition. The dissipation tests were done in three stages. The pore pressure at the top of the specimen, and outflow volume of water from the bottom of the specimen versus elapsed time were recorded in each stage. The duration of each stage varied from 16 sec to more than 3 hours, depending on the silt content of the specimen [28].

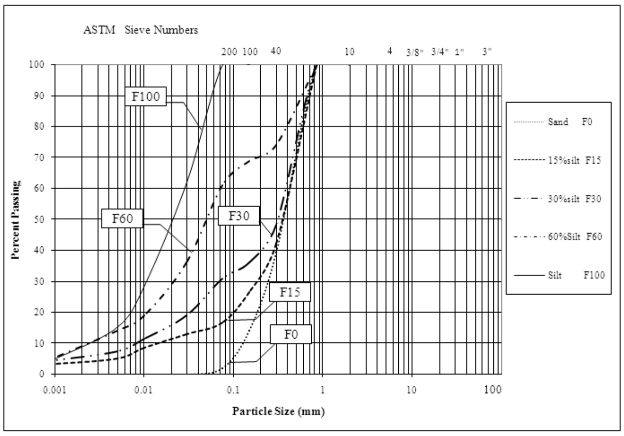

Fig. 1. Grain size distribution of natural soils samples used in the present study.

4. Results and analysis

4.1. Effect of silt content on void ratio and collapsibility of silty sands

Change of void ratio for samples with different silt content at different confining pressure is revealed in Figure 3. Different graphs in this Figure are for different confining pressures. It can be seen that the effective confining pressure and percentage of the silt content have important effect on the void ratio of specimens. By increasing the silt content (less than about 30%) at a constant confining pressure the void ratio decreases. However for the silt content more than SC_{th}=30% (SC_{th} is threshold fines content for change of behavior in silty sand), the void ratio increases with increase in silt content for all confining pressures.

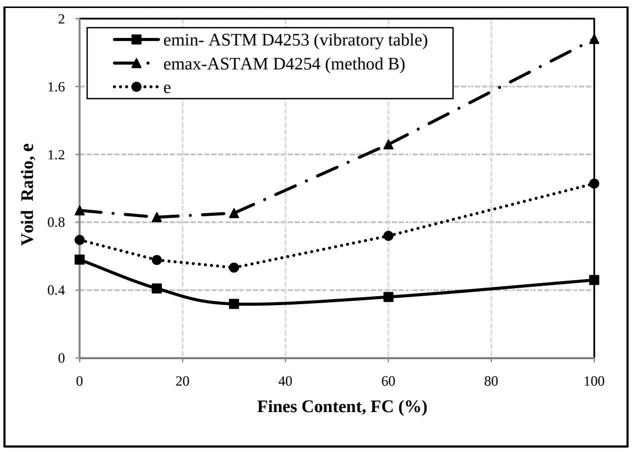

Fig. 2. Maximum and minimum void ratios versus fines content for Firouzkooh sand and non-plastic silt mixtures.

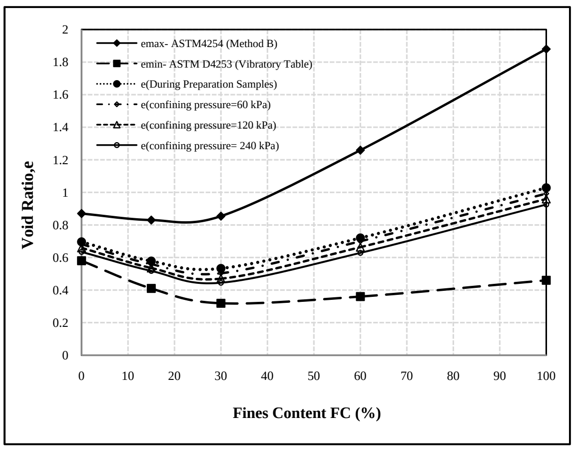

Fig. 3. Variation of void ratio for samples with different silt content at different confining pressures.

