Effect of distribution pattern of DSM columns on the efficiency of liquefaction mitigation

H. DehqanKhalili, A. Ghalandarzadeh*, M. Moradi, R. Karimzadeh

School of Civil Engineering, University College of Engineering, University of Tehran, Tehran, Iran

*Corresponding author. Tel: +98 21 66462616 & Fax: +98 21 66403808
Email address: aghland@ut.ac.ir

Abstract.
Liquefaction during earthquakes can result in severe damage to structures, primarily from excess pore water pressure generation and subsoil softening. Deep Soil Mixing (DSM) is a common method of soil improvement and is also used to decrease shear stress in liquefiable soils to control liquefaction. The current study evaluated the effect of Deep Soil Mixing (DSM) columns and implementation of different column patterns on controlling liquefaction and decreasing settlement of shallow foundations. A series of shaking table physical modelling tests were conducted for three different distribution patterns of Deep Soil Mixing (DSM) columns (i.e.: square, triangular and single) with a treatment area ratio of 30%. The treatment was applied to a liquefiable soil under a shallow model foundation. The results showed that the excess pore water pressure decreased 20% to 50% in comparison with the unimproved soil, depending on the Deep Soil Mixing (DSM) column pattern used. For improved soil, the shallow foundation settlement was about 10% that of the unimproved soil in the best case. The increase in soil shear stiffness after use of the Deep Soil Mixing (DSM) columns was compared with the results of existing practical relations to increase soil shear strength.

Keywords: Liquefaction mitigation; Deep soil mixing; Shaking table; Shear reinforcement

1 Introduction

Liquefaction is a destructive phenomenon which occurs in loose, saturated, sandy soil during an earthquake. It was the cause of heavy damage in the Niigata (1964) [1], Alaska (1964) [2] and Kobe (1995) [3] earthquakes. In all cases, liquefaction-induced deformation (e.g., lateral spreading) caused slope failure, failure of the foundations of bridges and
buildings and the destruction of underground structures. When rapid two-way shear stress is imposed on saturated sandy soil by seismic movement, it triggers an increase in pore water pressure [4]. The pore water pressure increases rapidly in loose cohesionless saturated soil and may result in the suspension of soil grains and in the soil strength and stiffness falling to zero for a few moments [5]. The behaviour of the sand suddenly changes from solid to liquid and could behave as a viscous flow. It is challenging to compare the different methods of controlling liquefaction in susceptible soil to choose the most appropriate method. Installation of vertical discrete columns (Stone, Deep Soil Mixing or Jet Grouting Columns) is a popular method in loose, saturated, cohesionless soils. The elements are set in a wall or single pattern to enclose the soil mass. The shear strain caused by an earthquake decreases as the average shear stiffness of the soil mass increases, which decreases the excess pore pressure. This provides a barrier to the transmission of excess pore water pressure from an unimproved area to the improved area [5]. Even if liquefaction occurs, lateral soil deformation or vertical settlement of the foundations will decrease because of the inherent resistance of the columns.

A limited number of case studies have been used to evaluate the performance of the Deep Soil Mixing (DSM) columns; however, during the Kobe earthquake (1995), it was observed that Deep Soil Mixing (DSM) walls performed effectively [6-7]. The efficiency and performance of Deep Soil Mixing (DSM) single or wall pattern columns in preventing liquefaction have been analytically investigated and numerically simulated. The investigations showed that treatment area ratio is a major parameter controlling Deep Soil Mixing (DSM) grid effectiveness [6,8,9,10].

Numerical simulations [11,12,13], suggest that Deep Soil Mixing (DSM) columns deform under shear stress and in flexure such that the benefit of shear reinforcement mechanisms decreases significantly. Deep Soil Mixing (DSM) grid performance has also been studied using physical modelling. The results of centrifuge tests [10,11] demonstrate that wall spacing, wall depth and input motion frequency are effective variables in shear grid performance to reduce excess pore water pressure generated in saturated, loose sand.

The shear reinforcement mechanism of Deep Soil Mixing (DSM) columns is often studied by assuming that the columns and surrounding soil exhibit shear strain deformation compatibility. Using this assumption, a significant portion of earthquake-induced shear stress is absorbed by the stiffer elements of the system (Deep Soil Mixing (DSM) columns)
and causes a considerable decrease in the cyclic stress ratio in the surrounding soil and the potential of liquefaction [14,15].

