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Applicability Analysis of Stone Column Against Liquefaction Under Repeated Dynamic Events

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Abstract. In the event of sudden earthquakes, loose sandy soil loses its shear strength tremendously due to the generation of excess pore water pressure causing liquefaction. Due to soil liquefaction, the infrastructures' foundation becomes unstable, thereby causing threats to its structural stability. Hence the application of suitable ground improvement methods becomes necessary for improving the liquefaction susceptibility of sandy soils. Ground reinforcement using the stone column proves to be an efficient reinforcement technique for improving liquefaction resistance of sandy soils. In this study, the stone column improvement technique was evaluated experimentally under repeated acceleration loading events. The experiments were carried out in a tank of dimension 1.4×1.0×1.0 m, placed over a uniaxial shaking table. A saturated sand bed having 40% relative density was prepared and tested under repeated acceleration loading of 0.3g and 0.4g at 5 Hz Frequency sequentially. A comparative analysis between untreated virgin sand bed and stone column treated ground was performed for performance assessment. The liquefaction assessment parameters such as generation and dissipation of pore water pressures, foundation settlement and soil displacement were monitored and compared. The results showed that there is a significant reduction in the excess pore water pressure generation even at repeated acceleration loading for the stone column stabilized bed. The test results concluded that soil densification and drainage are the key features in liquefaction mitigation system to enhance the safety of foundation structures in the seismic prone areas.

Keywords: Liquefaction; Soil improvement; Stone column; Pore water pressure; settlement.

1 Introduction

Soil is a complex component which shows variety of problematic behavior under dynamic loading conditions. One such most common complex phenomenon is liquefaction, occurring after an earthquake in loose sandy deposits. During shaking, the saturated sand completely loses its shear strength affecting the stability of foundation structures. Some of the evidence includes Niigata, Japan (1964) and Alaska (1964), the Tangshan, China (1976), the Kocaeli, Turkey (1999), the Wenchuan, China

(2008), and the Chilean earthquake (2010). The greater the magnitude of approaching seismic load, the greater the generation of excess pore pressure, and the higher will be the soil deformation. During this event, the excess pore water pressure generated will densify the soil bed in the post liquefaction stage [1]. The occurrence of soil densification when subjected to earthquake load also improves resistance to liquefaction in the subsequent earthquake. But still, mitigation of pore water pressure generation under repeated shaking will require attention to improve the soil resistance against liquefaction.

Stone columns or gravel drains proven to be an appropriate ground improvement technique to mitigate the effects of liquefaction [2] [3] [4] [5]. The granular pile acts as a porous medium in dissipating the excess pore pressure generated during an earthquake, thereby reducing the soil's susceptibility to liquefaction [6]. The effectiveness of these drains ultimately depends on the permeability of the material under consideration. Detailed experimental studies have been carried out on a fully saturated silty sand bed to view its response to liquefy under earthquake loads and evaluate gravel drains' effectiveness in controlling liquefaction [7]. However, studies on the effect of re-liquefaction resistance with stone column improvement are not available. The most suitable examples to reveal the impacts of re-liquefaction caused by the aftershocks of a seismic shaking are the Pacific Coast of Tohoku Earthquake in Japan (2011) [8]. There is a common idea among the researchers that densification of the soil stratum susceptible to liquefy will induce resistance against liquefaction in the subsequent shaking [9] [10] [11]. This paved a way to take up densification and drainage as a tool to control the effects of soil re-liquefaction [12] [13].

This study evaluated the stone column's performance in addressing the effects of liquefaction and re-liquefaction experimentally under repeated incremental acceleration loading. The experiments were carried out in a tank, placed over a uniaxial shaking table. The sand bed of 40% relative density was prepared and subjected to repeated acceleration loading with increments, leaving time interval to dissipate the pre-generated pore water pressure between each testing. A comparative analysis was conducted with stone columns installed ground and the virgin sand bed. The liquefaction assessment parameters such as pore water pressure, the dissipation rate of excess pore water pressure, foundation settlement, and soil displacement were monitored and analyzed.

The experimental studies revealed that in the case of untreated deposits, the potential to re-liquefy in the subsequent incremental accelerations might be due to the effect of non-uniform soil densification with depth which induces generation of pore water pressure in the subsequent loading. From the comparative analysis, it can be concluded that, installation of stone columns increased the rate of pore water pressure dissipation and improves the seismic response of liquefiable deposits, even at repeated shaking events.

2 Materials and Methodology

2.1 Soil used for sample preparation

The soil for the study was obtained from the Solani river bed in Roorkee and subjected to basic characterization, which revealed that the greyish brown-toned ground falls under the category of poorly graded sand with the Coefficient of uniformity (C_u) and Coefficient of gradation (C_c) as 2.634 and 1.147 respectively. The soil properties are listed in Table 1. The soil contained higher amounts of finer particles, as observed from the gradation data in Table.

