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Effectiveness of Geosynthetic Encased Stone Column Technique For Liquefaction Mitigation

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Abstract. Soil liquefaction and its associated deformations cause problems to the structural safety of infra-structures. There are different ground improvement techniques available for improving the strength of liquefiable soil. Ground improvement using stone columns is a well-developed and popular method for improving the strength of soil and also for reducing the generation of pore water pressure and minimizing ground deformations during liquefaction. However, when stone columns are subjected to repeated dynamic loading, the performance reduces gradually. This is mainly due to intrusion of finer soil particles into the stone column causing column clogging which affects pore water dissipation and loss of effective column confinement as well. In this study, a provision of geosynthetic confinement around the stone column has been adopted for improving the performance. The present study is focussed on the liquefaction mitigation of Solani River sand by adopting encased stone columns. The stone columns embedded in the soil with and without encasement were subjected to repeated acceleration loading of 0.3g and 0.4 g at 5 Hz frequency. The stone column is installed in the soil bed filled in large tank of size 1.3×1×1 m. The proposed study focusses on various influencing factors such as the generation and dissipation of excess pore water pressure, pore pressure ratio and foundation settlement with and without geosynthetic encased stone columns installations. The obtained results show a significant reduction in the pore pressure ratio and the adjoining soil deformation values of the soil bed treated with encased stone columns compared to unreinforced soil bed.

Keywords: Liquefaction, Soil Improvement, Encased Stone Column, Pore Water Pressure, Foundation Settlement.

1 Introduction

Earthquakes cause the ground to shake or vibrate gently, and sometimes violently. Vertical ground vibrations cause only little damages to the building because all structures are designed to withstand most of the vertical loads. But horizontal ground vibrations create enormous stresses on structural elements, and if severe, leads to total collapse of the building.

Buildings constructed on bedrock perform well whilst those built on loosely packed fine-grained soil such as sandy soils, silty soils tend to undergo complex phenomena called liquefaction during a seismic event. Liquefaction is considered as one of the greatest risks of earthquakes. It can be defined as the phenomenon in which loose, saturated, cohesionless soil changes its state from solid to liquid, and begins to flow. This is mainly due to the generation of excess pore water pressure which annuls the effective stress and results in shear failure of the soil bed.

In the past history severe earthquakes had its contribution to the soil liquefaction, namely, Niigata (Japan) and Alaska (USA) earthquakes (1964), the Tangshan (China) earthquake (1976), the Kocaeli (Turkey) earthquake (1999), the Wenchuan (China) earthquake (2008), and the Chilean earthquake (2010). The greater the intensity of approaching seismic load, greater will be the generation of excess pore pressure and higher will be the soil deformation, resulting in the densification of soil bed in the post liquefaction stage [1]. Sometimes a series of seismic events can also be catastrophic as it results in re-liquefaction of the liquefied soil due to severe earthquakes in the past. For example, the earthquake that occurred in the Pacific Coast of Tohoku in Japan (2011) reveal the effects of re-liquefaction caused by the aftershocks [2]. There is a common idea among the researchers that densification of the soil stratum susceptible to liquefy will mitigate the liquefaction phenomena [3] [4] [5]. This paved a way to take up densification as a tool to control the effects of soil re-liquefaction [6] [7].

In the recent past, several ground improvement technologies have been developed in order to mitigate soil liquefaction effects, such as the vibro-compaction, compaction piles, stone columns or gravel drains etc. Vibro-compaction is a ground improvement technique that compacts granular soils and therefore the soil particles get rearranged into a denser form. Stone column or gravel drains are cylindrical column of stones, when installed using vibro-replacement technique will improve the surrounding soil by densifying it. It allows drainage of pore water generated under any dynamic loading. In addition, the higher modulus of elasticity compared to surrounding soil facilitates higher load carrying capacity and overall settlement reduction. Stone columns have proved to be the most successful techniques in mitigating soil liquefaction due to its effectiveness in a wide variety of soils, even at deeper depths [8] [9] [10].

Currently, extensive research is in progress to improve the performance of stone columns. One such development in stone column technology is by adding an encasement around the column using geosynthetics such as geotextiles and geogrids. Recently a number of studies have been conducted where the performance of geosynthetic encased stone columns under dynamic shaking have been assessed using numerical modelling and experimental analysis [11] [12] [13]. However, from a thorough review of the previous studies, it is understood that the performance of geosynthetic encased stone column in liquefaction and re-liquefaction assessment, under repeated dynamic shaking needs better focus. Therefore, this project is focused on studying the performance and behavior of geosynthetic encased stone columns under repeated incremental acceleration loading of 0.3g and 0.4 g dynamic loads, embedded in the Solani river sand for liquefaction and re-liquefaction assessment.