The estimated increase in soil shear strength induced by Deep Soil Mixing (DSM) columns can be determined by different methods. One is the relation provided and extended to soil-cement columns which assumes shear strain compatibility between columns and the surrounding soil. [14,15]. The second is the theoretical bounds [16]. The third is the design equation which considers differences in the shear strain between columns and the surrounding soil. These analysis methods can be used to evaluate the average shear modulus for improved and unimproved soil. The finite element analysis developed by this method showed that shear strain incompatibility increases as the stiffness of the discrete columns increases [17]. For a realistic area replacement ratio for Deep Soil Mixing (DSM) columns, only a 10% to 30% decrease in shear stress is achievable [18].

The potential decrease in liquefaction and shear stress using Deep Soil Mixing (DSM) columns in the soil has not been investigated quantitatively. The effects of the different design variables (column implementation pattern, column length and treatment area ratio) have not been thoroughly explored. In the current study, a series of shaking table physical modelling tests were conducted to evaluate the quantitative effect of Deep Soil Mixing (DSM) columns and the effectiveness of implementation of different column patterns to control liquefaction and shallow foundation settlement. The relationship presented by Rayamajhi et al. (2014) [17], was investigated to estimate the increase in shear strength induced by Deep Soil Mixing (DSM) columns. The use of Deep Soil Mixing (DSM) columns to reduce the liquefaction risk and determine optimal implementation of column patterns was also investigated.

2 Shaking table tests

2.1 Test method

Table 1 compares the factors related to the model and prototype. The procedure for simulation of 1g shaking table tests has been proposed by Iai et al (1999) [19]. Table 1 provides information about scaling factors and selected values in the tests.

Firouzkooh sand (Gs = 2.69, e_{max} = 0.87 and e_{min} = 0.608) was used to create the model ground, which consisted of two layers. The bottom layer was 50 cm in height and was
assumed to be non-liquefiable and was compacted to achieve 90% of the maximum soil density. The upper layer was 45 cm in height and was made liquefiable using the wet tamping method to a relative density of 20%. The ground water level was 5 cm below the soil surface.

The 45 cm soil-cement columns were fabricated with a diameter of 5 cm inside the liquefiable soil using a deep soil mixing apparatus designed and built for this purpose (Figure 1). The soil-cement mixing was done by a shaft having six blades and two nozzles with a rotation speed of 20 rpm, a penetration rate of 1 m/min and a withdrawal speed of 0.5 m/min. Figure 2 shows that the Deep Soil Mixing (DSM) columns created by this machine are high-quality and homogeneous with a constant diameter throughout. For each column, 400 cc of grout was injected into the soil. Ordinary Portland cement grout with a water-cement ratio of 1:6 was used to construct the columns to produce the targeted unconfined compressive strength of 0.35 MPa after three days of curing.

2.2 Testing program

Figure 3 is a schematic of the test model configuration. A soil box 1.8 m in length, 1.2 m in height and 0.8 m in width was used. A different arrangement of soil cement columns were implemented in each model, giving a treatment area ratio \( A_t \) of about 30%. It was assumed that the end of the columns reached the surface of the non-liquefiable soil layer.

After construction of the Deep Soil Mixing (DSM) columns, an 18 kg rigid box with dimensions of 40 x 40 cm was placed on the Deep Soil Mixing (DSM) columns as a model raft foundation of a three-storey building. Several pore water pressure transducers and accelerometers were installed in the soil model (Figure 3). The vertical displacement of the soil surface was measured by four LVDT sensors. The models were shaken by sinusoidal waves at the 350 gal input level at a frequency of 3 Hz for duration of 5 s.

Table 2 summarises the experiments in which the effects of column type were investigated. As shown, the first model was unimproved and the second, third and fourth models each contained several Deep Soil Mixing (DSM) columns. The improvement area ratio was 30% in these three models. Figure 4 shows the different patterns and placement of the Deep Soil
Mixing (DSM) columns. In each model, the foundation is located on top of the Deep Soil Mixing (DSM) columns.

3 Experimental result and discussion

3.1 Settlement

Two LVDTs were used to measure settlement of the foundation. Figure 5 shows the amount of settlement caused by shaking at 0.35g. Figure 6 shows photographs of the UNI test model before and after shaking in which liquefaction-induced deformation can be clearly seen. The settlement measurements indicate that the Deep Soil Mixing (DSM) columns reduced vertical settlement of the foundation and the nearby unimproved area in comparison to the results of the UNI test. Figure 5 shows that, from T=2sec to T=4sec since the beginning of the tests was big difference between foundation settlements of UNI test sample than in the other tests. The initial rapid settlement shows very fast softening of the subsoil. Generation of maximum excess pore water pressure (EPWP) in the early cycles is also in accordance with the expected behaviour.