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 Tank specifications

The laboratory scaled model tests were performed using a rigid Perspex rectangular tank, having a dimension of 1.4 m × 1.0 m × 1.0 m, mounted over a uniaxial shaking table, as shown in Figure 1. To minimize boundary effects during shaking, a 50 mm thick Polyethylene foam is attached on both sides of the container.

2.3 Sample preparation

The method of soil bed preparation plays a crucial role in representing the response of in-situ soil towards liquefaction [14] [15] [16] [17]. Depending on the method of soil packing, the initial stress condition to initiate liquefaction may broadly vary under defined stress cycles for a sample of the same gradation and density. Considering the above, wet sedimentation method was adopted in this study for preparing saturated ground deposit

For testing, a 600 mm thick, the saturated sand bed was prepared inside the tank. The quantity of sand and water required to attain a fill density of 40% was estimated and measured. To achieve uniformity throughout the tank's depth, the sample was filled gradually by dividing the fill volume into three equal parts of the required water and soil. The sand was drizzled down into the container at a pre-calculated height (IS 2720) through a conical hopper arrangement having an inverted solid cone with a 60° angle attached at the end, which resulted in achieving a uniformly spread soil bed [18].

Fig. 1. Perspex glass tank placed over shaking table

During sample preparation, glass tube piezometers and strain-based pore pressure transducers were used for monitoring pore pressure response. The instruments were placed centrally along the fill volume at 0.2 m and 0.4 m height from the tank's base to monitor the excess pore water pressure developed during seismic loading.

2.4 Design and installation of gravel drains

The stone columns were designed based on the guidelines given in IS 15284 Part 1 (2003) [19]. The design criteria and the properties of stone chips used in constructing the gravel drain are listed in Table 2.

Table 2. Design criteria and properties of stone columns

Sl. No.	Parameters	Description	Units
1	No. of stone columns	3	Nos
2	Area replacement ratio (ARR)	5	%
3	Pattern of installation	Triangular	No Units
4	Diameter of stone column	160	mm
5	Type of stones	Granite chips	No Units
6	Classification	Uniform and angular	No Units
7	Gradation	2 – 10	mm
8	Unit weight	16±0.2	kN/m^3
9	Relative Density	73	%

Initially, the arrangement of stone columns is marked on the ground bed appropriately with proper dimensions. Then a hollow PVC pipe of outer diameter equivalent to the stone column diameter was driven inside the prepared sand bed, and subsequently, the sand inside the PVC pipe was removed and replaced with stone aggregates compacted in three layers to achieve the required density. The entire assembly was then left undisturbed for 24 hours.

2.5 Scale down model of foundation

To monitor foundation response under dynamic shaking, a scaled down shallow footing model was designed and used following dynamic similitude laws as given in Equation 1 [20].

$$N_{EI} = N_k \times N_L^3 \tag{1}$$

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) equal to 10 was used to model shallow footing. The foundation was modeled for 115 mm length, 115 mm wide, and 30 mm thick using steel material having a modulus of elasticity 200 GPa. For all the test series, the foundation model was installed at 30 mm depth inside the prepared ground centrally for evaluating its settlement during seismic shaking.

2.6 Testing conditions

Based on the peak ground acceleration (PGA) of several severe earthquakes, an acceleration of 0.3g and 0.4g at 5Hz frequency simulating high and very high intensity shaking was selected and applied to the ground. The shaking was carried out for 200 cycles lasting for 40 seconds for both unreinforced and stone column reinforced beds. To assess the liquefiable sand bed's re-liquefaction characteristics, the incremental acceleration was given after 24 hours, allowing the complete dissipation of EPWP generated from the previous acceleration loading. The generated pore-pressure, soil displacement, and foundation settlement were estimated and compared using piezometers and strain-based transducers for both unreinforced bed and stone column bed.

3 Results and Discussion

3.1 Effect of pore pressure generation on liquefaction potential

The shaking table tests were carried out on the soil bed packed with 40 % relative density for both stone column (SC) reinforced soil bed and unreinforced bed to assess stone columns' efficiency in mitigating liquefaction. An incremental acceleration of 0.3g and 0.4g were given to the prepared sand beds. The generated excess pore pressure and the corresponding pore pressure ratio for each incremental loading is measured and compared for the unreinforced and stone column reinforced bed conditions. The initial bed conditions before shaking are shown in Figure 2 and Figure 3, respectively.

