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1-g physical modelling of shallow foundation treated with polypropylene-reinforced soil-cement columns in liquefiable soil

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A B S T R A C T

The fibre-reinforced soil-cement columns are used as remediation measure against earthquake induced soil-liquefaction associated large settlements in liquefiable loose surface layer of sand. This loose surface layer of sand was overlying on the non-liquefiable dense bottom layer of sand. 1-g physical modelling of shallow foundation was carried out using shaking table. There were four 1-g physical models constructed for testing and the two types of improvements were used such as adjacent and beneath the structure. First model was 1-g physical model constructed without improvement, and three models were constructed with the provision of polypropylene-reinforced soil-cement columns. In the first treated model the columns were installed in the wooden fixity plate and adjacent to the structure, the second treated model was improved with improvement installed upon the soil-cemented fixity plate and provided adjacent to the structure, and in the third treated model with improvement installed upon the wooden fixity plate and provided beneath the structure. The results obtained in case of untreated 1-g physical model concludes that the penetration of structure inside the soil and settlement of structure, both are up to unacceptable limits. The results of first treated 1-g physical model concludes that the penetration and settlement of structure both are unacceptable. The results of penetration and settlement of structure in the case of second treated 1-g physical model are achieved up-to acceptable limits. The most successful type of improvement against the liquefaction-induced penetration and settlement of structure was achieved in case of third treated 1-g physical model in which the values of settlement and penetration are negligible. It is concluded that the improvement installed upon wooden fixity plate and provided beneath the structure is relatively the most efficient remediation measure against the earthquake induced soil-liquefaction induced settlement of structure.

1. Introduction

The seismically induced soil-liquefaction occurs in the areas which are near the rivers or other water bodies. The earthquake induced soil-liquefaction results in ground deformations [1]. Soil-liquefaction occurs in saturated soil during earthquakes, which may lead towards the destruction of life and constructed structures. Soil Liquefaction take place when a saturated soil considerably loses its stiffness and strength in reaction to an earthquake shaking or may be due to some unexpected variation in stress condition in which a soil that is usually a solid acts alike liquid. Stress-strain behavior of saturated soil depends on its relative density. The shear stress problem may consistently occur for many times during seismic event. Such continuous process of fluctuations in effective stress and strength increase is known as cyclic freedom of movement. Flow distortion may occur when the loose layer of sand liquefies [2]. The reduction in ground displacements or complete mitigation can be achieved if the design of sub-structure is efficient against the inertia and gravity combined, and driving forces.

1.1 Mitigation Measure against Seismic Damages

Martin et al. [3] have studied that the ground improvement is mostly carried out at sites, where poor soil conditions are observed. Considering the Kocaeli seismic event 1999 whose magnitude was 7.4. Izmit Bay contains soft soil that can increase the damages potential due to earthquake, so, ground improvement technique is carried out before any construction. In Arango [4], earthquake damage analyses of previous 35 years shown that large economic losses took place in areas where soil liquefaction related lateral movements had occurred near the open trench or free boundary, i.e. river. Many latest designs of beneath ground barriers which had been constructed under severe seismic conditions and the types of beneath ground barriers were described; i.e. network of soil-cement, network of structural piles, clay fill.

1.2 Physical Modelling

According to A. J. Brennan and Madabhushi, [5] the main problem is the soil liquefaction potential at depth which can be studied through dynamic centrifuge modelling. It is explained that the present amenities at Cambridge has been enhanced through developing a latest dynamic basin which has the capability of modelling a deep soil strata, i.e. 40 m deep, under 100 g condition.

Four number of centrifuge model tests were conducted by Mitrani and Madabhushi [6], to investigate the performance of inclined non-structural micro-piles as soil-liquefaction mitigation measure for already constructed buildings. Two stratums with same constructed buildings had been tested under varying durations of earthquake and its magnitudes. The first soil profile was consisted of deep and homogenous strata of loose sand. The second soil profile comprised of dense sand underlain by shallow strata of loose sand. The modelling of superstructure was carried out in single degree of freedom. It was investigated that micro-piles had no any damaging effect on performance of structure after and during earthquakes. No decisive proof was achieved to display that micro-piles confine the lateral movement of soil because of monotonic shearing from structure or hinder the pore water pressures from free field to the zone of foundation. Mutually these processes had serious effects on settlement of structure. Rayamajhi et al. [7] studied the reinforcement behavior of soil-cemented columns in liquefiable soil through centrifuge testing. Two improved and two un-improved models were gone through seismic and sine sweep motions of different intensities in order to assess the settlement, lateral displacement, pore water pressure, and acceleration responses. It was observed that shear reinforcement behavior of columns had not been effective in decreasing the repeated stress ratios in treated soil; triggering of liquefaction took place almost at the similar time in both improved and un-improved cases and magnitude of resulting settlement of soil was not reduced significantly. The columns from their bases were immovable counter to rocking, columns had deformed initially in flexure and shear. The columns were intact and supporting the superimposing structures even after the triggering of soil liquefaction.