Settlement in the SQP, TRP and SGP tests appears to be reasonably controlled. Deep Soil Mixing (DSM) improvement decreased the rate and maximum amount of settlement and delayed the settlement initiation time. Changes in the column implementation pattern had little impact on foundation settlement and the free field surface. Settlement of the foundation in the improved soil cases was about 10% that of unimproved soil; however, the SGP test showed the best results for settlement reduction.

One main feature of the Deep Soil Mixing (DSM) columns is their ability to support overlying structures even if liquefaction occurs in the surrounding soil, which reduces settlement. In the present models, the Deep Soil Mixing (DSM) columns extended to the top of the stiff soil layer (non-liquefiable dense layer), but did not penetrate it. No crack or fracture was found on the Deep Soil Mixing (DSM) columns and suitable performance was observed in the tests. The columns in the SQP, TRP and SGP tests remained undamaged, even after shaking (Figure 2).

Figure 5 shows that all foundation settlement occurred during the 5 s shaking period in the SQP, TRP and SGP tests. In the UNI test, however, 90% of foundation settlement occurred
during shaking, with a small portion contributed by post-shaking soil reconsolidation due to excess pore water pressure dissipation. This means that the inertial forces caused by shaking played an important role in reshaping the soil foundation system [20,21]. In addition, a large portion of foundation settlement occurred because of penetration of the footing into the liquefied soil [21].

3.2 Excess pore water pressure

Pore water pressure (PWP) transducers were installed at different locations in the soil to record the time history of the excess pore water pressure. PWP records from depths of 4.5 and 7.5 m showed that high excess PWP developed in the early stages of shaking and was maintained during shaking. Figure 7 shows that excess pore pressure under the foundation developed during the three cycles of shear deformation in unimproved soil (UNI test; z = -7.5 m). It appears that the development of excess pore pressure is initiated by a minimum shear strain of almost 1%. In this study, the time histories of the normalized excess pore water pressure (Ru) were recorded at different locations.

In Figure 8, at deeper locations, the excess pore water pressure started to dissipate after shaking and increase in the shallower layers because of the upward movement of water from the lower layers. Such a trend was observed during the 1995 Kobe earthquake, where upward seepage was evident for up to an hour after the earthquake [21]. This water flow could reduce the strength of the shallower layer or generate secondary liquefaction, causing large settlement or loss of bearing capacity [22]. Figure 8 shows that the excess PWP in the DSM-improved zone was less than in the unimproved zone (free field) at different depths. A comparison of the improved and unimproved tests clearly shows that Ru decreased after DSM column improvement. This difference was more evident at a greater depth (-7.5 m). The decrease in excess pore water pressure was 20% to 50% depending on the depth.

Figure 8 showed no significant difference in the increase of pore water pressure in the three column patterns; however, the single and triangular patterns showed slightly less increase in pore pressure than the foursquare pattern, especially at greater depths. This figure also shows negative PWP, or suction, in the soil enclosed by the columns in the first cycles of shaking. This could result from a reduction in water level and permeability which led to deterioration of the water flow between the Deep Soil Mixing (DSM) columns. Suction was
also seen in areas close to the walls of the box. This is a limitation of the rigid box physical model and this error can be controlled by using a laminar box [21].

In the UNI test, the Ru values below the foundation were lower than the free field values at the same levels. This could be completely reverse if there no foundation was placed on the soil, because the soil near the surface is more sensitive to liquefaction than the soil at greater depths [21]. Earlier centrifuge and 1g shaking table tests on foundations supported by liquefiable sand have revealed that excess PWP is lower under the foundation than in the free field [23,24]. This can be explained by the shear-induced dilative soil response during deformation of the saturated soil below the footing. Moreover, foundation loads decrease the liquefaction potential [25]. Similar behaviour was observed by Koga et al. (1990) [26] and Sadrekarimi et al. (2005) [20], in 1g shaking table tests.

### 3.3 Soil behaviour

Figure 9 shows the acceleration time histories for the SQP test at 0.35g, 3 Hz and 5 s of shaking. During shaking, the acceleration amplitude in the soil decreased from the bottom of the soil to the surface in response to the increase in PWP and the consequent liquefaction. The dynamic shear stress and shear strain time series were computed at different depths for each model using accelerometer array data. In these analyses, a 1D shear beam condition was assumed and the procedures developed by Brandenberg et al. (2009) [27], and Kamai et al. (2010) [28], were used.