4.2. Variation of volumetric strain after cyclic loading

In this part the effect of silt content on volume change of soil specimen after cyclic loading and dissipation of excess pore water pressure will be discussed. Test conditions and results for the hollow torsional tests conducted in the current study is presented in Table 2 also Figure 4 shows a diagram of pore water pressure dissipation with time after stopping the cycle loading (after liquefaction). It can be seen in this figure that the rate of pore water pressure dissipation for specimens with more silt content is slower due to lower permeability of these specimens. Variation of volumetric strain for different specimens at different confining pressures after liquefaction is shown in Figure 5. Each reported volumetric strain in this graphs is based on results of four tests which were performed with different CSR (cyclic stress ratio). Figure 5 show that increasing confining pressure caused an increase in volumetric strain (εv) of specimen for all the percentage of fines content. It can be seen that also the rate of increase of volumetric strain with confining pressure for all specimens is almost constant. The change of cyclic stress ratio (CSR) versus the number of cycle causing liquefaction of the specimens with different silt

Table 2. Physical properties of tested materials.

row	σ_3 (kPa)	q (kPa)	FC (%)	eo	εv (%)	e1	Drc	N	row	σ3 (kPa)	q (kPa)	FC (%)	eo	εv (%)	e1	Drc	N
1	60	13.5	0	0.696	0.823	0.682	0.6481	136.4	31	120	24	30	0.533	1.287	0.513	0.6369	9.3
2	60	18	0	0.696	0.823	0.682	0.6482	75.2	32	120	26.2	30	0.533	1.370	0.512	0.6393	0.8
3	60	24	0	0.696	0.849	0.682	0.6497	15.4	33	240	30	30	0.533	1.658	0.508	0.6475	114.3
4	60	27	0	0.696	0.880	0.681	0.6514	2.6	34	240	36	30	0.533	1.726	0.507	0.6494	35.0
5	120	31.4	0	0.696	1.251	0.675	0.6732	18.6	35	240	48	30	0.533	1.980	0.503	0.6567	2.8
6	120	40.2	0	0.696	1.624	0.668	0.6950	7.4	36	240	69.8	30	0.533	2.335	0.497	0.6669	0.3
7	120	48	0	0.696	1.641	0.668	0.6960	2.8	37	60	12	60	0.720	0.964	0.703	0.6180	175.0
8	120	60	0	0.696	1.780	0.666	0.7041	1.6	38	60	18	60	0.720	1.167	0.700	0.6219	36.0
9	240	48	0	0.696	1.641	0.668	0.6960	898.0	39	60	21.8	60	0.720	1.252	0.698	0.6235	10.2
10	240	58.9	0	0.696	1.780	0.666	0.7041	323.5	40	60	26.6	60	0.720	1.370	0.696	0.6258	1.3
11	240	72	0	0.696	1.980	0.662	0.7158	43.6	41	120	13.5	60	0.720	1.793	0.689	0.6339	118.6
12	240	79.7	0	0.696	2.060	0.661	0.7205	1.3	42	120	18	60	0.720	1.945	0.687	0.6368	50.0
13	60	12	15	0.578	0.981	0.563	0.6369	127.6	43	120	27	60	0.720	2.165	0.683	0.6410	15.0
14	60	13.5	15	0.578	1.133	0.560	0.6426	66.3	44	120	36	60	0.720	2.605	0.675	0.6494	2.2
15	60	18	15	0.578	1.270	0.558	0.6477	23.3	45	240	36	60	0.720	1.709	0.691	0.6322	242.7
16	60	24	15	0.578	1.450	0.555	0.6545	3.2	46	240	48	60	0.720	2.284	0.681	0.6432	132.0
17	120	30	15	0.578	1.032	0.562	0.6388	226.1	47	240	65.5	60	0.720	2.774	0.672	0.6526	7.2
18	120	32.7	15	0.578	1.218	0.559	0.6458	124.5	48	240	72	60	0.720	2.927	0.670	0.6555	0.3
19	120	36	15	0.578	1.658	0.552	0.6623	43.2	49	60	12	100	1.028	1.370	1.000	0.6196	42.6
20	120	42	15	0.578	1.742	0.551	0.6655	3.2	50	60	13.1	100	1.028	1.489	0.998	0.6213	25.0
21	240	36	15	0.578	1.709	0.551	0.6642	665.0	51	60	14.4	100	1.028	2.013	0.987	0.6287	10.3
22	240	41.5	15	0.578	1.844	0.549	0.6693	236.0	52	60	18	100	1.028	2.165	0.984	0.6309	0.9
23	240	48	15	0.578	1.929	0.548	0.6725	21.8	53	120	12	100	1.028	2.351	0.980	0.6336	39.9
24	240	72	15	0.578	2.250	0.542	0.6845	0.2	54	120	14.4	100	1.028	2.791	0.971	0.6399	23.8
25	60	15.3	30	0.533	0.998	0.518	0.6286	259.0	55	120	18	100	1.028	3.096	0.965	0.6442	7.8
26	60	18	30	0.533	1.286	0.513	0.6368	132.5	56	120	24	100	1.028	3.299	0.961	0.6471	2.3
27	60	21	30	0.533	1.370	0.512	0.6393	31.2	57	240	14.4	100	1.028	3.062	0.966	0.6437	138.4
28	60	24	30	0.533	1.489	0.510	0.6427	2.7	58	240	18	100	1.028	3.349	0.960	0.6478	68.5
29	120	13.1	30	0.533	0.930	0.519	0.6267	373.4	59	240	24	100	1.028	3.891	0.949	0.6556	33.3
30	120	18	30	0.533	1.133	0.516	0.6325	167.4	60	240	36	100	1.028	4.703	0.933	0.6672	3.7