1.3 Liquefaction Associated Settlement of Shallow Foundations

Seismically induced liquefaction of soil creates a problem for structures constructed upon shallow foundations. One of the major problem is the excessive settlement of structure that occurs due to earthquake shaking. For insurance and design of structures it is hence very much important to evaluate the extent of settlements. The procedures are relating potential settlement to the size of foundation and depth of liquefiable soil strata. However, the effect of bearing capacity is an important aspect too [8]. Lu et al. [9] have studied that Liquefaction of soil may occur if stiffness and strength of soil reduces by fluctuations in stress condition, and structural settlement may occur after soil liquefaction followed by a seismic event.

1.4 Soil Liquefaction and Its Mitigation

The improvement of soft soil grounds was carried out through sand compaction pile method [10]. Sand compaction pile method is also used as a mitigation measure against soil liquefaction. Wijewickreme and Atukorala [11] have studied that soil liquefaction executes geotechnical hazards mainly as permanent ground distortions and loss of soil bearing capacity, so, the design of foundations should be carried out in such a way to mitigate from these hazards.

Soil stabilization through mixing is an eco-friendly and economic method that is progressively being carried out throughout the world for the improvement of soft soil. Carasca [12] has presented a good knowledge of mechanical and physical properties of improved soil and also its behavior to optimize the design of mixing procedure.

Grouting is another method used as a ground and soil improvement technique in which a flowable material is inserted inside the soil under pressure in order to change the behavior and characteristics of the soil. The common usages of grouting are such as construction of foundations and abutments. Grouting technique is an expensive and time consuming mitigation measure against post-development issues. After the application it is not flawlessly reliable even after great care is carried out. But, in many cases grouting technique is the sole technique used as a solution for the soil problems [13]. The less pressure infiltration grouting had been adopted in order to mitigate the damaged central core stratums of five old dams. The damages, such as wet zones, fluid clay cores, slope failures, and sinkholes upon the downstream seeming induced by extreme seepage [14]. In this study various mitigative measures against liquefaction were discussed and also new technologies presently under investigation. The deep mixing technique was used as a mitigation measure for existing structures against soil liquefaction [15].

Adalier et al. [16] have suggested that in most of the cases the densification may not be achieved through vibro-stone columns in silty non-plastic soils. It was previously suggested that the re-distribution of shear stress in order to analyze the stone columns as soil liquefaction mitigation measure particularly in silty non-plastic soils. The saturated stratum of silt and its response were investigated under the conditions of dynamic excitation at base by conducting the centrifuge testing. The impact of stone columns upon deformations and pore water pressure were studied. The results concluded that the stone columns were effective against soil liquefaction induced settlement of silty non-plastic soils particularly beneath the shallow foundations. Adalier and Elgamal [17] have suggested that the hazard of liquefaction and related ground distortions may be mitigated by using gravel drain (stone columns) technique.

Rollins and Oakes [18] state that while modest testing proposes that the vertical drains may be resulted as effective mitigation measure against liquefaction induced displacements and pore pressures, no complete-scale installation had been subjected to a seismic event. Due to the deficiency in performance, the complete-scale conditions were found as a weakness to growing use of this technique. To overcome such problem, complete-scale tests can be conducted through vertical drains in liquefiable loose sand layer using high speed system of actuator and laminar shear box.

Bahadori et al. [19] conducted a study based on analyzing the impact of gravel drainage and rubber columns on decrease of soil liquefaction potential of saturated sand using shaking table. Rubber columns was more effective technique as compared to gravel columns at high input motion and high density index for drainage purpose. Decrease in deformations due to soil liquefaction can be achieved by increasing the diameter and number of rubber and gravel columns. Soil liquefaction is very much destructive hazard that may cause destruction to constructed structures during a seismic event.

