SECOND EUROPEAN CONFERENCE ON EARTHQUAKE ENGINEERING AND SEISMOLOGY, ISTANBUL AUG. 25-29, 2014

EFFECTS OF RELATIVE DENSITY AND COEFFICIENT OF CONSOLIDATION ON RE-LIQUEFACTION POTENTIAL OF SAND

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ABSTRACT

The relative density and the void ratio can be considered as the main demonstration to evaluate the liquefaction resistance of clean sands. It is expected that the soil can improve its liquefaction resistance since relative density of sand deposit significantly increases following the initial liquefaction. Contrary to this expectation, earthquake records demonstrate that densified sand may be liquefied again at smaller cycles by the similar or lesser seismic loading. In this study, 1-g laminar box tests are performed on a total of four loose and the most liquefiable silica sand in order to assess the effects of relative density and coefficient of consolidation on re-liquefaction potential of sand. Each of these small effective grain size samples is placed into the laminar box. These soil samples are subjected to the three series of shaking tests by using the one degree of freedom shaking table system. The re-liquefaction cycles of the sand, which has previously experienced liquefaction under the same seismic loadings, indicate that post-liquefaction reconsolidation of the sand deposits affects the re-liquefaction resistance.

INTRODUCTION

Several case histories demonstrate that densified sand might not improve its liquefaction resistance and can be liquefied again (re-liquefaction) by the subsequent smaller cyclic or seismic loadings (e.g. Yasuda and Tohno, 1988). This phenomenon and the factors which have the effects on the liquefaction and re-liquefaction resistance of clean sand deposits have been searched by several researchers (e.g. Mesri et al., 1990; Ohara, 1992; Oda et al., 2001; Olson et al., 2005; Ha et al., 2011). There is controversy on the reasons of reduction in the re-liquefaction resistance at smaller or similar seismic loading conditions (Oda et al., 2001; Olson et al., 2005). Ha et al. (2011) performed 1-g shaking table tests to five sands, which have differing gradation characteristics (D_{10}=0.11-0.40mm, C_u=1.53-2.57) in order to examine the role of gradational characteristics on re-liquefaction resistance. They used a laminar box with dimensions of 192 cm x 44 cm (in plan) x 60cm high. A 40-cm thick saturated sand layer was placed in the laminar box by water pluviation method. They used the input acceleration consisted of 20 sinusoidal cycles of 0.15 g amplitude (4 Hz wave with a 5 sec duration). As a result of the shaking table tests, they concluded that liquefaction and re-liquefaction resistances do not correlate well with a single index property such as relative density. However, compressibility as well as the hydraulic conductivity which is associated with the grain size of the sand shows the reasonable correlation with re-liquefaction resistance. On the other hand, they did not regard the effects of the different cyclic loadings on the liquefaction and re-liquefaction resistance. The related literature points out that there are still uncertainties in the reasons why the re-liquefaction resistance of

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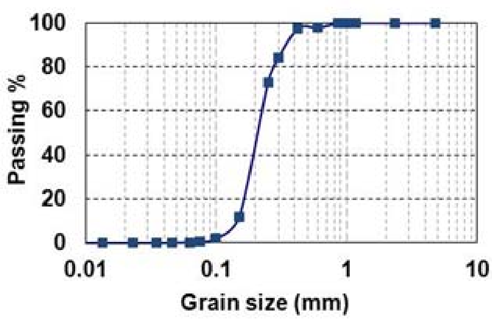
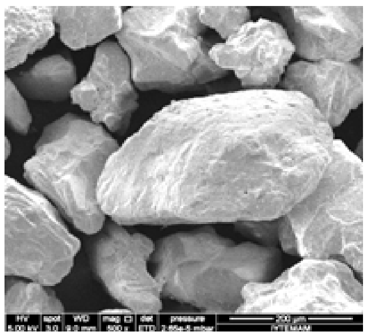
the sand decreases with an increase in relative density. Hence, there is a need to understand the role of relative density, hydraulic conductivity, compressibility of sands and different cyclic demands on the liquefaction and re-liquefaction resistance.

