Re-liquefaction of Sand in Shaking Table

Dikshit Babu Nepal[1], Jianliang Deng[1, 2], Jinjian Chen[1], Tshegofatso Maakoe[1]

1 State Key Laboratory of Ocean Engineering, Department of Civil Engineering, Shanghai Jiao Tong University, Shanghai 200240, China
2 Corresponding author: dengjianliang@sjtu.edu.cn; Tel: +8613916341178

Abstract. Subsequent aftershocks can induce reliquefaction of sand. In the shaking table experiment, the settlement, the excess pore water pressure and the acceleration response of the sand in the process of reliquefaction were measured. It was found that: (1) the liquefaction resistance of sand was the lowest in the second liquefaction, not in the first liquefaction; (2) Liquefaction resistance is depth-dependent, and the soil layer near the surface of soil is most likely to be liquefied; (3) In the model test, the attenuation of seismic shear wave acceleration is not only related to distance but also to the vibration frequency and to the liquefied layer; (4) The amplitude of the acceleration response increases with the number of the shaking. This result indicates that: (1) the phenomenon of re-liquefaction of sand induced by earthquake is not same as that of the first liquefaction. Therefore, the conventional strategy for liquefaction resistance needs to be improved before it can be implemented for analysis of reliquefaction. (2) The attenuation of seismic shear waves in liquefied soil layer has some properties different from the normal soil layer; thus the traditional attenuation calculation method needs improvement in the evaluation in the analysis of site liquefaction.
Keywords: Re-liquefaction, Shake table, Excess pore water pressure (EPWP), Sand resistance

1. Introduction

Reliquefaction of sand has been reported extensively in the recent earthquake research and reconnaissance [4, 7] while prehistoric liquefaction [10,14,17] illustrated that the sand deposits once liquefied might still get liquefied again in the future by a subsequent earthquake similar or even smaller than the previous one which is very alarming issue. Finn et al. [2] conducted the triaxial shear test and observed that the reliquefaction resistance of sand decreases during the subsequent earthquake. Following this pioneering work, Ishihara et al. [5] and Suzuki et al. [11] also investigated the reliquefaction resistance of the sand using the triaxial test and found that the sand reliquefies more easily despite the increase in density in the second earthquake. Additionally, Oda et al. [9] explained that the destruction of aged soil structures by large shear straining makes the liquefied sand to act like a fresh deposit following a post-liquefaction consolidation which may reduce the post shaking liquefaction resistance [12]. Olson et al. [8] further reinforced that the re-liquefaction resistance of the sand may not increase despite the increase in density due to post-liquefaction consolidation. Similarly, by conducting triaxial test, Wang et al. [15] revealed that the re-liquefaction resistance of silt also decreases.

On the other hand, some studies show that the reliquefaction resistance of sand can increase in the subsequent earthquake. Based on the laboratory test of dynamic shear modulus after primary consolidation, and field measurement of penetration resistance after ground densification, Mesri et al. [6] explained the resistance of the sand in re-liquefaction may increases only after the significant increase in relative density of the sand. In contrary, Ye et al. [19] argued and stated the reliquefaction resistance of the sand remains constant, but the dissipation time of excess pore water pressure (EPWP) decreases with increasing density. Yamada et al. [16] concluded that the reliquefaction resistance of the sand may increase or decrease depending upon the level of anisotropy developed before the shaking. After performing shaking table test in sand, Ha et al. [3] and Ye et al. [18] concluded that the reliquefaction resistance of sand decreases during the second shaking and starts increasing after the second shaking. Despite of intensive research, the results observed in the sand reliquefaction test is inconsistent. In this study, a series of shaking table tests were conducted successively in a sandy model to investigate the liquefaction resistance of sand by measuring the settlement in the first and the subsequent shakings. Furthermore, the response of EPWP and acceleration in multiple layers were also studied.