1.5 Ground Improvement Techniques

It is challenging to construct the heavy structures on soils with low density index. One of the soil stabilization techniques to counter this task is a provision of soil cement columns formed by the method called deep mixing [20]. For the purpose of depositing the dredged soft fill at dumping site consists of soft clay, soil-cement columns technique is commonly used in order to improve the soft ground [21]. 1-g laboratory model tests are conducted to investigate bearing capacity of the soil and soil-cement formed composite ground [22]. The other tactic is jet-grouting and it is practiced based on the requirements such as; economy, mechanical properties, pressure injections, and its application [23]. The unconfined compressive strength (UCS) tests were conducted in order to determine the effectiveness of twin-jet technique mainly dependent upon appropriate adequacy of critical parameters, such as; slurry of cement to water glass ratio, water to cement ratio, and cement content [24]. When structures are constructed upon cohesive soils having very low strength and impermeability, long term settlement and/or structural stability problems may occur. Dewatering is one of the technique through which long term settlement and lack of stability in structure can be prevented [25]. Multiple ground shaking measurements were practiced while dynamic compaction at industrial site. The shakings of ground which contains many shallow trenches were measured and there were many aspects of shaking which were investigated, such as amplitude attenuations, response spectrum, fourier spectrum, and waveform [26].

Brennan and Madabhushi [27] have presented one of the method to mitigate the seismically induced soil liquefaction is the provision of quick dissipation of pore water pressures by using the vertical drains through liquefiable soil. The centrifuge testing was carried out in order to clearly understand the drain behavior. The results concludes that the pore pressure develops from the centrifugally increasing soil zone which contributes to drainage through vertical drains. The geometry of this increasing zone varies with time. It was shown that the water from deeper soil mass drains first and reduction of efficacy of the vertical drain for adjacent soil stratum. It was concluded that such zones may be useful for analyzing further drain systems consists of more complicated geometries.

1.6 Polypropylene Fibre Inclusion

The study was carried out by Tang et al. [28], to assess the effects of discontinuous short polypropylene fibre on the mechanical performance of cemented and un-cemented clayey soil, and on the strength of clayey soil. Twelve groups of samples of soil had been prepared at 3% of polypropylene fibre contents, i.e. 0.25%, 0.15%, and 0.05% by weight of soil, and at two percentages of proportion of cement, i.e. 8% and 5% by weight of soil [29, 30]. Then direct shear tests and unconfined compressive strength tests were conducted after curing periods, i.e. at 28 days, at 14 days, and at 7 days. The results had shown that the provision of fibre reinforcement in cemented and un-cemented soil instigated a rise in the axial strain, shear strength, and UCS at the failure, it also reduced the loss of post-ultimate strength and stiffness, and changed the brittle behavior of cemented soil to more ductile.

Chen et al. [31] conducted a laboratory testing on strength performance of cement-clay admixture with provision of polypropylene fibre. Two kinds of fibres were used, first was monofilament fibre (polypropylene) and other was fibre packets fragmented from textile polymer bags. It was concluded that the fibre additive can considerably improve the ductility and strength of the Shanghai clay treated with cement. Both type of improvements in cement clay achieved their ultimate strength at 0.5% of fibre content. Unconfined compressive strength of the specimen may decrease while further increasing the fibre content. Nonetheless, polypropylene fibre performed better as compared to fibre packets, difference was <5%.

Correia [32] conducted a study based on investigating the effect of fibre and binder proportions upon the mechanical properties of soft soil ‘Baixo Mondego’, it was reinforced or non-reinforced with polypropylene fibres and chemically improved with binders. The experimental setup was consisting of four kinds of tests, first to determine the UCS and remaining three to determine tensile strength. Decrease in strength after the peak strength value and variation in behavior from brittle to ductile was investigated.