Fig. 2. Unreinforced bed – 40% RD – Initial condition

Fig. 3. SC reinforced bed – 40% RD – Initial condition

The prepared bed condition was left undisturbed for 24 hours before subjecting it to an acceleration load of 0.3g initially. The excess pore water pressure generated during the shaking is measured using glass tube piezometers and strain-based transducers to understand the variation of generated pore pressure along the depth of fill. The pore water pressure distribution for the initial 0.3g acceleration loading is shown in Figure 4 (a) and (b).

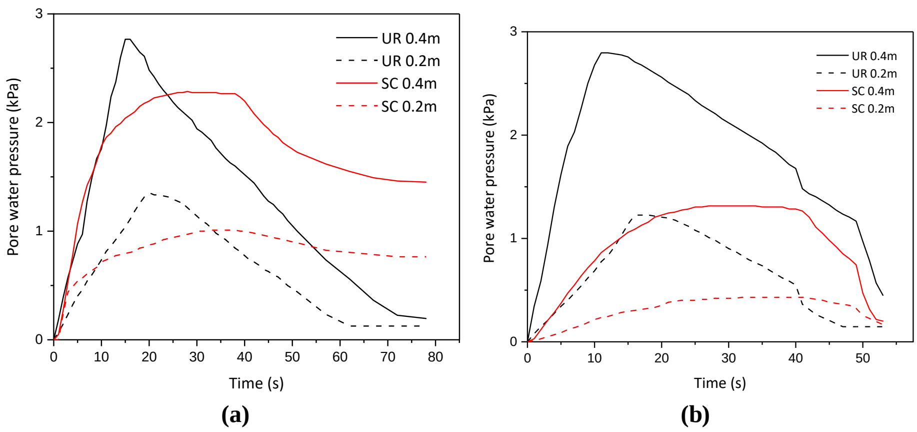

Fig. 4. (a) Porewater pressure generation – 0.4 m and 0.2m from top – 0.3g **(b)** Porewater pressure generation – 0.4 m and 0.2m from top – 0.4g

The generation of EPWP was observed by sand boils in the case of unreinforced condition, whereas in the case of stone column reinforced bed the column dissipates the generated pore water pressures. The generated pore water during shaking is shown in Figure 5 (a) and (b) respectively.

(a) **(b)**

Fig. 5. (a) Sand boils depicting generation of EPWP in unreinforced bed **(b)** Water seeping out initially from the gravel drains in SC reinforced bed.

Based on the results observed, it can be concluded that the generated excess pore water pressure measured by the transducers were found to be maximum in the bottom and minimum in the top for both unreinforced and reinforced bed condition. This effect may be due to the effect of overburden loading. Provision of gravel drains minimize generation of pore water pressure during shaking and dissipated generated excess pore water pressures mitigating occurrence of liquefaction. For the untreated soil, the top and bottom transducers recorded a peak pore pressure of 1.35 kPa and 2.76 kPa, which was 25.2% and 17.4% higher than the stone column reinforced bed. The time taken by the soil bed to liquefy, as recorded by the top and bottom transducers, were 20s and 15s, which was prolonged to 30s and 35s, respectively, in the case of stone column reinforced bed. This proves that stone columns have improved permeability characteristics that mitigate generation and improves dissipation rates of generated EPWP and delays occurrence of liquefaction. This observation was in line with earlier studies [21] [22] [23].

After letting enough time for the dissipation of EPWP generated during 0.3g acceleration loading, an incremental acceleration of 0.4g was applied to the same bed condition to assess the effect of re-liquefaction. The bottom transducers recorded higher EPWP, similar as in the case of 0.3g acceleration for the unreinforced condition (Figure 5 (b)). The peak pore water pressure in the case of unreinforced condition was 1.23 kPa and 2.79 kPa recorded in the top and bottom transducers, respectively, causing re-liquefaction of the soil bed. However, only 0.5 kPa and 1.31kPa excess pore pressure was observed in the case of a stone column reinforced bed. The reduction was about 65% and 53% for the top and bottom conditions. Also, the time to reach the peak pore pressure in the case of a stone column reinforced bed sensed by the bottom piezometer was found to be 27 seconds, which is almost 2.5 times greater than that seen in pristine bed condition.

When 0.3g and 0.4g are compared, there is a gradual increase in the generation of EPWP as the acceleration loading increases in the unreinforced condition. Contradicting this statement, in 0.4g, the bed reinforced with gravel drains reported a deficient generation of pore pressure than in the case of 0.3g. This may be due to the intensive rearrangement of particles, causing the soil bed to densify at higher rates expelling all the water in between the particles at the post liquefaction stage of 0.3g loading. This combined effects of densification and pore pressure generation after 0.3g resulted in a lesser generation of pore pressure in 0.4g acceleration loading. Hence it may be concluded that stone columns effectively mitigate liquefaction and re-liquefaction effects due to intense earthquakes.