2 Materials and Methodology

2.1 Soil used for the bed preparation

The soil taken for the study was Solani river sand, which is locally available in the region. In order to prepare a sample similar to that of in-situ ground conditions, basic laboratory tests were conducted to identify the index properties of the sand. The soil contained higher amounts of finer particles from sieve analysis. The basic characterization tests were performed as per IS codal procedure and soil properties are listed in Table 1.

Table 1. Soil index properties

Sl. No.	Parameters		Value	Unit
1	Type of soil		Poorly graded Sand	
2	Specific Gravity (G)		2.65	No unit
3	Minimum Density (γ min)		1.4011	g/cc
4	Maximum Density (γ max)		1.6644	g/cc
5	40 % RD	Density (γ)	1.494	g/cc
		Permeability (k)	0.00811	cm/s
		Cohesion (c)	0	kPa
		Angle of internal Friction (φ)	32	◦
7		Youngs Modulus (E)	12000	kPa
8	Sand	Coarse (4.75-2 mm)	0.034	%
		Medium (2-0.425 mm)	9.77	%
		Fine (0.425-0.075 mm)	88.25	%
9	Silt and Clay (<0.075 mm)		1.946	%

2.2 Sample preparation

The laboratory model of the ground was prepared inside a rigid Perspex rectangular tank with dimension 1.3×1×1m, which is fixed on the shaking table using bolts (Figure 1). A 0.05m thick polyurethane foam was attached along the two walls of tank perpendicular to shaking direction in order to minimize the boundary effects on soil sample. Prior to sand bed preparation, leakage tests and sandblasting were done in order to confirm the application of shear stress and input ground accelerations from base to prepared ground. The liquefaction response of soil significantly depended on the method of sample preparation [14] [15] [16] [17].

For experimental studies, a 0.6m thick soil bed of 40% relative density was made layer by layer in order to mimic the real ground profile in the tank. To obtain an uniform saturation, wet sedimentation method was adopted [7]. In this process the required amount of water to fill a single layer was added and then an equivalent amount of sand was poured for the same layer. Thus, allowing the sand particles to settle down slowly throughout the water column, thereby confirming the complete saturation of the soil layer. The same procedure was repeated for subsequent layers too. The sand was rained down in the tank through a hopper having an inverted solid cone of 60° angle attached to the end [16], as shown in Figure 2. The hopper is placed at a pre-calculated height found out by performing repeated relative density tests as per IS 2720 – Part XIV and ASTM D4254 – 2006, to achieve 40% relative density.

Fig. 1. Perspex glass tank placed over shaking table

Fig. 2. Sample preparation

For the purpose of recording generation of excess pore water pressure during the shaking event, a glass tube piezometer was connected to the side of the tank at 0.2m and 0.4m height from the base of the tank. In addition to glass tube piezometers, strain-based pore pressure transducers were also placed at 0.2m and 0.4m at different positions to monitor generation and dissipation of pore water pressures. The transducers were connected to dynamic data acquisition system.

2.3 Installation method of Encased Stone Columns

For experimental investigations under reinforced condition, three encased stone columns (ESCs) having 0.16m diameter at 0.32m c/c spacing with area replacement ratio (ARR) 5% were installed in soil prepared with 40% relative density. The installation of encased stone columns involved, initially, punching a hollow PVC pipe of outer diameter equivalent to diameter of stone column vertically inside the saturated sand bed carefully until it hit the bottom of the tank. A geotextile material was used for the purpose of encasement. The boundary of geotextile material was tailor stitched using nylon threads to have maximum seam strength and was tied at the bottom, forming a geotextile sack. Once the pipe hit the bottom of tank, the saturated sand is excavated using a manually operated auger boring device, as shown in Figure 3. And the sack is then inserted into the casing pipe. The aggregates of size ranging between 0.002m to 0.01m is used to fill the geotextile sack layer by layer and compacted in order to achieve a density of 18±0.2 kN/m^3, followed by the withdrawal of the PVC pipe, as shown in Figure 4. Finally, the prepared soil sample reinforced with geosynthetic ESCs, was left undisturbed for 24 hours. The encased gravel drains where centrally arranged in a preferred triangular pattern around the scaled foundation model as per IS 15284 Part 1 (2003). Proper marking of the layout was done based on the design before installing the stone columns.