2. Materials and Methods

Four pore water transducers and four accelerometers were fixed in the model box designed and fabricated for the shake table test, as shown in **Figure 1**. Three accelerometers (A_2, A_3, and A_4) were buried in the sand, and one accelerometer (A_1) was set on the shaking table near the base of the model to measure the input acceleration. The linear variable differential transformer (LVDT) of 50mm capacity was used to measure the ground subsidence, which was extended with a plate (3cm×3cm×0.2cm thick) to prevent the tip from plunging in the sand. In this experiment, yellow sand within the liquefiable range as reported by Tsuchida [13] was used. The grain size distribution and the index properties of the sand used for the experiment is as shown in **Figure 2** and **Table 1** respectively. The model box was filled with water, and oven-dried sand was allowed to fall from a narrow tip funnel so that the sand lies above the prior sand particles by gravity. The excess water at the surface was removed by siphoning. Finally, out of many experimental trials, analysis was performed for the sample with the least relative density (22.9%). The water content, void ratio, and degree of saturation of the thus prepared sample were obtained to be 28.4%, 0.754, and 98.1% respectively.

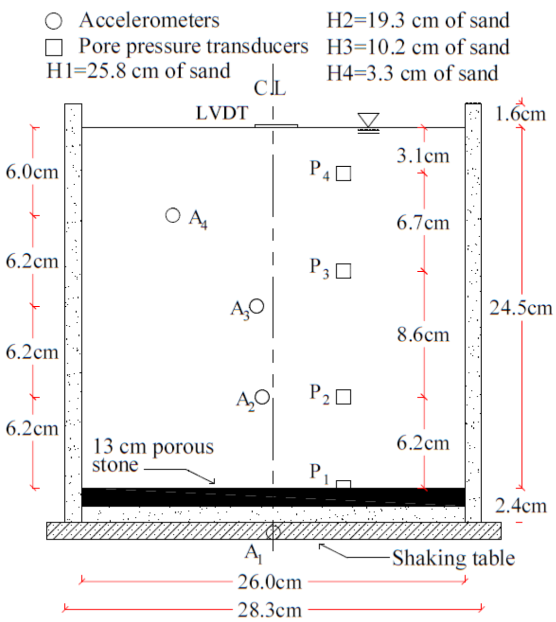

Figure 1. Schematic model and layout.

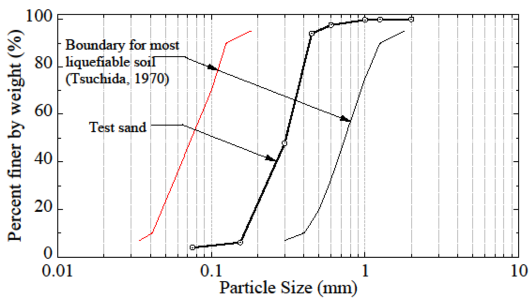

Figure 2. Grain size distribution of sand

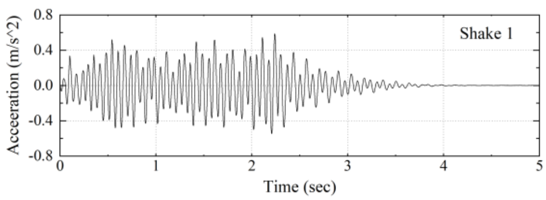

Figure 3. Acceleration time history during for first shaking event.

Table 1. Index and gradation property of experimental sand

Grain size (mm)				Coefficient of uniformity (C_u)	Specific gravity (G_s)	Dry unit weight (kN/m^3)		Void ratio	
D_{10}	D_{30}	D_{50}	D_{60}			Min.	Max.	Min.	Max.
0.17	0.24	0.31	0.34	2.10	2.60	14.30	17.00	0.53	0.82

The sand model was subjected to input acceleration of similar PGA till the 6th shaking as shown in **Figure 3**. The subsequent shakings were applied only after the dynamic pore water pressure were completely dissipated. The ground settlement was measured before and after each excitation. Based on the settlement data, the relative density (D_r) and void ratio (e) after each test were evaluated.