σ_3: Confining pressure.

q: Stress amplitude.

FC: Fines content.

e_o: Initial void ratio.

ε_v: Volumetric strain

e_1: Void ratio after consolidation.

D_{rc}: Relative density after consolidation.

N: Number of cycles for the samples to liquefy.

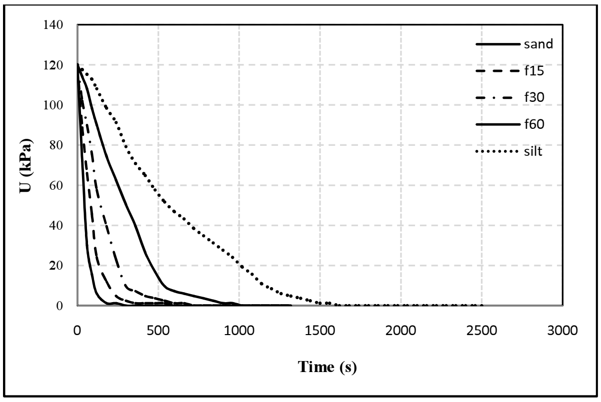

Fig. 4. Variation of Pore water pressure in time according to different samples tested

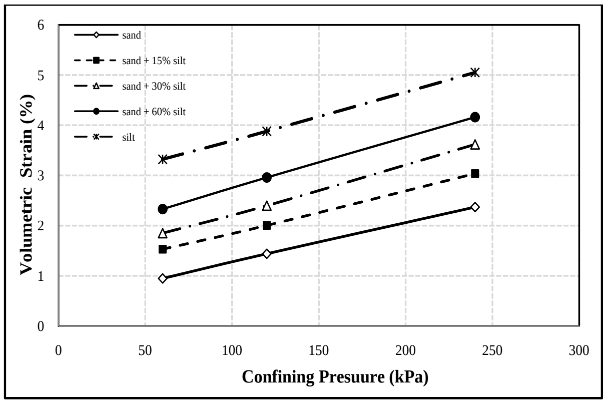

Fig. 5. Variation of volumetric strain versus confining pressure in different tested samples.