2. Materials and Methods

2.1 1-g Physical Modelling

The 1-g physical modelling of shallow foundation was carried-out by using manually operating shaking table. The shaking table is a machine that was used in order to apply the dynamic loading in 1-g physical modelling of shallow foundation treated with fibre-reinforced soil-cement columns. It was a manually operating shaking table as shown in Fig. 1. It was consisted of a soil model container with installed taps along the length for draining of water, pedals connected with a disc through a crank and disc was connected with displacing plate through a connecting rod. The diameter of disc was 0.3048 m (or 12 in), stand plate of soil container having dimensions, L x W x H = 0.9144 x 0.9144 x 0.0508 m^3 displacing with the help of wheels of diameter equal to 0.1016 m (or 4 in), fastened rails having dimensions, L x W x H = 0.9144 x 0.0254 x 0.0254 m^3, that enables the wheels to revolve by providing a reliable and smooth track for wheels to move upon, bottom stand plate for resting of entire load and mechanisms having dimensions, L x W x H = 1.524 x 0.9144 x 0.0508 m^3. Dimensions of soil model container considering energy absorbing boundaries; interior dimensions, L x W x H = 0.7112 x 0.7112 x 0.9398 m^3 and exterior dimensions, L x W x H = 0.8128 x 0.8128 x 0.9398 m^3.

There was a to and fro motion in 1-g shaking table while operating. For half motion the displacement was equal to 0.2667 m (or 10.5 in) and for one motion it was equal to 0.5334 m (or 21 in). The structure was weighed equal to 244.6522 N and dimensionally it was equal to; $L \times W \times H = 0.3048 \times 0.3048 \times 0.4572\ m^3$. The frequency and amplitude of dynamic loading were equal to 0.5 cycles per second and 0.0254 m (or 1 in), respectively. Therefore, half motion was achieved in 1 second and full motion was achieved in 2 seconds. The effects of acceleration, amplitude, displacement, and frequency were the actual milestones to be achieved after dynamic loading that lead towards the succession of the model. The shaking table test was carried out under 1-g motion and it was an experimental setup in order to simulate the behavior of constructed structure and also the freedom of response by applying dynamic loading, manually.

Fig. 1. Shaking table

The type of soil used in the model was sand which is a cohesion-less soil. The results of sieve analysis and relative density tests are shown in Table 1 and Table 2. There were four models which were constructed and tested through 1-g physical modelling of shallow foundation treated with fibre-reinforced soil-cement columns in liquefiable soil. The first 1-g physical model was untreated and remaining three 1-g physical models were performed using fibre-reinforced soil-cement columns and various fixity end conditions for columns. The capacity of number of columns in each fixity plate was 12. The shallow foundation upon liquefiable surface loose layer of sand underlain by non-liquefiable bottom layer of dense sand. The relative density of non-liquefiable bottom layer of dense sand was 90% and for liquefiable loose surface layer of sand was 10%, respectively.

The material required for casting of fibre-reinforced soil-cement columns was; river sand, polypropylene fibre, cement and water. The materials, e.g. fibre and cement, were weighed by dry weight of sand. The various proportions of fibre content were 0%, 0.25%, 0.50%, and 1%, whereas, for cement content proportions were 20%, 25%, and 30% for the casting of unconfined compressive strength test specimens. The diameter and length of the column specimen were 0.0508 m and 0.1016 m (or 2 in and 4 in), respectively. The water/cement ratio was taken as 1:1. The optimum proportions of materials such as: fibre and cement for the casting of fibre-reinforced soil-cement columns used in 1-g physical modelling were obtained through unconfined compressive strength test of fibre-reinforced soil-cement columns. The results of unconfined compressive strength test of columns are shown in Table 3 [29, 30].

Table 1

Results of sieve analysis

Sieve (U.S. Alternative)	Weight retained (lb)	% Retained	% Passing
No.4	0	0	100
No.10	0	0	100
No.40	0.000463	0.035	99.965
No.200	1.254	96.79	3.2034
Pan	0.005291	0.399	99.601

Table 2

Results of relative density test

Properties	Trial No. 1	Trial No. 2
Minimum Void Ratio ($e_{\min}$) gm/cc	0.617	0.628
Maximum Void Ratio ($e_{\max}$) gm/cc	1.00	0.97
Wet Density (γ_{wet}) gm/cc	1.51	1.51
Dry Density (γ_{dry}) gm/cc	1.00	0.99
Water Content (w)	15.40	16.30

2.2 Installation of Energy Absorbing Boundaries

The sheets of commercial foam were used as energy absorbing boundaries and installed in the model container through epoxy Fig. 2. These energy absorbing boundaries were installed in order to achieve field conditions.