3. Result and Discussion

3.1. PWP response and reliquefaction mechanism

Liquefaction occurs when EPWP (Δu_x) reaches the initial vertical effective stress (σ'_{vo}), i.e., excess pore water pressure ratio ($r_u = \Delta u_x/\sigma'_{vo}$) is unity. However, the directly measured EPWP has significant fluctuation due to the irregular input acceleration. Therefore, all the Δu_x histories have been smoothened by using 1-D median filtering and smooth function in MATLAB to show the trend of Δu_x and were compared to r_u =1 condition for respective sand layer. Here, the sand layer H4 has been omitted due to its inconsiderable height which imparts insignificant response. In some cases (Figure 4), r_u corresponding to the liquefied height is obtained slightly higher than 1. Ecemis et al. [1] explained this phenomenon to be either a result of the settlement of the piezometers or the rise in the water table elevation.

The time required to reach the condition r_u =1 were significantly varying for the different shaking events. For liquefied sand layers (r_u =1), N_L and N_R are event-dependent parameters whose value depends upon the product of dominant frequency and time; where, N_L and N_R are the numbers of cycles required to trigger liquefaction during the *1st* and following shakings, respectively. However, for some layers where sand couldn't reliquefy (r_u <1), the corresponding maximum r_u for non-liquefied layers are listed in Table 2, and **Figure 5** graphically represents its data.

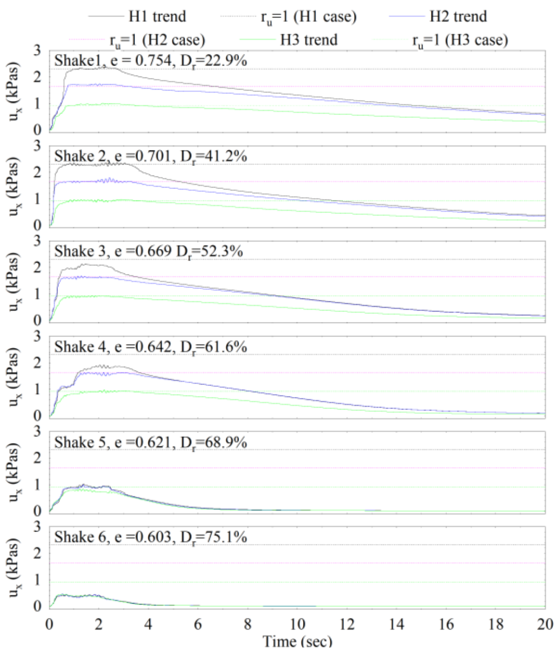

Figure 4. Excess PWP time history in each of the shaking events.

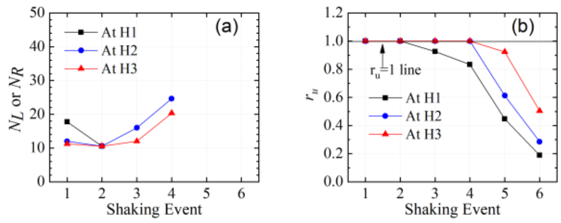

Figure 5. Graphical representation of (N_L or N_R) and r_u with successive shaking events.

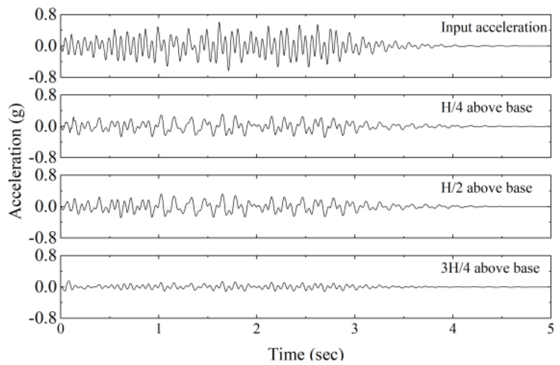

Figure 6. Acceleration time history measured at different height during the 2nd shaking event

Table 2. Number of cycles required to trigger liquefaction/reliquefaction during each of the shaking events.