Fig. 3. Excavation of saturated soil using auger boring

Fig. 4. Filling of geotextile tube

2.4 Repeated incremental acceleration conditions for testing

In the present study, the liquefaction and re-liquefaction potential of the prepared laboratory scale saturated ground with and without geosynthetic encased stone column reinforced ground subjected to repeated shaking events were evaluated based on the acceleration loading selected for the study. Considering the PGA of several severe earthquakes, an acceleration of 0.3g and 0.4g at 5Hz frequency was selected and applied using a uni-axial shaking table. The shaking was carried out for 200 cycles lasting for time period of 40s and the generated pore pressure was recorded by the piezometers and strain-based transducers placed at 0.2m and 0.4m depths. The liquefaction and re-liquefaction characteristics of the prepared saturated ground bed were evaluated experimentally under repeated incremental acceleration loading conditions. The application of continuous incremental acceleration loading was applied to the ground bed only after the complete dissipation of generated excess pore water pressure from previous acceleration loading, the time taken for generation and dissipation of excess pore water pressures were monitored continuously using glass tube piezometers and pore water pressure transducers.

2.5 Scale down model of foundation

To estimate foundation settlement under repeated acceleration events, a scaled-down shallow footing model was placed over the prepared ground bed in order to study the foundation failure of a structure during liquefaction and reliquefication events. The procedure adopted for scaling of the foundation model is based on dynamic similitude laws given by [18] is shown in Equation 1.

$$N_{(EI)} = N_{(K)} \times N_{(L)}{}^{3} \tag{1}$$

Where,

$N_{(EI)}$ = Scale factor for flexural rigidity

$N_{(K)}$ = Scale factor for stiffness

$N_{(L)}$ = Scale factor for linear dimensions

A scale down factor (n) of 10 was used to model shallow footing. The foundation was modelled for 115mm length, 115mm wide, and 30mm thick using steel material having a modulus of elasticity 200GPa. The foundation model was installed at 30mm depth inside the prepared ground at center position for evaluating settlement during shaking. Along with LVDTs, the foundation settlement was measured manually using a meter scale in order validate the readings acquired by the sensors. In addition to foundation settlement, surface soil displacement was also estimated.

3 Results and Discussion

3.1 Effect of excess pore pressure generation on liquefaction potential

A comparative study is carried out based on the experimental data to understand the performance of encased stone column in liquefaction and reliquefaction under repeated shaking load. The generated excess pore pressure (EPP) and its corresponding pore pressure ratio at different depths for each incremental loading is compared and analyzed between unreinforced and reinforced conditions. The prepared samples with and without ESCs are shown in Figures 5 and 6 respectively.

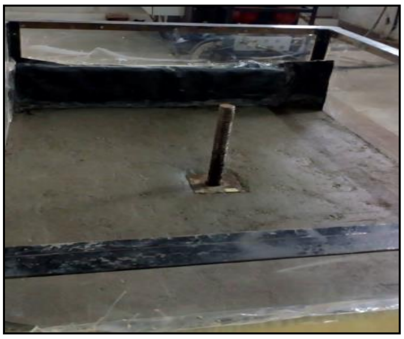

Fig. 5. Prepared unreinforced ground arrangement

Fig. 6. Photographic view of Encased stone column

The excess pore pressure (EPP) distribution curve for 0.3g acceleration loading at 200 cycles is represented in Figure 8. Under unreinforced soil condition, a maximum EPP of 2.76kPa was generated at the bottom point in 15s from the beginning of shaking, which is approximately 5s prior to the maximum EPP of 1.35kPa generated at the top point. Formation of sand boils were observed under dynamic loading, as shown in Figure 7. The soil stratum behaviour had similiar results obtained by [19] [20] [21]. However, in reinforced soil condition, there was more than 80% reduction in the generation of maximum EPP when compared to unreinforced soil condition.