Table 3

Results of unconfined compressive strength test

Cement content (%)	Fibre content (%)	Average stress (kPa)
20	0	2748.956
25	0	5206.981
30	0	7808.087
20	0.25	3249.738
25	0.25	6369.553
30	0.25	8795.344
20	0.5	4599.628
25	0.5	6683.179
30	0.5	9105.188
20	1	2485.819
25	1	4103.287
30	1	4286.660

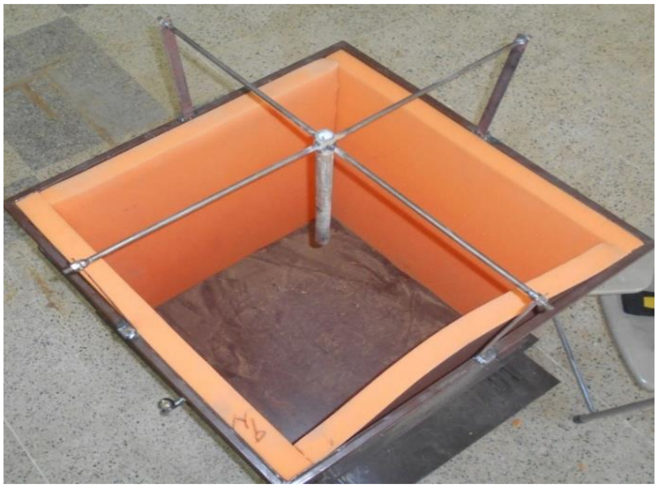

Fig. 2. Energy absorbing boundaries

2.3 Preparation of Untreated 1-g Physical Model

This was the 1-g physical model in which there was no any provision of mitigation measure against the soil-liquefaction associated ground settlement. Initially, the sand was poured in to the soil container and then compacted to achieve the relative density equal to 90%, so, that the non-liquefiable bottom layer of dense sand can be achieved. Then the liquefiable surface layer of loose sand was laid above the non-liquefiable bottom layer of dense sand and its relative density was equal to 10%. These two layers of sand were laid, each of 0.3048 m (or 12 in) depth. The structure was placed carefully above the liquefiable surface layer of loose sand and finally the equipment such as piezometers, strain gauge, and the cross bar assembly were installed Fig.3.

Fig. 3. Installed equipment

2.4 Preparation of Treated 1-g Physical Model

In case of 1-g physical treated models, initially, filling of sand up-to 0.127 m (or 5 in) was carried out in non-liquefiable bottom layer of dense sand. Then fixity plate was provided to achieve the fixity condition for fibre-reinforced soil-cement columns Fig. 4. In case of 'treated 1-g physical model-I with treatment adjacent to structure' and 'treated 1-g physical model-III with treatment beneath the structure', a wooden fixity plate of thickness equal to 0.0508 m (or 2 in) was provided and fibre-reinforced soil-cement columns were fixed at 0.0254 m (or 1 in) inside the fixity plate in order to attain the fixity condition. In the case of 'treated 1-g physical model-II with treatment adjacent to structure' a soil-cemented fixity plate of thickness equal to 0.127 m (or 5 in) was provided, the columns were fixed 0.1016 m (or 4 in) inside the fixity plate. The fixity plates were used in order to prevent the columns from rotation and displacement. After the provision of fibre-reinforced soil-cement columns vertically in liquefiable surface loose layer of sand underlain by the non-liquefiable bottom layer of dense sand, the filling of sand for remaining 0.4826 m (or 19 in) height was carried-out. So, the total filling of sand was up-to 0.6096 (or 24 in) comprising of 0.3048 m (or 12 in) non-liquefiable bottom layer of dense sand and 0.3048 m (or 12 in) liquefiable surface layer of loose sand. The structure was placed at the top of liquefiable surface layer of loose sand and finally the equipment's were installed.

Fig. 4. Fixity plates

2.5 Saturated Condition for the Soil

Water was poured inside the model container and the tap valves were kept open. This step was carried until the uniform flow of non-turbid water from the taps was achieved. In this way the soil was uniformly saturated. Then the water taps were closed in order to maintain that uniformly saturated or steady condition for the soil.

2.6 Soil-Liquefaction after Dynamic Loading

The shaking table was operated manually. The frequency and amplitude of shaking table were held in reserve as equal to half cycle per second and 0.0254 m (or 1 in), respectively. The dynamic loading was carried out for 60 seconds. Since the frequency was 0.5 cycles per second. Therefore, in total 30 cycles in 60 seconds were achieved. After the application of dynamic loading the soil liquefaction was occurred.