In this study, a total of four loose and the most liquefiable silica sand deposits (D_{10}=0.12mm, C_u=1.17) were prepared inside the 1-g laminar box by using the hydraulic filling method. Each of these samples was subjected to three series of shaking tests by using the one-degree of freedom shaking table system. Before each shaking tests, the relative density and compressibility of the sand was determined throughout the depth from the piezocone penetration tests. The flexible aluminum laminar box with dimensions of 180cm x 60cm (in plan) x 150cm high was used for the tests. A 140-cm thick saturated sand layer was placed in the laminar box by hydraulic filling method. The box consisted of 24 rings, which can move with a low friction on each other and simulate the free-field conditions effectively during different cyclic loadings. The system also involves instrumentation and associated testing hardware to collect detailed data on the progression of liquefaction phenomena during each shaking. The quality and reliability of the measured data from the laminar box and shaking table system are presented by Ecemis (2013). The measured excess pore water pressure at various depths inside the soil model was used to define the number of cycles required to cause initial liquefaction (N_L) and re-liquefaction (N_{R-L}). The shaking table tests were performed on the initially liquefied loose sands in order to find out the role of relative density and coefficient of consolidation on the liquefaction and re-liquefaction resistance. At each shaking table tests, we explored the role of relative density, post liquefaction reconsolidation and cyclic loadings on the liquefaction and re-liquefaction resistance of sand.

MATERIAL AND TESTING PROGRAM

The fine silica sand is used in the experimental work. Its specific gravity (ASTM D854), minimum (ASTM D4254) and maximum void ratios (ASTM D4253) are found to be 2.61, 0.80 and 0.60, respectively. From the sieve analysis results (Fig.1a), it is seen that effective particle size (D_{10}) is 0.12mm and mean grain size (D_{50}) is 0.21mm. The coefficient of uniformity (C_u=D_{60}/D_{10}) is found as 1.17. It is defined as poorly graded sand (SP) according to the unified soil classification system. The sand particles used in the experiments are naturally formed sub-angular sand grains which are supplied locally. Fig.1b shows the particle shape of the sand grains (SEM picture).

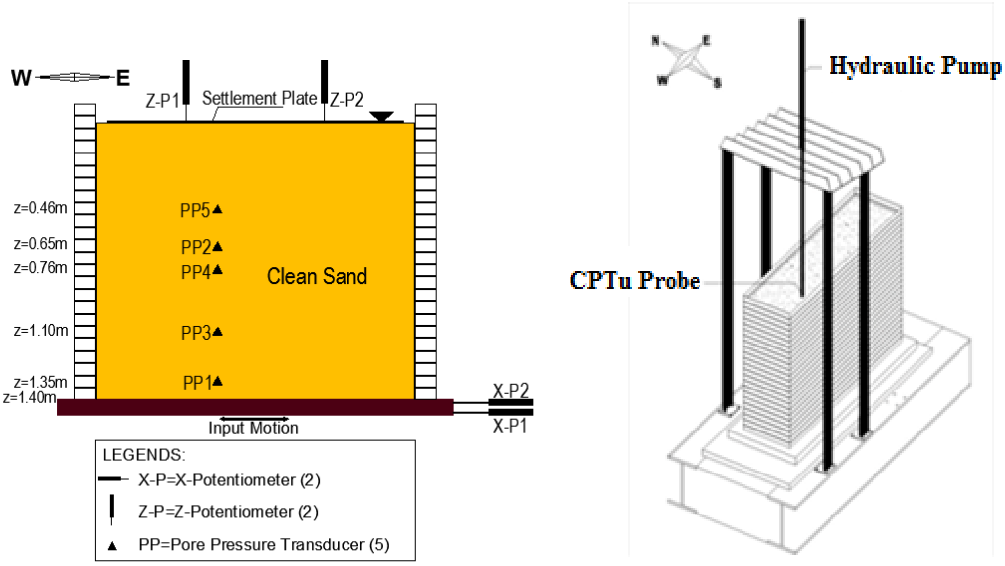




(a) (b)

Figure 1. (a) The grain size distribution curve and (b) scanning electron micrograph of the sand used in the shake table tests

In this study 1-g laminar box which has dimensions of 180cm×60cm in plan and 150cm high was used. The laminar box consists of 24 identical shaped aluminum rings (hereafter named as laminates) which move relative to each other. Aluminum was chosen to reduce the weight of the laminates and their inertial effects on the soil movements. In order to significantly reduce the friction between the layers and to make the distribution of friction uniform, rings are separated and supported by eight rollers (diameter of 47mm) mounted between each laminate. A 1mm thick EPDM rubber membrane was placed inside the box to provide air tightness and not to let the soil to come in contact with the laminate walls directly. The stiffness and the thickness of the membrane were chosen in order not to affect the movement of the laminar box. The instrumentation on the laminar box includes two potentiometers (X-P1 and X-P2) on the shake table for lateral movement measurements of the shake table. The instrumentation within the soil specimen includes five piezometers for measuring the pore water pressure at different depths. In addition, two potentiometers (Z-P1 and Z-P2) were used on the sand surface to measure the surface settlements during and after each shaking table tests. Fig.2a shows the layout of the different type of instruments on the shake table and within the soil.