content is shown in Figure 6. As it can be seen in this Figure, clean sand and pure silt specimens have maximum and minimum CSR, respectively. Cyclic resistance ratio (CRR) is a parameter to estimate safety factor of soil against liquefaction when it is divided by CSR. This parameter is equal to the CSR at the 15th cycles of loading. Figure 7 shows the effect of silt content on liquefaction resistance ratio (CRR) in a constant relative density of 60%. This Figure is similar to the modified liquefaction resistance diagrams presented by polito and Martin [17]. Figure 7 also reveals the relation of CRR with silt content at the different confining pressures with the same relative density (60%). It can be seen in this Figures that, for low value of silt content ($SC<SC_{th}$, where SC_{th} is threshold fines content for change of behavior in silty sand) an increase in silt content causes decrease of liquefaction resistance. However with the amount of silt content more than the SC_{th} till the silt content of about 60% liquefaction resistance tend to be increased and then decreased again. Value of silt contents which are reported as limits for change of behavior (30% and 60%), are based on the results of test performed in this research. However the determination of exact values of these limits needs more tests results. Figure 7 also shows that the change of confining pressure from 60 kPa to 240 kPa causes an approximately 40% decrease in the cyclic resistance. This result shows a good agreement with the observation of research presented by Amini and Qi [16].

4.3. The Model presented in this study

In the 1970s, Seed et al. [1-3] developed an empirical model for predicting the rate of excess pore water pressure (r_u) using data from tests performed on clean sands. Equation (1) expresses their model.

$$r_u = \frac{1}{2} + \frac{1}{\pi}\arcsin\left[2(N/N_l)^{1/\alpha} - 1\right] \tag{1}$$

In their model, r_u is a function of the cycle ratio, which is the ratio of the number of applied uniform cycles of loading with constant amplitude (N) to the number of cycles with the same amplitude which is required to cause liquefaction in the soil (Nl), with an empirically parameter of α.

In 1976, Booker et al. proposed an alternative version of this model. This model is presented in Eq. (2) [29]:

$$r_u = \frac{2}{\pi}\arcsin\left(\frac{N}{N_l}\right)^{\frac{1}{2\alpha}} \tag{2}$$

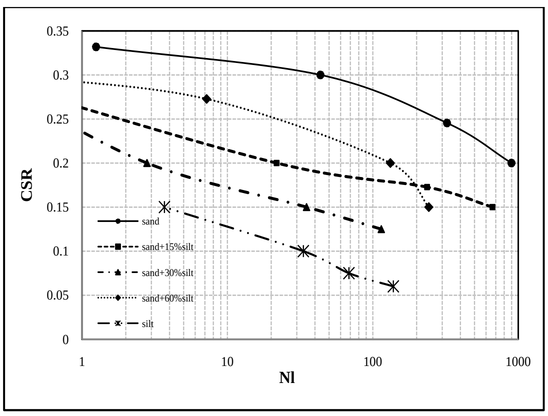

Fig. 6. Effect of fines content on the liquefaction resistance of sand-nonplastic fines mixtures for constant values of relative density (Dr= 60%)

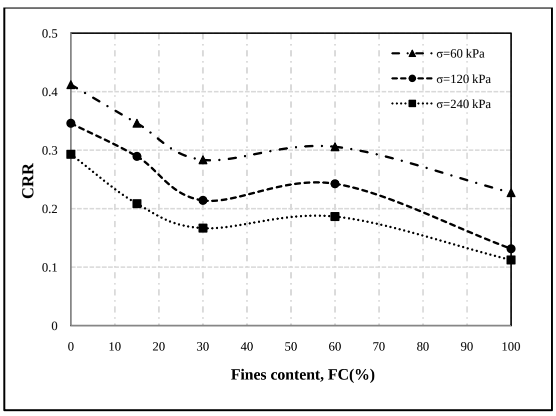

Fig. 7. Effect of fines content on the liquefaction resistance ratio (CRR) for constant relative density (Dr= 60%).

In this model, parameters of r_u, N, N_l, and α are also the same as equation (1).