2.7 Recording of Readings

The penetration of structure inside soil and settlements were measured. The pore water pressure head was measured through piezometers. Then the water which mounted at the surface due to liquefaction of soil, was drained-out through tap valves.

2.8 Dismantling of the Model

The structure was unloaded after the removal of cross bar assembly containing strain gauge. Water tap valves were operated, so, that the water can be drained out. After draining out of water from the model container, piezometers were removed. The sand removal process was carried out in order to assess and examine the columns from top to bottom. The columns were carefully examined from top to bottom and then removed from the model container. Then, the remaining soil was excavated and removed from the soil model container.

3. Results and Discussion

3.1 Untreated 1-g Physical Model

The shallow foundation was constructed upon untreated liquefiable surface loose layer of sand which was underlain by the non-liquefiable bottom layer of dense sand. The shaking table was operated manually and model was examined. Measurements of penetration of structure inside soil and settlement from each side of the structure were taken after shaking Fig. 5. The results were very un-satisfactory, e.g. average settlement value of structure was 0.152 m (or 5.975 in) and average penetration of structure inside liquefiable soil was equal to 0.108 m (or 4.25 in). These large settlements and penetration of structure were carried while shaking of 1-g physical model due to a shear distortion of saturated liquefiable surface loose layer of sand and due to soil-liquefaction as well. After shaking of 1-g physical model, due to saturated soil stiffness and reduction of strength of soil, soil-liquefaction and associated penetration of structure inside soil and ground settlements were carried out. The results of penetration of structure inside soil, pore-pressure head and ground level or settlement in case of untreated 1-g physical model are shown in Tables 4, 5 and 6.

Fig. 5. Penetration of structure inside the ground after the application of dynamic loading

Table 4

Settlement of Ground from Sides of the Structure in Untreated 1-g Physical Model

Side	Dry sand (m)	Before dynamic loading (m), S_1	After dynamic loading (m), S_2
Left	0.184	0.211	0.366
Right	0.187	0.216	0.368
Rear	0.182	0.206	0.358
Front	0.182	0.208	0.356
Average settlement	0.183	0.210	0.362
		Settlement (S_2-S_1)	0.152 m

Table 5

Penetration of Structure inside the Ground in Untreated 1-g Physical Model

Side	Penetration (m)
Left	0.1016
Right	0.1143
Rear	0.1016
Front	0.1143
Average settlement (m)	0.10795

Table 6

Average Pore Pressure Heads in Untreated 1-g Physical Model

Activity	Average pore pressure head (m)
Soil saturation	0.279
Dynamic loading	0.368

3.2 Treated 1-g Physical Model-I with Treatment Adjacent to Structure

The shallow foundation was constructed upon untreated liquefiable surface loose layer of sand which was underlain by the non-liquefiable bottom layer of dense sand. It was improved with fibre-reinforced soil-cement columns adjacent to structure. The fibre-reinforced soil-cement columns from their ends were fixed partially in horizontal direction in a wooden fixity plate. After the application of dynamic loading through shaking table, the columns were gone through rotation Fig. 6. Results of level of ground and penetration were almost same as were in the case of untreated 1-g physical model. Due to the un-intactness, the fibre-reinforced soil-cement columns were resulted in a distortion at their bases. Most of the columns were went through horizontal displacement from top, and rotation and displacement from their base Fig. 7. In this treated 1-g physical model-I with treatment adjacent to structure, the penetration of structure inside soil was equal to 0.092 m (or 3.625 in), and the average settlement was equal to 0.117 m (or

4.60 in). After dismantling of structure it was observed that the fibre-reinforced soil-cement columns had been damaged during shaking table test Fig. 8. The results of penetration of structure inside soil, pore-pressure head and ground level or settlement in case of treated 1-g physical model-I with treatment adjacent to structure are shown in Tables 7, 8 and 9.