The shear strain was computed through double integration of acceleration to obtain the horizontal displacement and then differentiating the displacement with respect to depth to produce the shear strain. A high-pass Butterworth filter was applied to accelerations having an order of 4 and a corner frequency (fc) of 1.0 Hz, which gives reasonable displacement. In improved models, the stress-strain responses cannot be used to identify how the shear stresses are distributed between columns and the surrounding soil, but they can provide a basis for evaluating how the columns affect overall system stiffness [18]. By calculating the cyclic shear strain and shear stress, the stress-strain behaviour of the soil can be plotted. Figure 10 shows the stress paths at depths of -4.5 and -7.5 m in the UNI test. The stress paths indicate that the effective stress in the soil decreased significantly as excess pore pressure developed in the unimproved soil.
The stress-strain behaviour of the unimproved soil is shown in Figure 11. In the UNI test, soil stiffness in the free field decreased rapidly and the stress-strain curve became a horizontal line denoting very high damping and zero stiffness. The unimproved soil under the foundation showed stiffer behaviour than the free field stress points.

The stress-strain behaviour of the improved soil models is shown in Figure 12. In these models of SQP, TRP and SGP tests, it was predicted that soil would maintain its resistance. The maximum shear strain in the soil was lower for the improved cases (2%-5%) than for the unimproved case (~8%). In the shallow layers of the improved models, the peak shear strain was observed in the stress-strain responses at 25%-60% of the values shown in UNI test, despite the similarity of the peak shear stresses. These results indicate that the overall behaviour of the Deep Soil Mixing (DSM) columns increased their ability to stiffen the soil profile. According to the Figure 13, shear modulus of the soil increased 70% especially at the first eight cycles after implementation of the stiff Deep Soil Mixing (DSM) columns; thus, the columns helped reduce the shear strain in the surrounding soil and reduce lateral displacement in the soil. Figure 12 shows the stress-strain hysteresis loops for single, triangular and foursquare patterns at two depths under the foundation. A comparison of the results of different patterns shows that the single pattern exhibited roughly stiffer shear behaviour and less strain under the same shear stress than the other patterns. It can be said that, in this pattern, the volume of soil which was enclosed between the columns was lower than for the other two patterns, so the shear deformation decreased. Moreover, it can be seen that the shear strain at the first loading cycle in the free field was certainly greater than that in the improved areas in all tests. The Deep Soil Mixing (DSM) columns under the foundation slightly affected the shear behaviour of the nearby area and areas farther away from the foundation.

3.4 Analysis of shear reinforcing effect of DSM columns

Estimates of the increase in soil shear strength induced by the Deep Soil Mixing (DSM) columns was analysed using the relation provided by Baez (1995) [14], and the extended to soil-cement columns method by Durgunoglu (2006) [15], which assumes shear strain compatibility between columns and the surrounding soil. It was then analysed using the theoretical bounds proposed by Gueguin et al. (2013) [16], and the design equation presented by Rayamajhi et al. (2014) [13], which consider differences in the shear strain between the columns and surrounding soil. Using Rayamajhi et al. (2014) [13], the average...
shear modulus for improved soil, $G_{avg}$, is divided by the small-strain shear modulus for unimproved soil, $G_s$, and can be estimated as:

\[
\frac{G_{avg}}{G_s} = \left( \frac{\tau_{avg}}{\gamma_{avg}} \right) \cdot \frac{1}{G_s} = \frac{1+A_r(\gamma_r G_r-1)}{1+A_r(\gamma_r-1)}
\]

2. \[
\gamma_r = \frac{\gamma_{soil-cement}}{\gamma_{soil}} = 1.04 \cdot (G_r)^{-0.65} - 0.04
\]

where $\tau_{avg}$ is the average shear stress for improved soil, $\gamma_{avg}$ is the average shear strain for improved, $G_r = G_c/G_s$ is the shear modulus ratio, $G_c$ is the small-strain shear modulus of the column, $\gamma_r$ is the shear strain ratio, $\gamma_{soil-cement}$ is the shear strain in the soil-cement column, $A_r$ is treatment area ratio by Deep Soil Mixing (DSM) and $\gamma_{soil}$ is the shear strain in the soil. This relationship considers shear strain incompatibility between the columns and the surrounding soil using the term $\gamma_r$. When $\gamma_r = 1$, this relation corresponds to the assumption of shear strain compatibility [5].