Fig. 7. Formation of sand boils after 0.3g acceleration (Unreinforced condition)

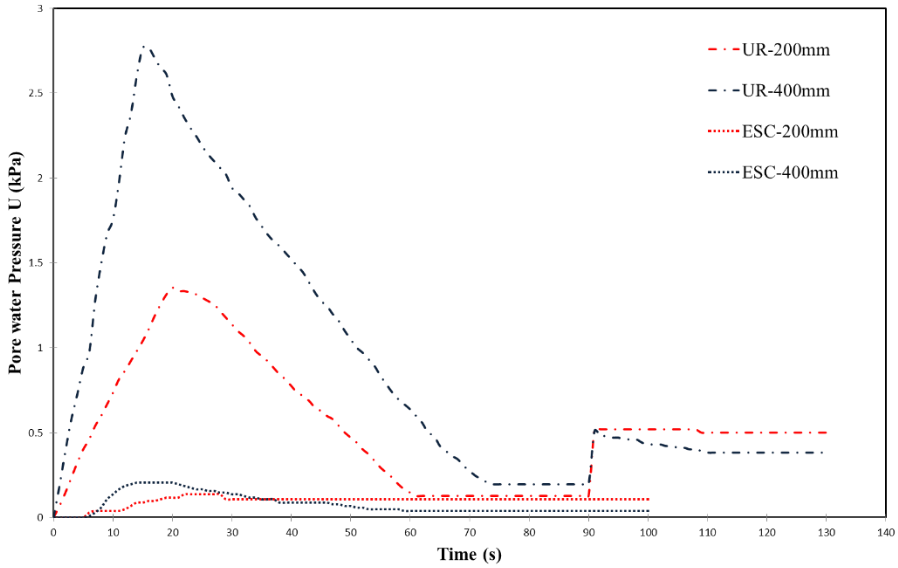

Fig. 8. Porewater pressure generation for 0.3g acceleration (200mm and 400mm, from top)

After 0.3g acceleration loading was performed on both unreinforced and reinforced soil samples, they were left undisturbed for 24 hours for settling down of excess pore water generated. Once the pore water pressure was stabilized, 0.4g acceleration was applied to each sample.

The excess pore pressure distrubution curve for 0.4g loading is represented in Figures 9. Under unreinforced condition, a maximum pore pressure of 2.79kPa was generated at the bottom point in 10 seconds of shaking. While a maximum pore pressure of 1.22 kPa was generated at the top point 5 seconds later. Similiar patterns of 0.3g acceleration loading was observed in the generation of excess pore pressure, expect for which, the magnitudes were greater in 0.4g loading. However, it is worthwhile noting that, as the intensity of the shaking increased, the time of generation of maximum EPP decreased. Even at high intensity shaking, the soil treated with geosynthetic ESCs exhibited approximately 75% reduction in the generation of excess pore pressure at the bottom point and very minimal pore pressure was generated at the top point when compared to unreinforced condition. Also, the sand boils and bed cracks were absent under reinforced soil condition. Thereby proving the effectiveness of geosynthetic encased stone columns in liquefaction mitigation.

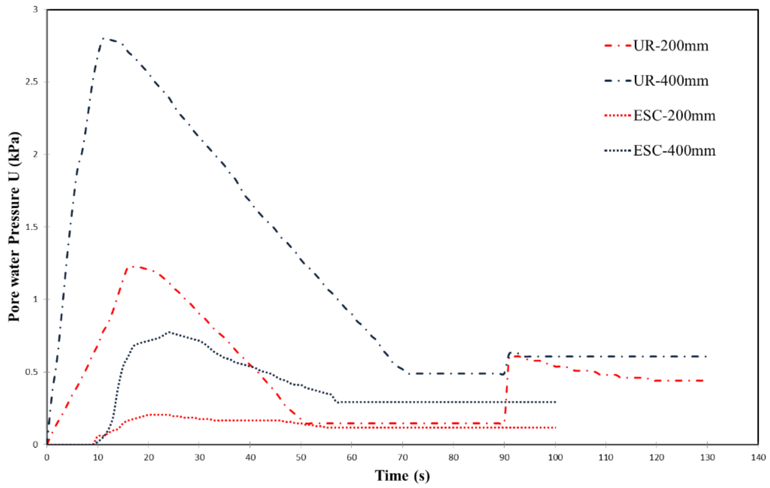

Fig. 9. Porewater pressure generation for 0.4g comparison (200mm and 400mm from the top)

3.2 Effect of pore pressure ratio (r_u)

Pore pressure ratio indicates liquefaction susceptibility of the soil. The variation of pore pressure ratio with respect to time for both unreinforced and reinforced condition subjected to 0.3g anf 0.4g loading have been plotted in Figure 10. Under unreinforced condition, a maximum pore pressure ratio of 0.66 and 0.67 was generated for 0.3g and 0.4g loading respectively. Thus, indicating that the soil was liquefied with the expulsion of water through the formation of sand boils on the surface. At the same time, when soil treated with ESCs, only a pore pressure ratio of 0.05 and 0.18 was computed for 0.3g and 0.4g loading respectively, because of the rapid dissipation of excess pore water through the vertical drains before liquefying soil.