Height	Event 1	Event 2	Event 3	Event 4	Event 5	Event 6
H1	N_L=18	N_R=10	r_u= 0.93	r_u= 0.83	r_u= 0.41	r_u= 0.19
H2	N_L=12	N_R=10	N_R=16	N_R=25	r_u= 0.61	r_u= 0.28
H3	N_L=11	N_R=10	N_R=12	N_R=20	r_u= 0.92	r_u= 0.50

Figure 5 shows that the values of N_L in H1 ~ H3 for the first shaking are significantly larger than the corresponding values of N_R for the second shaking event. Therefore, the resistance to reliquefaction in the second shaking event decreases relative to the first shaking event despite of the increase in the density. The destruction of initial anisotropic sand fabrics during the first shaking could possibly be the reason for the rate of increase in pore water pressure during the second shaking. However, N_R exhibits the upward trend during the subsequent shaking events, meanwhile, reliquefaction resistance also starts to increase with density which makes the second event most vulnerable to reliquefaction phenomenon. The value of N_R in the second shaking event has been reported by Ha et al. [3] and Ye et al. [18] to be minimum which validates the results observed in this study. As indicated in Figure 4, all the sand layers in H1, H2, and H3 heights were liquefied in both the *1st* and the *2nd* shaking events, but reliquefaction was only observed for H2 and H3 heights in *3rd* and *4th* shaking events indicating that H2 and H3 are more likely to re-liquefy than H1. For *5th* shaking event, none of the sand height re-liquefied, however, the value of r_u for H3 height was 0.92 (implies closely re-liquefied) which means that the shallow layer (e.g. H3 layer) is most susceptible to re-liquefaction. Therefore, reliquefaction behaviors are depth-dependent. Similarly, none of the sand layers were re-liquefied even in the *6th* shaking event ($r_u \leq 0.5$). Finally, due to the significant reduction in the value of r_u, the need for further investigation was irrelevant. According to Ha et al. [3], the small increases in effective confining stress and relative density with depth contributes to this depth-dependent reliquefaction behavior. Since the effective confining stress and the relative density could not be measured precisely, more studies are needed to further investigate the underlying mechanism of depth-depending reliquefaction.

3.2. Acceleration response and mechanism

The amplitude of acceleration response attenuates with the propagation distance from the point of excitation, while the rate of attenuation is related to the frequency. The seismic waves were still under effect even after the corresponding input excitations were completely ceased. Thus, the input acceleration time history for 3 seconds for all the shakings were subjected to fast Fourier transform (FFT) to obtain the dominant frequency. A typical acceleration time history and its FFT analysis, are shown in **Figure 6** and **Figure 8**, respectively. That FFT normalization was based on mean square amplitude with a sampling interval of 0.001 seconds. Thus obtained first dominant frequency was nearly 16 Hz for all the input shakings, whereas, the second dominating frequency was 7 Hz. Besides, other sub and super harmonics were also observed during those periods.

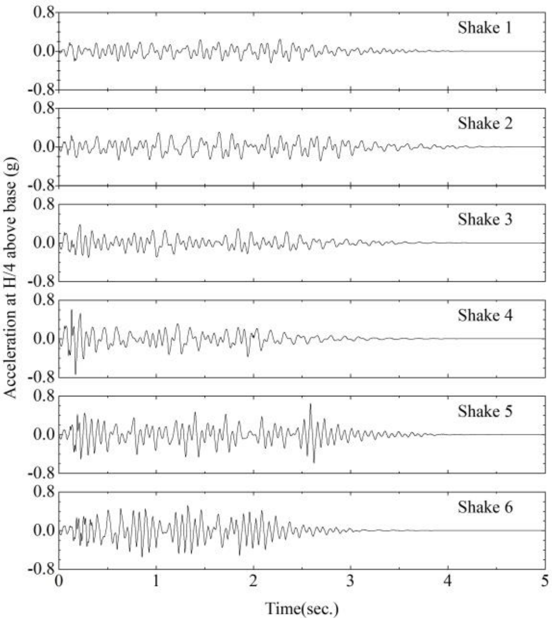

Figure 7. Acceleration response measured by accelerometer A_2 placed H/4 above the base.

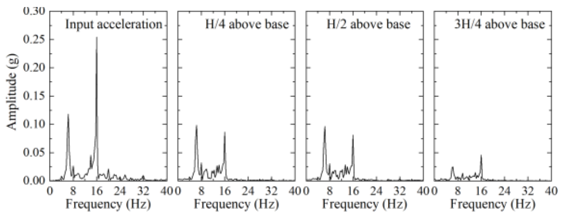

Figure 8. FFT for acceleration measured at different height during the 2nd shaking event.