Two parameters (α and N_l) of both equations of (1) and (2) can be determined using results of stress-controlled cyclic triaxial tests, as well as other types of undrained cyclic tests. For a soil sample, N_l increases with increasing relative density and it also decreases when the magnitude of stress increases, (increase of CSR). The use of N_l has its drawback as it can only be applied to liquefiable soils such as dense sand and soils with non- plastic fines which can still undergo significant pore pressure build up and deformation due to cyclic softening [8-11]. Researchers showed that both Eqs (1) and (2) can produce good results when compared with the results of cyclic triaxial and cyclic simple shear tests on clean sand. Lee and Albaisa [10], recommended an upper and lower bounds for residual pore pressure ratio for Monterey sand and sacramento sand. These bounds and other bounds presented by Seed et al [1-3] and EL Hosri et al. [11], are shown in Fig 8. In addition to the two calibration parameters, implementation of either Eq. (1) or (2) for the earthquake site response analyses requires that the earthquake motion be converted to an equivalent number of uniform cycles [31]. Such load conversion procedures are outlined in Seed et al. [1, 2], Hancock and Bommer [30], Green and Terri [31] and Polito, Green and Lee [24]. This required conversion is the greatest disadvantage in using either Eq. (1) or (2) for predicting pore pressure generation in soils subjected to earthquake-type loadings [24, 25].

Comparison of test results of current research with the bounds from model of Seed et al. [1-3], and model of EL Hosri et al. [11] are presented in Figs. 9 to 13. These Figures are for clean sand, pure silt, and sand samples with different silt content (15%, 30% and 60%). These Figures show that both models cannot predict all the test results satisfactory. However predication of these models are better for clean sand and samples with silt content of less than 30%. In other word, model of Seed et al. and EL Hosri et al. cannot be used for specimen with more than 30% silt content.

It is clear, from these Figures, that pore pressure generation characteristics of silty sands up to 30% silt content are almost

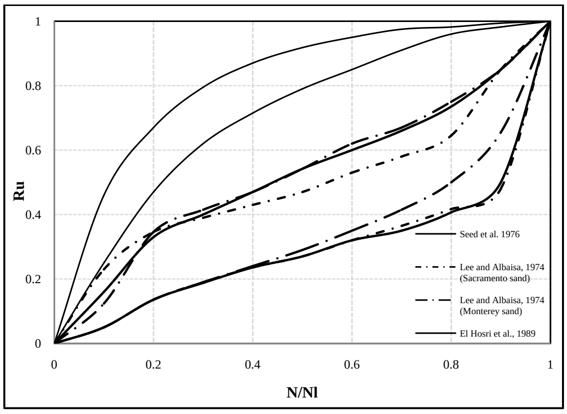

Fig. 8. Pore pressure generation data for sands and clayey silts reported in literature
Series 1: clean sand (seed et al., 1976); Series 2: Sacramento river sand (Lee and albaisa, 1974); Series 3: monterey sand (Lee and albaisa, 1974); Series 4: Clayey silt (El hosri et al., 1984)

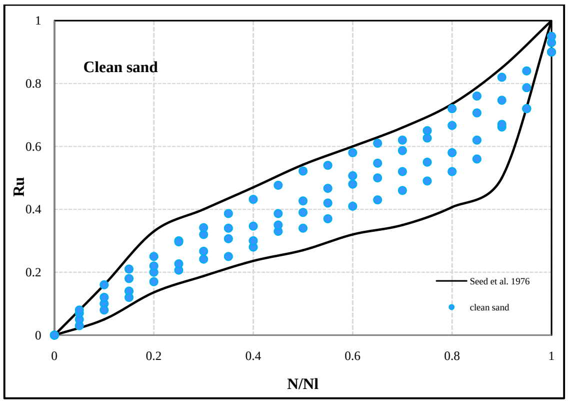

Fig. 9. Excess pore water pressure generation data for clean sand.