In this research, the shear stiffness values for the unimproved soil profile ($G_s$) and experimental shear stiffness for improvement tests ($G_{avg}$) were calculated for cycles of each hysteresis curve at UNI test (Figure 11) and cycles on the hysteresis chart related to SQP, TRP and SGP tests for points in soil and under the foundation (Figure 12). Figure 13 shows that an increase in the number of cycles during shaking and in PWP gradually decreased the shear stiffness. Soil improvement using Deep Soil Mixing (DSM) columns also increased the shear stiffness in comparison to that of the unimproved soil.

The small-strain shear moduli for the Deep Soil Mixing (DSM) columns ($G_c$) were estimated to be 110 MPa. Young's secant modulus was evaluated by unconfined compression testing on the Deep Soil Mixing (DSM) column material after three days of curing (immediately after the shaking table tests). This secant shear modulus can be calculated using the theory of elasticity by assuming a Poisson's ratio of 0.2. The small-strain shear modulus for the Deep Soil Mixing (DSM) column was then estimated to be 50% greater than the secant value. This was a result of known limitations in conventional unconfined compression testing for determining small-strain moduli [14].

In Figure 14, the normalized average shear modulus ratio ($G_{avg}/G_s$) computed using Eq. (1) and experimental $G_{avg}/G_s$ related to SQP, TRP and SGP tests were plotted versus Number of cycles for the $A_r=30\%$. Figure 14 shows that changing the Deep Soil Mixing (DSM) column...
pattern had little impact on the increase in shear stiffness during shaking but the SQP pattern has less shear stiffness than other patterns. The experimental $G_{\text{avg}}/G_s$ values for improvement cases were close to $G_{\text{avg}}/G_s$ using Eq. (1) in the shakes before the fifth cycle, but with increasing cycles, there was a significant difference between the two values of $G_{\text{avg}}/G_s$.

4 Conclusion

In this experimental study, four shaking table tests were conducted to investigate the effect of Deep Soil Mixing (DSM) columns as well as the patterns of column implementation on liquefiable sand. An unimproved soil model and three improved soil models using Deep Soil Mixing (DSM) columns in different patterns were tested. The major observations from testing are:

(a) DSM improvement can reduce the rate of settlement as well as the maximum amount of settlement. The settlement of a foundation in the improved soil cases was 10% that of the unimproved case. The implementation pattern of the columns had little impact on settlement of the foundation or the free field surface; however, the single pattern showed less settlement in comparison to the other patterns.

(b) Comparison of the unimproved soil case and the improved cases clearly showed that $R_u$ decreased after Deep Soil Mixing (DSM) column improvement. This decrease in excess pore water pressure was 20% to 50% depending upon the depth. The single and triangular patterns showed less excess pore pressure than the foursquare pattern, especially at greater depths.

(c) In the unimproved soil case, the $R_u$ value during shaking was lower below the foundation than in the free field at the same depth. The foundation loads decreased the liquefaction potential as has previously been proven by Adalier et al. (2002) [24], Koga et al. (1990) [26], and Sadrekarimi et al. (2005) [20].

(d) Negative pore water pressure (or suction) was recorded between the Deep Soil Mixing (DSM) columns in the initial cycles of shaking.

(e) During shaking, the maximum shear strain ($\gamma_{\text{max}}$) in the improved soil was lower (2%-5%) than for the unimproved soil (~8%). In the shallow layers of the improved
models, the peak shear strain observed in the stress-strain responses was 25%-60% of the values shown in the UNI test, despite the similar increases in peak shear stress.

(f) The increase in shear stiffness from the Deep Soil Mixing (DSM) columns by \( A_r=30\% \) was roughly 1.5 times for the first five cycles and 3 times for the subsequent cycles in comparison to that of the unimproved soil. The column implementation pattern had little impact on the increase in shear stiffness during shaking but the SQP pattern has less shear stiffness than other patterns.

(g) The results of the current shaking table tests and the analysis by Rayamajhi et al. (2014) [13], which considers flexure of the reinforcing columns, provided reasonable (slightly low) estimates for the effect of shear reinforcement of DSM columns in low shear strain and first cycles.

(h) The results of this study showed that the single (SGP) and foursquare (SQP) patterns can transfer acceleration more easily and impose more acceleration on the overhead structure.