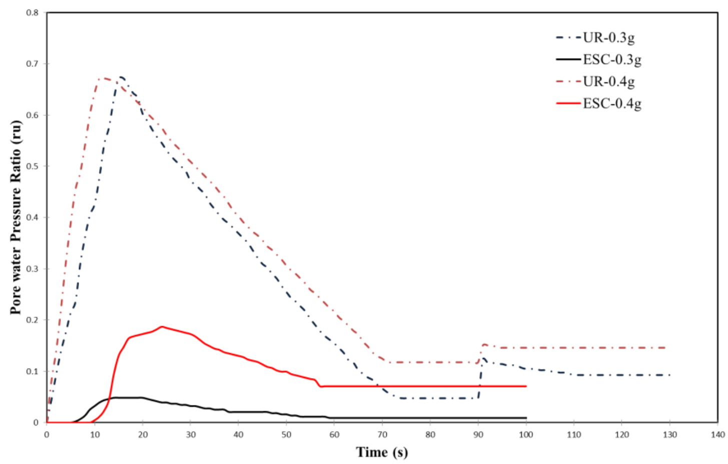

Fig. 10. Porewater pressure ratio for 0.3g and 0.4g (400mm from top)

3.3 Foundation settlement

The foundation settlement obtained from LVDTs for soil treated with and without encased stone columns under 0.3g and 0.4g loading have been compared in Figure 11. Failure of foundation accompanied with large settlement was observed when 0.4g acceleration (severe earthquake condition) was applied to the unreinforced liquefiable grounds. However, with the introduction of ESCs, there was more than 50% reduction in the foundation settlement. This is because the effective area of encased stone column overlapped with base of the foundation.

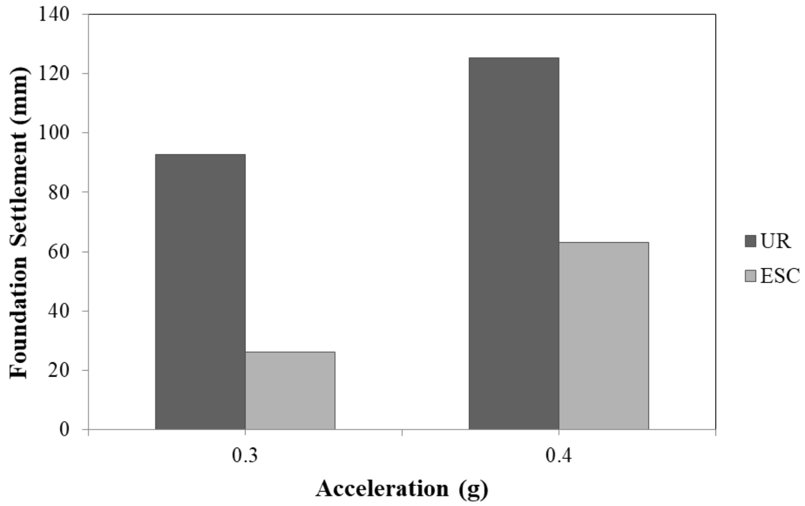

Fig. 11. Foundation Settlement

4 Conclusions

In this study, the effectiveness of geosynthetic encased stone columns in mitigating the liquefaction and reliquefaction potential of the soil was studied experimentaly. For this purpose, Solani river sand was taken as the soil sample, prepared at a relative density of 40%, was subjected to repeated incremental acceleration of 0.3g and 0.4g magnitude at 200 cycles. The parameters for determining the liquefaction behaviour of soil was the generation of excess pore pressure, foundation settlement and soil displacement. A maximum EPP of 2.76 kPa and 2.79 kPa was observed when 0.3g and 0.4g acceleration loading was applied to unreinforced soil. In both the loading, the soil was found to be liquefied. However, no significant EPP was generated when the soil was treated with geosynthetic encased stone columns. Also, there was more than 50% reduction in the foundation settlement when the soil was treated with geosynthetic ESCs. Thereby proving the effectiveness of geosynthetic ESCs in mitigating liquefaction and reliquefaction behaviour of soil.