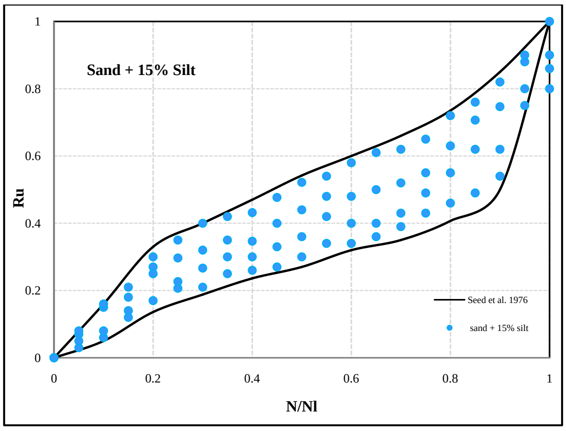

Fig. 10. Excess pore water pressure generation data for silty sands (sand + 15% fines).

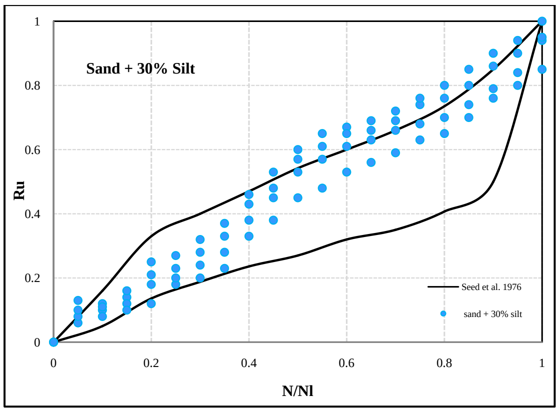

Fig. 11. Excess pore water pressure generation data for silty sands (sand + 30% fines).

similar to that of clean sand. However, for the sandy silt specimens (silt ≧ 30%), the pore pressure generation patterns deviate from that of clean sand (Figs. 11 and 12). The build up pore water pressure is much faster at the beginning of loading, and the rate slows decrease as considerable amount of pore water pressure builds up. It is interesting to note that, build up pore water pressure curves for pure silt (Fig. 13) is not much

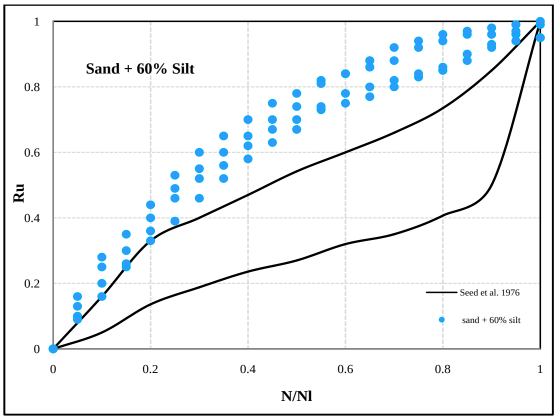

Fig. 12. Excess pore water pressure generation data for silty sands (sand + 60% fines).

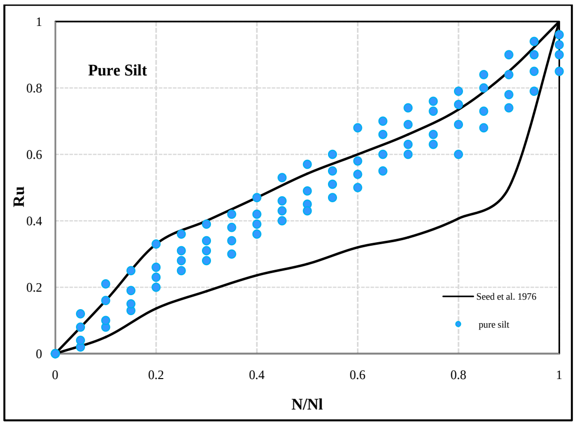

Fig. 13. Excess pore water pressure generation data for pure silt.

different from that of clean sand which is in agreement with findings of others [8-12], and follows the concept that a non-plastic silt may be considered as a sand of very fine particles [24]. However, it should be noted that, in the field, dissipation of pore water pressure for silt will be much slower compared with sand due to its low permeability. When it comes to natural silts, their behavior is much different from each other. This trend has been also observed for clayey silts as discussed by researchers [9-12]. Due to this fact, using results of experimental tests presented in this work, it was tried to make possible correction on equations (1) and (2) to predict the pore water pressure buildup for specimen in all range of silt content. This modified model will be explanted in next section.