(a) (b)

Figure 2. (a) The layout plan of the instrumentation and (b) cone penetration system

Each of the sand models was prepared in the $0.42m^3$ sample preparation basin by mixing the soil samples with water before each test. These samples were then placed in the laminar box by using the hydraulic fill deposition technique (Whitman, 1970). This sand placement method allows sand particles to sink slowly through water under the action of gravity. This is like a natural alluvial deposition of sands in rivers or lakes. The deposited sand beds are slightly anisotropic in the particle arrangement, due to placement of the sand under the action of gravity.

The deposition process is involved the following steps: (1) the membrane is placed inside the box and filled with water to a depth of about 30cm; (2) soil and water mixture from sample preparation basin is pumped by a slurry pump to uniformly place sand throughout the laminar box; (3) it is waited for the soil grains settle inside the laminar box; (4) after some time a water pump is used to deliver the excess water from the laminar box back to the sample preparation basin. This closed-loop pumping system is adjusted to maintain average 30cm height of water above the sand surface inside the laminar box to achieve the loose deposition. Almost the top 30cm of the soil inside the box is found to be slightly denser than the soil at the bottom, due to the limitation of maintaining the 30cm water near the top of the box. This reduction in the water height on the soil surface increased the relative density of the soils due to change in the settling heights of the sand particles. This cycle is maintained till the height of the saturated loose clean sand sample in the laminar box reaches to 1.4m.

Total of four sand deposits are prepared inside the laminar box and subjected to individual sinusoidal motions for 12 seconds with different peak ground accelerations (PGA) under the undrained conditions. Three subsequent shakings are applied with maximum input accelerations of 0.05g, 0.10g, 0.16g and 0.21g at each sand deposit in the scope of Test 1, Test 2, Test 3 and Test 4, respectively. The frequency of each shake is 2Hz. The multi-directional shaking and modes of shear of the strong shaking in the field is different from the laboratory. Therefore, the peak ground accelerations used for the sine wave based laboratory experiments should be scaled up by a factor of 1/0.65 in order to achieve real behavior of soil during the earthquake (Seed and Idriss 1971). The levels of base shaking used for this study are summarized in Table.1 with the equivalent PGA values as recommended by Seed and Idriss (1971). The cyclic stress ratio (CSR) is determined from the equation proposed by Seed and Idriss (1971) as Eq.1.

$$CSR=0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_{vo}}{\sigma_{vo}'}\right)r_d \quad (1)$$

where σ_{vo}'=effective initial vertical stress, σ_{vo} = total initial vertical stress, g = acceleration of gravity, and r_d = stress reduction factor varying from a value of 1 at the soil surface to a value of 0.9 at a depth of about 9m. The CSR values for each shaking tests are also listed in Table.1.

The number of cycles required to cause liquefaction (N_L) and re-liquefaction ($N_{R\text{-}L}$) are specified by multiplying the time when the liquefaction started to occur with 2 Hz frequency of the seismic motion.

Table 1. Levels of shaking and cyclic stress ratio

Shake Table Test No.	Shake Number	PGA	$(PGA)_{eq}$	CSR
-	-	*g*	*g*	-
Test 1	1st Shake	0.05	0.08	0.11
	2nd Shake			
	3rd Shake			
Test 2	1st Shake	0.10	0.15	0.21
	2nd Shake			
	3rd Shake			
Test 3	1st Shake	0.16	0.25	0.35
	2nd Shake			
	3rd Shake			
Test 4	1st Shake	0.21	0.32	0.46
	2nd Shake			
	3rd Shake			

The piezocone penetration tests (CPTu) are performed to evaluate the relative density of the sand prior to each shaking tests. As displayed in Fig.2b, the 60^o tapered, 10cm^2 tip area cone (diameter of 3.57cm) penetrated into the soil at a constant penetration rate of 2cm/s (ASTM D3441) by using the hydraulic pump. The CPTu tests provide measurements of cone penetration resistance (q_c), friction resistance (f_s) and pore water pressure (u_2) values along the depth.