(i) It appears that, in order to decrease the hazard of liquefaction, the single pattern was a more appropriate alternative than the other two alternatives. The major reason for this was that the pore water pressure and foundation settlement for the single pattern were lower than for the foursquare and triangular patterns.

References


Table 1. Comparison of 1g test for model and prototype.

Figure 1. Soil mixing apparatus.

Figure 2. Soil mixing columns (D = 5 cm, H = 45 cm).

Figure 3. Configuration of 1g model tests.

Table 2. Test arrangements.

Figure 4. Deep Soil Mixing (DSM) column patterns: (a) square (SQP); (b) triangular (TRP); (c) single (SGP).

Figure 5. Settlement of foundation after 0.35g of shaking.

Figure 6. Model deformation showing liquefied layer: (a) before shaking; (b) after shaking.

Figure 7. Excess pore water pressure vs. shear strain in UNI test (z = -7.5).

Figure 8. Time histories of normalized excess PWP (Ru) at different locations.

Figure 9. Acceleration time history models for: SQP at 0.35g, 3 Hz and 5 s of shaking.

Figure 10. Shear stress vs. shear strain of unimproved soil test a) Z=-7.5m, b)Z=-4.5m.

Figure 11. Stress paths of unimproved soil under foundation (UNI test): (a) and (b) under foundation; (c) and (d) outside of foundation.

Figure 12. Shear stress vs. shear strain in improved soil under the foundation: (a) & (b) SQP; (c) & (d) TRP; (e) & (f) SGP.

Figure 13. Plot of shear modulus (G) vs. number of cycles (N) for three DSM patterns and the unimproved case.

Figure 14. Plot of average shear modulus for Deep Soil Mixing (DSM) columns based on the shear reinforcement mechanism from shaking table tests using Rayamajhi et al. [13].
Table 1.

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Figure 1.

Figure 2.
Figure 3.

Table 2.

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Figure 4.
**Figure 5.**

![Graph showing settlement over time](image)

**Figure 6.**

![Images depicting foundation](image)

(a) ![Foundation Image](image)  
(b) ![Foundation Image](image)

**Figure 7.**

![Shear strain vs. EPWP](image)
Figure 8.

Figure 9.
Figure 10.

(a)  

(b)  

Figure 11.

(c)  

(d)
Figure 12.
Brief Technical Biography

**Hesam Dehqan Khalili** received his BSc degree in Civil Engineering from Hormozgan University of Technology in 2006 and his MSc degree in Geotechnical Engineering from Shahid Beheshti University in 2008. He has professional experiences in design and execution of soil improvement methods. He is currently PhD student at the School of Civil Engineering at the University of Tehran. His research and engineering interests include Experimental Soil Mechanics, Liquefaction Problems, Soil Improvement, Physical modeling, and designing DSM and Jet Grouting to solve Foundation settlement problems.

**Abbas Ghalandarzadeh** is an associate professor at School of Civil Engineering, University of Tehran, where he is also currently Head of the Soil Mechanics and Centrifuge Laboratory. He received his Ph.D. in Geotechnical Engineering from the University of Tokyo in 1997. He
is a Member of the Technical Committee of TC2 of the International Society of Soil Mechanics and Geotechnical Engineering. His research interests are mainly in the area of Experimental Geotechnics, particularly in model and element testing. Dr. Ghalandarzadeh has undertaken much research in the area of Earthquake Geotechnical Engineering including specifically: the dynamic behavior of rockfill dams with asphalt concrete cores, the seismic behavior of quay walls, reinforced earth and soil improvement techniques, and more recently dynamic behavior of all types of geomaterials.

Majid Moradi is an Associate Professor at School of Civil Engineering, College of Engineering, University of Tehran, Iran. He received his B.Sc in Civil Engineering in 1989 and M.Sc in Geotechnical Engineering in 1992 both from University of Tehran, and Ph.D in Geotechnical Engineering from University of Manchester, U.K in 1999. He has a broad experience in physical modeling in geotechnics especially employing geotechnical centrifuge facility. Moreover, he has supervised numerous M.Sc. and Ph.D students with the research subject of the physical and numerical modeling. He is author and co-author of several research papers in the field of geotechnical engineering especially physical modeling with centrifuge facility.

Reza Karimzadeh received his BSc degree in Civil Engineering from Tabriz University in 2014 and his MSc degree in Geotechnical Engineering from University of Tehran in 2017. His present research interests include Shaking table and centrifuge Physical modeling and Geo-environmental problems.