Table 7

Settlement of Ground from Sides of the Structure in Treated 1-g Physical Model-I with Treatment Adjacent to Structure

Side	Dry sand (m)	Before dynamic loading (m), (S_1)	After dynamic loading (m), (S_2)
Left	0.180	0.207	0.323
Right	0.187	0.211	0.323
Rear	0.179	0.207	0.323
Front	0.180	0.206	0.330
Average settlement	0.182	0.208	0.324
		Settlement (S_2-S_1)	0.117 m

Table 8

Penetration of Structure inside the Ground in Treated 1-g Physical Model-I with Treatment Adjacent to Structure

Side	Structure penetration inside the ground (m)
Left	0.095
Right	0.089
Rear	0.095
Front	0.089
Average settlement (m)	0.092

Table 9

Average Pore Pressure Heads in Treated 1-g Physical Model-I with Treatment Adjacent to Structure

Activity	Average pore pressure head (m)
Soil saturation	0.279
Dynamic loading	0.381

Fig. 6. Columns rotated after the application of dynamic loading

Fig. 7. Penetration of structure inside the ground after shaking

Fig. 8. Damaged Columns after the Application of Dynamic Loading

3.3 Treated 1-g Physical Model-II with Treatment Adjacent to Structure

The shallow foundation was constructed upon untreated liquefiable surface loose layer of sand which was underlain by the non-liquefiable bottom layer of dense sand. It was improved with fibre-reinforced soil-cement columns adjacent to structure. Fixity plate made of sand-cemented material was used in order to provide fixity for fibre-reinforced soil-cement columns. The columns were provided adjacent to structural foundation. The columns were intact after shaking Fig. 9. The columns did not rotated and displaced from its base due to the provision of soil-cemented fixity plate. This type of treatment was resulted to be an effective mitigation measure against soil-liquefaction associated settlement and penetration of structure inside ground Fig. 10. In this treated 1-g physical model-II with treatment adjacent to structure, the average value for penetration of structure inside soil was equal to 0.003 m (or 0.125 in), and the average value for settlement was equal to 0.029 m (or 1.156 in). This type of treatment was very much satisfactory. The columns were intact in non-liquefiable bottom layer of dense sand and resulted as an effective mitigation measure against soil-liquefaction associated settlement and penetration of structure inside ground. The fibre-reinforced soil-cement columns were un-damaged and intact at their basis due to the provision of sand-cemented fixity plate Fig. 11. This kind of treatment is effective for old (or existing) structures and also effective as retrofitting of existing structure. Recommendation: Some segment of a soil underneath the structure and also nearby the edge of foundation may reduce the settlements up-to tolerable limits and can further reinforce the ground as well. The results of penetration of structure inside soil, pore-pressure head and ground level or settlement in case of treated 1-g physical model-II with treatment adjacent to structure are shown in Tables 10, 11 and 12.

Fig. 9. Columns were intact after the application of dynamic loading

Fig. 10. Penetration of structure inside the soil was negligible after shaking

Fig. 11. Columns were un-damaged due to intactness

Table 10

Settlement of Ground from Sides of the Structure in Treated 1-g Physical Model-II with Treatment Adjacent to Structure

Side	Dry sand (m)	Before dynamic loading (m), (S_1)	After dynamic loading (m), (S_2)
Left	0.130	0.130	0.162
Right	0.130	0.130	0.158
Rear	0.133	0.133	0.158
Front	0.130	0.130	0.162
Average settlement	0.131	0.131	0.160
		Settlement (S_2-S_1)	0.029 m

Table 11

Penetration of Structure inside the Ground in Treated 1-g Physical Model-II with Treatment Adjacent to Structure

Side	Structure penetration inside the ground (m)
Left	0.003
Right	0.003
Rear	0.003
Front	0.003
Average settlement (m)	0.003

Table 12

Average Pore Pressure Heads in Treated 1-g Physical Model-II with Treatment Adjacent to Structure

Activity	Average pore pressure head (m)
Soil saturation	0.279
Dynamic loading	0.381

3.4 Treated 1-g Physical Model-III with Treatment beneath the Structure

The shallow foundation was constructed upon untreated liquefiable surface loose layer of sand which was underlain by the non-liquefiable bottom layer of dense sand. It was improved with fibre-reinforced soil-cement columns and were provided beneath the structure. This type of treatment has performed very well against the liquefaction related settlement and penetration of structure inside soil during shaking of ground Fig. 12. This treatment was resulted as the most successful kind of mitigation measure against soil-liquefaction associated penetration of structure inside soil and settlement Fig. 13. In this treated 1-g physical model-III with treatment beneath the structure, settlements were almost negligible with recorded average settlement values of 0.00079 m with no signs of penetration of structure. In spite of this, the rotation of fibre-reinforced soil-cement columns has occurred because of provision of wooden fixity plate. In this treated 1-g physical model, the columns had performed even superior. After dismantling of model the columns were examined and it was perceived that fibre-reinforced soil-cement columns were un-damage during shaking or dynamic loading Fig. 14. The results of penetration of structure inside soil, pore-pressure head and ground level or settlement in case of treated 1-g physical model-III with treatment beneath the structure are shown in Table 13, 14 and 15.