The relative density along the soil depth is indirectly determined from the measured cone penetration resistance by using the empirical relationship given by Lunne, Robertson and Powell (1997) as Eq.2.

$$D_r=-98 + 66 \times \log_{10}\left(\frac{q_c}{\sqrt{\sigma_{vo}'}}\right) \quad (2)$$

where D_r = relative density in percentage; σ_{vo}' = effective initial vertical stress in the same units as that of measured cone penetration resistance, q_c. The relative densities of clean sand samples prepared by hydraulic filling method were found between 11% and 35% throughout the depth. These values are consistent with the relative density intervals measured immediately after deposition in clean sand hydraulic fills (Whitman, 1970; Poulos and Hed, 1973; Mitchell et al., 1999; Thevanayagam et al., 2009).

Several constant head permeability tests (ASTM E2396) were performed in the laboratory in order to measure the hydraulic conductivity (k) of the sand at different relative densities. Fig.3 shows the variation of hydraulic conductivity against relative density of the fine silica sand that has effective grain size of 0.12mm. The hydraulic conductivity decreases with an increase in relative density due to reduction in pore size characteristics as well as the shape of the voids. Hence, hydraulic conductivity and relative density are inversely proportional to each other.

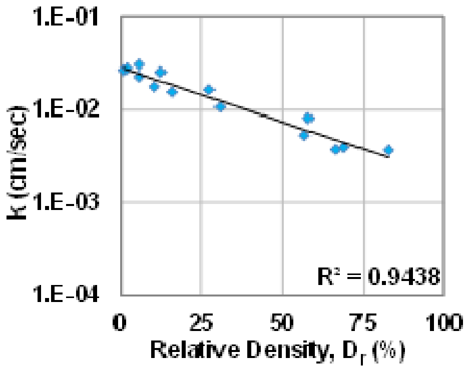


Figure 3. The relationship between the hydraulic conductivity (k) and relative density (D_r)

In the literature, many correlations are available in order to estimate the compressibility of the soils from the measured cone penetration resistance (Sanglerat 1972; Kulhawy and Mayne 1990; Robertson 2009). In this study, the compressibility of the clean sand is estimated from the CPTu tests based on the correlation proposed by Robertson (2009) as Eq.3.

$$m_v = \frac{1}{\alpha_M \left(q_t - \sigma_{vo}\right)} \tag{3}$$

where σ_{vo} = total initial vertical stress, q_t = corrected total cone resistance, and α_M is a parameter related to the soil behavior type index which is modified by Robertson and Wride (1998).

After determination of the hydraulic conductivity and compressibility corresponding to intended relative density, the coefficient of consolidation (c_v) is determined from Eq.4.

$$c_v = \frac{k}{m_v \gamma_w} \tag{4}$$

where γ_w is the unit weight of water.

EFFECT OF RELATIVE DENSITY ON LIQUEFACTION AND RE-LIQUEFACTION RESISTANCE

The above mentioned shaking table test results enable us to find the influence of relative density on the liquefaction and re-liquefaction potential of the sand. Fig.4a illustrates the relation between the initial relative densities and CSR for each shake at five different locations in the sand profile. At each cyclic loading, we observed a pronounced increase in relative density from initial liquefaction to re-liquefaction. The relative densities increase for each shake following the first shaking test. At first shake, all relative densities were less than 50% whereas after third shake all relative densities found greater than 50%. This significant increase in relative density is due to the dissipation of excess pore water pressure and the consolidation settlement after and during liquefaction.

Fig.4b demonstrates the relation between the number of cycles required to cause initial, second and third liquefaction (N_L) and CSR values at different depth. As shown in the figures only at CSR of 0.11, large enough excess pore pressure is not developed to liquefy the sand during the third shaking at a depth of 0.46m. For four different cyclic/seismic intensities, the sand re-liquefied throughout its almost entire profile during all subsequent shakings, despite the increase in relative density resulting from shaking and reconsolidation. Also, both increases and decreases in the re-liquefaction resistance of the sand deposits are recorded at each seismic demand.