References

1. Thevanayagam, S., Martin, G. R., Shenthan, T., & Liang, J. (2001). Post-liquefaction pore pressure dissipation and densification in silty soils.
2. Huang, Y., & Yu, M. (2013). Review of soil liquefaction characteristics during major earthquakes of the twenty-first century. Natural hazards, 65(3), 2375-2384.
3. Krishna, A. M., Madhav, M. R., & Latha, G. M. (2006). Liquefaction mitigation of ground treated with granular piles: Densification effect. ISET Journal of earthquake technology, 43(4), 105-120.
4. Mitchell, J. K. (2008). Mitigation of liquefaction potential of silty sands. In From research to practice in geotechnical engineering (pp. 433-451).
5. Salem, Z. B., Frikha, W., & Bouassida, M. (2017). Effects of densification and stiffening on liquefaction risk of reinforced soil by stone columns. Journal of Geotechnical and Geoenvironmental Engineering, 143(10), 06017014.
6. Ye, B., Yokawa, H., Kondo, T., Yashima, A., Zhang, F., & Yamada, N. (2006). Investigation on stiffness recovery of liquefied sandy ground after liquefaction using shaking-table tests. In Soil and Rock Behavior and Modeling (pp. 482-489).
7. Ye, B., Ye, G., Zhang, F., & Yashima, A. (2007). Experiment and numerical simulation of repeated liquefaction-consolidation of sand. Soils and Foundations, 47(3), 547-558.
8. Adalier, K., & Elgamal, A. (2004). Mitigation of liquefaction and associated ground deformations by stone columns. Engineering Geology, 72(3-4), 275-291.
9. Seed, H. B., & Booker, J. R. (1977). Stabilization of potentially liquefiable sand deposits using gravel drains. Journal of Geotechnical and Geoenvironmental Engineering, 103(ASCE 13050).
10. Baez, J. I., & Martin, G. R. (1992). Quantitative evaluation of stone column techniques for earthquake liquefaction mitigation. In Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, Spain (Vol. 3, pp. 1477-1483).
11. Cengiz, C., & Güler, E. (2018). Seismic behavior of geosynthetic encased columns and ordinary stone columns. Geotextiles and Geomembranes, 46(1), 40–51. doi:10.1016/j.geotexmem.2017.10.001
12. Chen, J.-F., Li, L.-Y., Xue, J.-F., & Feng, S.-Z. (2015). Failure mechanism of geosynthetic-encased stone columns in soft soils under embankment. Geotextiles and Geomembranes, 43(5), 424–431. doi:10.1016/j.geotexmem.2015.04.016
13. Chen, J.-F., Wang, X.-T., Xue, J.-F., Zeng, Y., & Feng, S.-Z. (2018). Uniaxial compression behavior of geotextile encased stone columns. Geotextiles and Geomembranes, 46(3), 277–283. doi:10.1016/j.geotexmem.2018.01.003
14. Mulilis, J. P., Arulanandan, K., Mitchell, J. K., Chan, C. K., & Seed, H. B. (1977). Effects of sample preparation on sand liquefaction. Journal of the Geotechnical Engineering Division, 103(2), 91-108.
15. Maheshwari, B. K., Singh, H. P., & Saran, S. (2012). Effects of reinforcement on liquefaction resistance of Solani sand. Journal of Geotechnical and Geoenvironmental Engineering, 138(7), 831-840.
16. Varghese, R. M., & Latha, G. M. (2014). Shaking table tests to investigate the influence of various factors on the liquefaction resistance of sands. Natural hazards, 73(3), 1337-1351.

17. Banerjee, R., Konai, S., Sengupta, A., & Deb, K. (2017). Shake Table Tests and Numerical Modeling of Liquefaction of Kasai River Sand. Geotechnical and Geological Engineering, 35(4), 1327-1340.
18. Moncarz, P. D., & Krawinkler, H. (1981). Theory and application of experimental model analysis in earthquake engineering (Vol. 50). California: Stanford University.
19. El-Sekelly, W., Abdoun, T., & Dobry, R. (2016). Liquefaction resistance of a silty sand deposit subjected to pre-shaking followed by extensive liquefaction. Journal of Geotechnical and Geo-environmental Engineering, 142(4), 04015101.
20. Darby, K. M., Boulanger, R. W., & DeJong, J. T. (2018). Volumetric strains from inverse analysis of pore pressure transducer arrays in centrifuge models. In Geotechnical Earthquake Engineering and Soil Dynamics V: Liquefaction Triggering, Consequences, and Mitigation (pp. 626-636). Reston, VA: American Society of Civil Engineers.
21. Padmanabhan, G., & Shanmugam, G. K. (2020). Reliquefaction Assessment Studies on Saturated Sand Deposits under Repeated Acceleration Loading Using 1-g Shaking Table Experiments, Journal of Earthquake Engineering, 1-23.