4.4. Modification of models for predicting pore water pressure build up

The develop model of pore pressure build up data for clayey silt with plasticity index 5-15% reported in El Hosri et al. [11] is shown in the Figure 14 for comparison purposes. It can be observed that this model is an upper bound for the pore pressure generation patterns of non plastic silty soils. According to the above discussion, it can be concluded that an approximate pore water pressure generation pattern of any non-plastic soil or soil with very low plasticity can be obtained from the Figures 9 to 13. Implementing the presented tests results in as statistical software, named DATAFIT, was carried out to modify the pore water pressure build up models. The output of this software is an equation that predicts ru with maximum correlation coefficient (R^2).

The suggested equation is:

$$r_u = \left(\frac{u_g}{\sigma'}\right) = \frac{2}{\pi}\arcsin\left(\frac{N}{N_l}\right)^{1/2\alpha} + \beta\sqrt{\left(1-\left(2\frac{N}{N_l}-1\right)^2\right)} \tag{3}$$

In which

u_g is excess pore water pressure

σ' is effective confining pressure

N is number of loading cycles

N_1 is number of cycles to liquefaction

In Equating 3, α and β are two constants which are defined for different type of soils based on their silt content in Table 3. Comparison of the modified model in this research with models of others researchers for different soil types are presented in Figs. 14 and 15. As can be seen from Fig. 15, the modified model presented in this study shows good predication for the pore water pressure in silty sands.

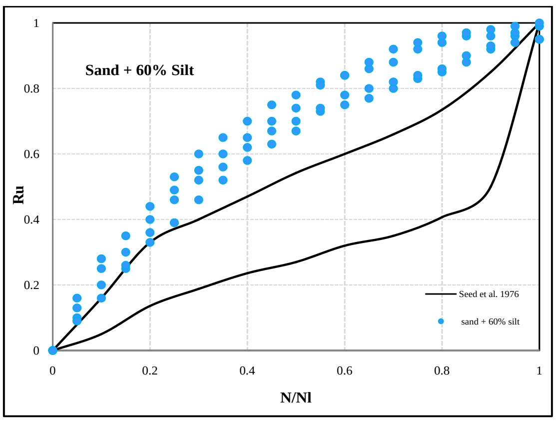

Fig. 12. Excess pore water pressure generation data for silty sands (sand + 60% fines).

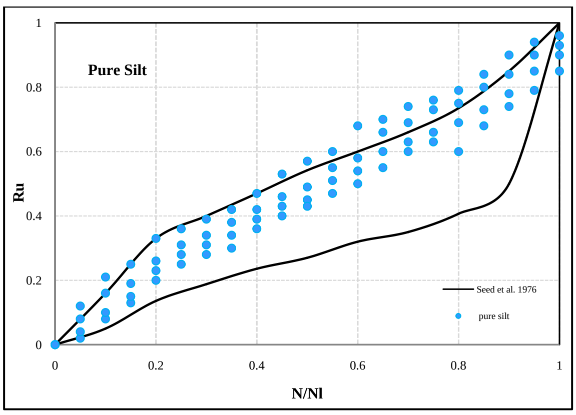

Fig. 13. Excess pore water pressure generation data for pure silt.

Table 3. Coefficients of α and β.