By evaluating two graphs for each depth, it is concluded that the amount of increase in relative density cannot be possibly the main reason for the liquefaction resistance and re-liquefaction resistance of sand deposits. There should be other factors, which may play a crucial role on the change in the re-liquefaction resistance of the sand deposits, rather than the sole relative density.

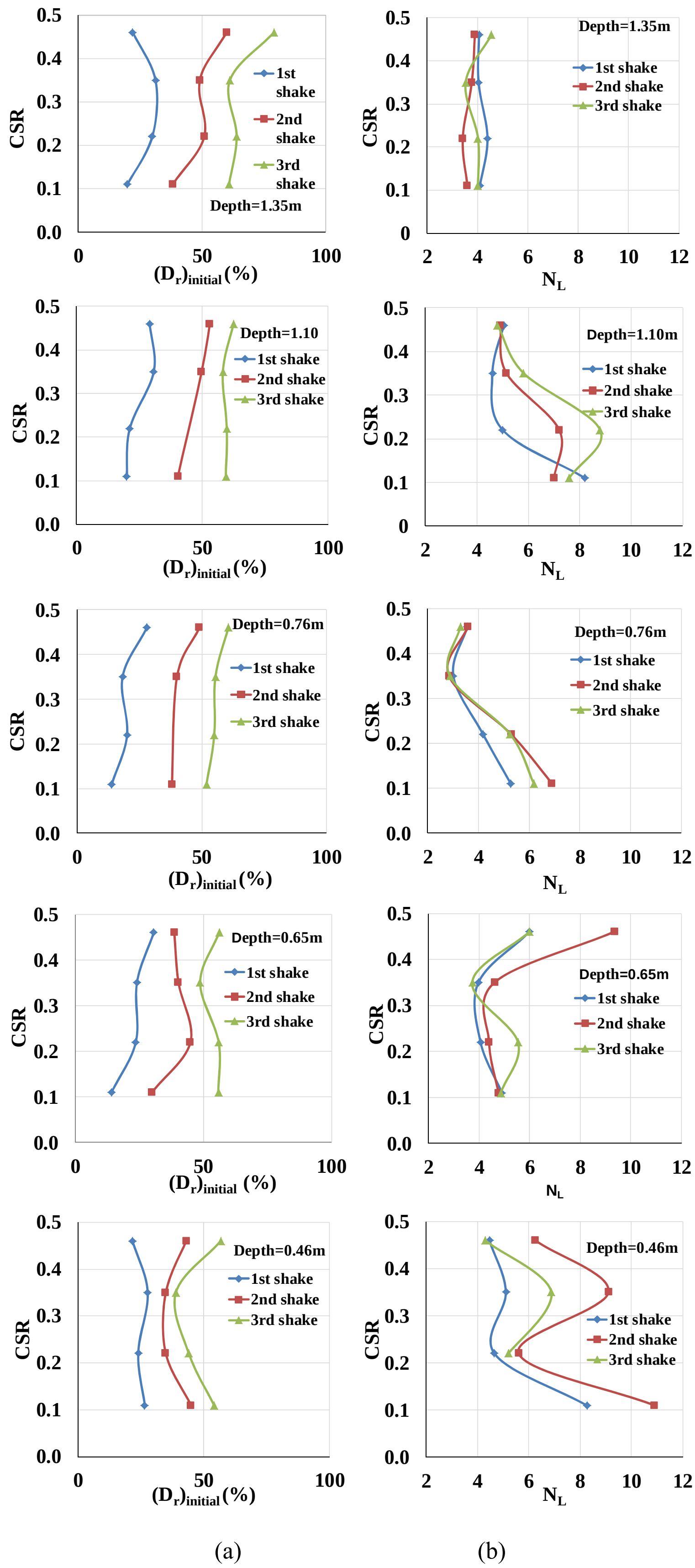


Figure 4. Variation of (a) liquefaction resistance and re-liquefaction resistance, and (b) relative density at five different depths