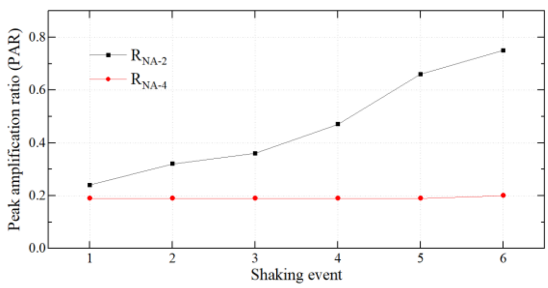

Figure 9. R_{NA} for the sand layer at H/4 and 3H/4 above the base.

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It should be noted that the attenuation rate with lower frequency can be much smaller than that of higher frequency: For the acceleration of 16 Hz, it attenuates from 0.25g to 0.07g when it propagates from the table (point of excitation) to the height H/4 (from the base); however, for the acceleration of 7Hz, it marginally attenuates from 0.12g to 0.1g (Figure 9). The mechanism of this different attenuation rate may be dependent on the natural frequency of the sand layer, hence, further studies are necessary for its deeper understanding. The amplitude of acceleration response might increase with the number of shakings during which the densification of the sand layer increases correspondingly. **Figure 7** shows the accelerations measured by accelerometer A_2 at the height H/4 from the base. The peak values of acceleration with frequency 16 Hz (FFT spectrum based on measured accelerations) during each shaking event was obtained.

The normalized acceleration response R_{NA-2} (or R_{NA-4}) is the ratio of the peak acceleration measured by A_2 (or A_4) to that measured by A_1. R_{NA-2} and R_{NA-4} thus calculated in each shaking event is presented in **Figure 9**, where R_{NA-2} increases significantly with the number of shaking events. This shows that the attenuation of acceleration decreases with the number of shaking events. The reason for the decrease in attenuation is possibly related to the increase in relative density of the sand layer. However, R_{NA-4} increases marginally with the number of shaking events. This small increase in R_{NA-4} is possibly due to the strong mitigation effect of re-liquefied layer below A_4 layer (during the shear wave propagation). This study found that the attenuation of seismic shear wave acceleration is related to the liquefied layer. The reasons for the small R_{NA-4} (about 0.2) is not due to the small distance (20 cm from the shaking table to the position A_4), but probably due to the liquefied layer below A_4 where the shear module might be very small. Similarly, the value of R_{NA-2} is 0.2 ~ 0.3 at the first two shaking events which could be due to the liquefied lowest layer. To date, the effect of number of shaking and the position of the liquefied layer on attenuation of seismic shear wave has been overlooked. As the attenuation of seismic shear wave is very crucial factor for the evaluation of site liquefaction, the traditional attenuation calculation method needs its improvement in the future practice.

4. Conclusion

A series of shaking table tests were conducted to investigate the multiple liquefaction phenomenon in layered sand via its acceleration response and the excess pore water pressure, and following main conclusions have been derived:

(1) Besides the relative density of sand layer, other factors such as the structure or the fabrics of grains determine the re-liquefaction behaviour of sand because the lowest liquefaction resistance was observed in the second shaking event (not in the first shaking event where the relative density was the largest one among the all events). This is possibly due to the destruction of initial anisotropic sand fabrics during the first shaking leading to the increase in rise of PWP during the second shaking. Therefore, the relative density of sand layer does not uniquely determine the reliquefaction behaviour.

(2) Shallow sand layers are highly susceptible to multiple liquefaction in comparison to the deeper sand layers. Therefore, reliquefaction phenomenon is influenced by the depth of the targeted layer. However, further intensive research needs to be conducted to understand the depth-dependent behaviour considering the density and confining stress of that particular layer.

(3) Distance is not only the influential parameter for the attenuation of seismic shear wave acceleration, but is also affected by the number of shaking events, the vibration frequency and the position of liquefied layer. To be specific, the attenuation rate with lower frequency can be much smaller than that of higher frequency. Additionally, normalized acceleration response is influenced by the position of liquefied layer rather than the distance from the point of excitation. Hence, for evaluating the site liquefaction, the conventional method of calculation needs to be improvised.

5. References

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