Soils	Seed	this study		R^2
	α	α	β	
Sand	0.6-1	0.6-0.8	0-0.25	0.94
sand+15%silt	0.6-1	0.3-0.6	0.2-0.3	0.92
sand+30%silt	0.6-1	0.2-0.4	0.25-0.3	0.9
sand+60%silt	0.6-1	0.2-0.4	0.45-0.6	0.91
Silt	0.6-1	0.6-0.8	0.1-0.2	0.92

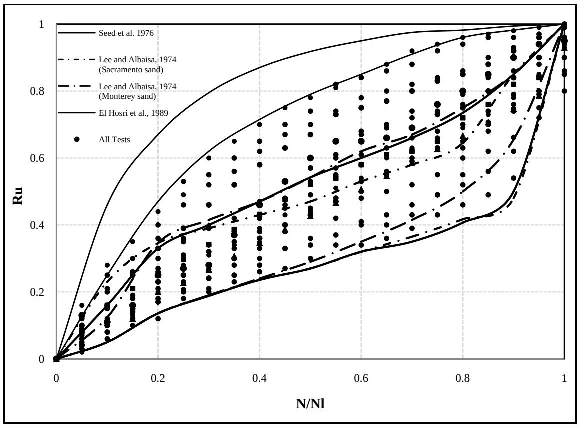

Fig. 14. Excess pore ater pressure generation data for all specimens and comparison with all model discussed in literature review.

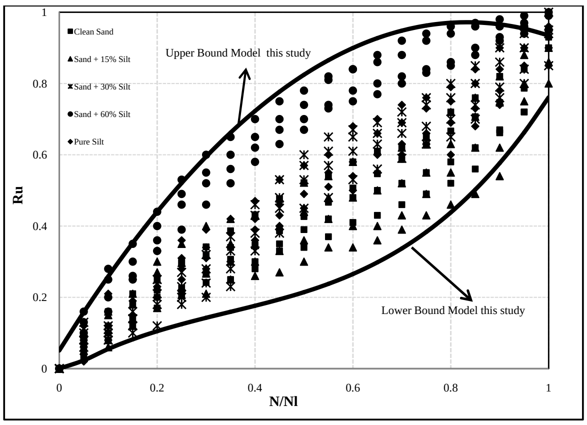

Fig. 15. Excess pore water pressure generation data for all specimens and comparison with model presented in this study.

5. Conclusion

A series of Laboratory undrained cyclic hollow torsional tests followed by drainage were conducted to study the pore pressure generation, and post-liquefaction dissipation and densification behavior of silty sands and sandy silts. The study on the selected soils draws the following conclusions:

1. The increase in the silt content (perecent passing the No.200 sieve) caused an increase in the liquefaction resistance of silty sands.

2. Pore water pressure generation characteristics (Ru vs. N/Nl) for sands and silty sands up to about 30% silt content follow the same trend found for clean sands by Seed et al. [1, 2]. The generation rate for silt and sandy silt (silt ≧ 30%) is somewhat faster than that of clean sand as it would be expected. A set of diagrams are presented, which can be used to get approximate pore pressure generation pattern of silty soils. Also, modified pore pressure generation model based on the number of cycles, suitable for sands as well as silty soils, is presented.

3. Similar to the previous studies reported by Amini and Qi [16], when the confining pressure increases, the liquefaction resistance of silty sands decreases.

4. For low values of silt content ($SC < SC_{th}$), an increase in silt content with the same relative density decrease the liquefaction resistance. This behavior is similar to the one implied in the modified liquefaction resistance diagram of polito et al [17]. A different trend is observed in the Fig. 9 for constant relative density. In this case for high values of silt content (SC > SCth), an increase in silt content (30 t0 60%) with the same relative density increases the liquefaction resistance and with more increase in silt content (for values up to 60%) the liquefaction resistance decreases.

5. The liquefaction resistance of pure sand is much more than pure silt and silty sands, while several studies have indicated that sands deposits with silt content are much more susceptible to liquefaction than clean sand.

6. Increasing the percentage of silt content, volumetric strain rate increases at constant relative density. This rate increased with increasing percentage of silt content. On the other hand, such increase in the rate for clean sand is less than that of silty sand.

7. The presence of silt indicated a decrease in excess pore water pressure generation.

8. It was seen that the increasing rate of volumetric strain with confining pressure for all the specimens were almost constant.

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