3.2 Effect of pore pressure ratio (r_u)

The pore pressure ratio is the ratio between the pore water pressure (U) and the overburden pressure (σ_{vo}'), which can indicate a soil's susceptibility to liquefaction [24]. Figure 8 represents the pore pressure ratio variation with time for the unreinforced and SC reinforced bed condition for 0.3g and 0.4 g acceleration loading sensed by the bottom transducer. It can be seen that in both the loading conditions, the unreinforced bed liquefies, owing to a pore pressure ratio of about 0.66 and 0.67, respectively [23]. In the case of a stone column reinforced bed, under 0.3g, the pore pressure ratio of 0.57 was observed due to initial ground preparation. However, the installed stone columns delay the generation of peak pore pressure ratio and effectively dissipates generated EPWP. Under 0.4g loading, the peak pore pressure ratio further reduced to 0.31 and improves reliquefaction resistance of the prepared ground. The reduction may be due to the effects of soil densification due to successive acceleration and the higher dissipation rates of stone columns, as discussed in the earlier sections. This proves the stone columns' efficiency to mitigate liquefaction when subjected to the seismic load of higher peak ground accelerations (PGA).

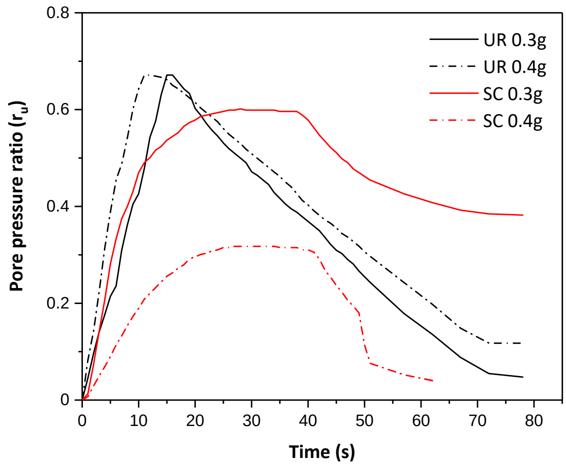

Fig. 8. Porewater pressure ratio – 0.3g and 0.4g – 40 cm from top

3.3 Soil displacement and Foundation settlement

Figure 9 represents the foundation settlement versus the incremental acceleration loading condition compared between unreinforced stone columns reinforced sand beds

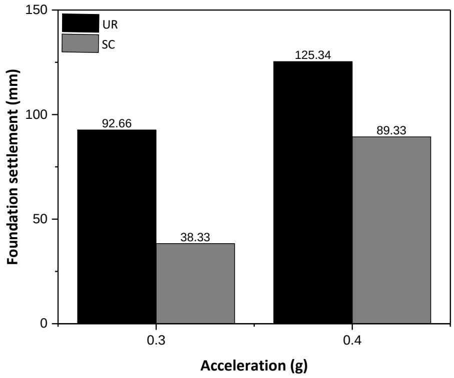

Fig. 9. Foundation Settlement

From the above graph, it is visible that when there is an increase in the acceleration loading intensity, the foundation settlement has also increased considerably. In the unreinforced condition, the maximum foundation settlement observed after giving 0.3g and 0.4g incremental accelerations were found to be 93 mm and 125 mm, which was reduced to 38 mm and 89 mm with a percentage reduction of about 59 % and 29% respectively in the case of stone column reinforced ground bed. This concluded that the gravel drains installation has considerably reduced the foundation settlement implying a higher bearing capacity of the stone column reinforced ground.

4 Conclusions

In this study, the efficiency of the stone column in mitigating liquefaction and re-liquefaction was experimentally evaluated. An incremental repeated acceleration loading of 0.3g and 0.4g was applied to a tank placed on a uni-axial shaking table filled with Solani River sand compacted at a relative density of 40%. The generation of excess pore water pressure was higher and faster in the bottom layers than the top when subjected to 0.3g and 0.4g loading, respectively, in the unreinforced condition. The foundation settlement proportionally increased as the successive acceleration loading increased, attaining a value of 93 mm and 125mm, respectively. To reinforce the liquefiable ground and minimize the EPWP generation and foundation settlement, stone columns with a 5% area replacement ratio were installed. At 0.3g acceleration loading, the peak pore pressure generation was reduced by 17.39% in the bottom layers with stone column improvement.

Further, in the successive 0.4g acceleration loading, the stone columns reinforced bed proved to be more effective similar to 0.3g load application due to the combined effect of densification and drainage characteristics. The pore pressure ratio reduction is 0.32, inferring that the bed did not liquefy at repeated shaking, thus proving gravel drains more efficient to mitigate liquefaction even at higher successive acceleration loading. It can be concluded that the stone column reinforced bed improves liquefaction resistance, delays generation of pore water pressure, and improve the safety of the foundation even at repeated shaking events.

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