EFFECT OF COEFFICIENT OF CONSOLIDATION ON LIQUEFACTION AND RE-LIQUEFACTION RESISTANCE

Given these observations, we can determine the influence of the consolidation characteristics to liquefaction and re-liquefaction resistance at different cyclic loading conditions. As shown in Fig.5a, the ratio of $(N_L)_2/(N_L)_1$ are plotted against the coefficient of consolidation for four different cyclic loadings. The coefficient of consolidation value given in Fig.5a is determined after the first liquefaction test. As shown in the Fi.5a, the ratio of $(N_L)_2/(N_L)_1$ decreased as post-liquefaction consolidation increased. From CSR 0.11 to 0.46, the second liquefaction resistance determined smaller than the first liquefaction resistances when the post-liquefaction consolidation is greater than about $5x10^4$ mm^2/sec to 10^5 mm^2/sec, respectively. The decrease in the second liquefaction resistance of the sand column depends on the following factors: (1) destruction of the loose and aged soil structure during the initial liquefaction results in a significant increase in relative density, (2) significant densification results in a significant increase in post-liquefaction consolidation and decrease in secondary compression following the first shaking, (3) reduction in secondary compression do not allow sand grains adjust into a more stable configuration under constant vertical effective stress (Terzaghi et al., 1996, Mesri et al., 1990), as a result sand column becomes freshly deposited, and re-liquefy more readily. Reduction in the second liquefaction resistance is consistent with the prior observations of Olson et al. (2005) and Ha et al. (2011).

Fig.5b shows the ratio of $(N_L)_3/(N_L)_2$ against the coefficient of consolidation for four different cyclic loadings. The coefficient of consolidation value given in Figs.5b is determined following the second liquefaction test. As shown in the figure, the ratio of $(N_L)_3/(N_L)_2$ increased as post-liquefaction consolidation of the sand increased. From CSR 0.11 to 0.46, the third liquefaction resistance determined greater than the second liquefaction resistances for the post-liquefaction consolidation more than about $6x10^4$ mm^2/sec to $2x10^5 mm^2$/sec, respectively. The increase in the third liquefaction resistance of the sand column depends on the following factors: (1) destruction of the freshly deposited sand column during the second shaking and an additional increase in relative density, (2) increase in relative density results in an increase in post-liquefaction consolidation and decrease in secondary compression following the second shaking, (3) secondary compression do not influence the dense soil and, as a result, third liquefaction resistance increase. Increase in the third liquefaction resistance is consistent with the prior observations of Mesri et al. (1990), Olson et al. (2005), and Ha et al. (2011).

Hence, these results show that, the liquefaction and re-liquefaction resistances are not only depending on the relative density but also to the particle interlocking and coefficient of consolidation, which depends on the hydraulic conductivity and the compressibility of the sand deposits. This also shows that, different cyclic loadings do not significantly affect the limit coefficient of consolidation value, where the re-liquefaction resistance is smaller than the liquefaction resistance. We also compared these outcomes with the results of Ha et al. (2011) who performed a series of shake table tests at the same cyclic loading (peak amplitude of 0.15g). They conducted cyclic tests using the five different gradation characteristics sands. They found that, at each sand deposit, the re-liquefaction resistance increases significantly when the coefficient of consolidation exceeds about 10^4 mm^2/sec. The difference in the determined limit coefficient of consolidation value in their study might depend on the difference at CSR and the effective grain size characteristics (hydraulic conductivity) of the tested sand deposits.

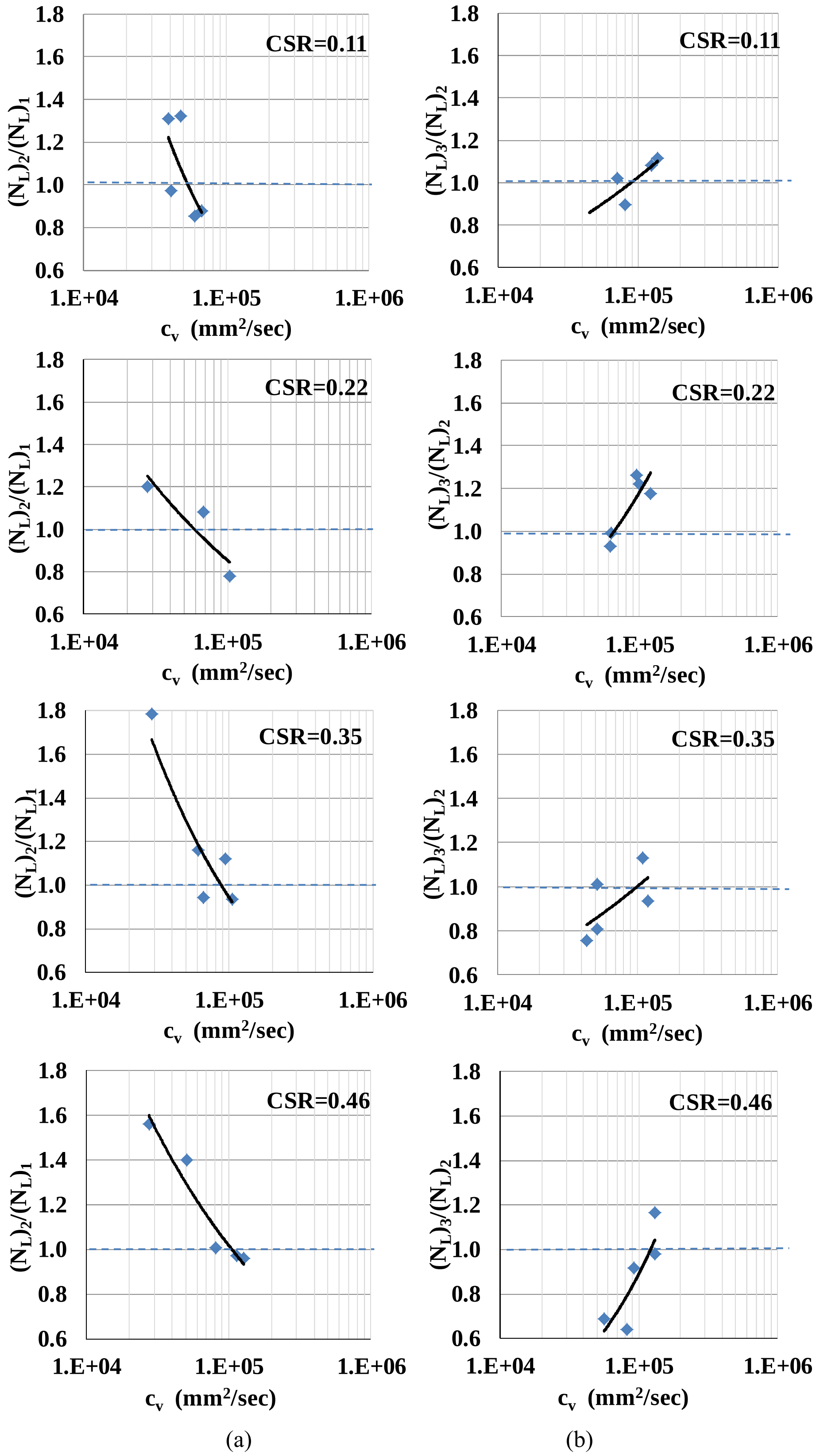


Figure 5. The relation between (a) (NL)2/(NL)1 and cv (b) (NL)3/(NL)2 and cv for CSR 0.11, 0.22, 0.35 and 0.46.

CONCLUSIONS

The saturated sand deposits, which had been liquefied previously, could be liquefied again at smaller cycles, even though the relative density of the soil increases significantly. In order to understand the effect of relative density and consolidation characteristics on the liquefaction and re-liquefaction potential of fine silica sand (effective grain size of 0.12) we prepared four samples inside the laminar box by hydraulic filling method. Total of four shaking table tests performed at different shaking intensities. At each seismic demand, the input excitation is imposed three times. Based on these test results, the following features can be summarized as follows:

1. Despite significant increase in relative density re-liquefaction resistance of sand deposits may decrease. Therefore, it is concluded that the change in re-liquefaction resistance is not only related with relative density.

2. It is observed that for variable CSR values the second liquefaction resistance is smaller than the first liquefaction resistance when the post-liquefaction consolidation exceeds the value of $5x10^4$ mm^2/sec to 10^5 mm^2/sec. This is due to the reduction in secondary compression.

3. It is concluded that for variable CSR values the third liquefaction resistance is greater than the second liquefaction resistance when the post-liquefaction consolidation exceeds the value of $6x10^4$ mm^2/sec to $2x10^5$ mm^2/sec. The relative density of soil increases due to post-liquefaction consolidation after the second liquefaction. Decrease in secondary compression can not affect dense soil. As a result, third liquefaction resistance increase.

4. It is observed that particle interlocking and the coefficient of consolidation might affect the liquefaction and re-liquefaction resistances. However, the limit coefficient of consolidation value (when the liquefaction resistance is greater than the re-liquefaction resistance) is not substantially influenced by different cyclic loading.

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