Table 13

Settlement of Ground from Sides of the Structure in Treated 1-g Physical Model-III with Treatment beneath the Structure

Side	Dry Sand (m)	Before dynamic loading (m), (S_1)	After dynamic loading (m), (S_2)
Left	0.152	0.152	0.152
Right	0.124	0.149	0.152
Rear	0.124	0.149	0.149
Front	0.124	0.149	0.149
Average settlement	0.125	0.150	0.151
		Settlement (S_2-S_1)	0.0008 m

Table 14

Penetration of Structure inside the Ground in Treated 1-g Physical Model-III with Treatment beneath the Structure

Side	Structure penetration inside the ground (m)
Left	0.000
Right	0.000
Rear	0.000
Front	0.000
Average settlement (m)	0.000

Table 15

Average Pore Pressure Heads in Treated 1-g Physical Model-II with Treatment Adjacent to Structure

Activity	Average pore pressure head (m)
Soil saturation	0.279
Dynamic loading	0.368

Fig. 12. Absolute intact columns after shaking

Fig. 13. Penetration of structure inside the soil was negligible after shaking

Fig. 14. Columns were un-damaged after shaking

3.5 Settlement vs Time

The expected average settlements and penetration values of all models with respect to time are shown below in Tables 16 and 17. The comparison between the average settlements and penetration of structure inside ground of all the model tests are shown in Figs. 15 and 16.

Table 16

Comparison between average settlements with respect to time of 1-g physical model tests

Time (s)	Average settlement (mm) Untreated model	Treated model-I	Treated model-II	Treated model-III
5	50.80	45.72	12.70	0.762
10	101.60	76.20	22.86	0.79375
15	127.00	101.60	25.40	0.79375
20	139.70	104.14	26.67	0.79375
25	142.24	108.68	27.94	0.79375
30	144.78	112.22	28.956	0.79375
35	147.32	114.30	29.1846	0.79375
40	149.86	116.84	29.36875	0.79375
45	151.13	116.84	29.36875	0.79375
50	151.511	116.84	29.36875	0.79375
55	151.638	116.84	29.36875	0.79375
60	151.765	116.84	29.36875	0.79375

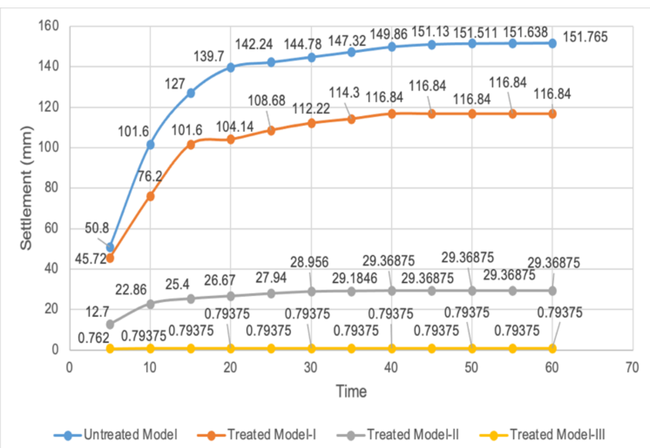

Fig. 15. Settlement vs time curves for untreated and treated models

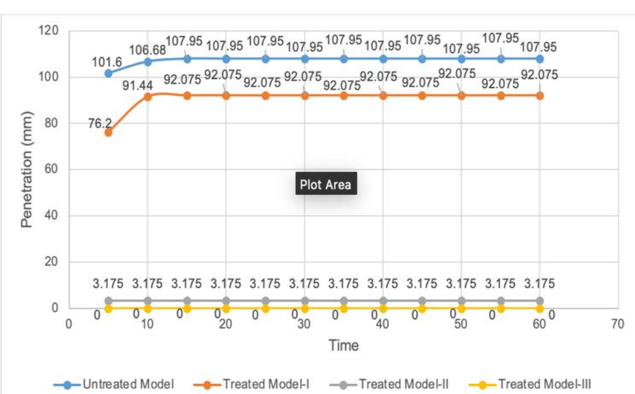

Fig. 16. Penetration vs time curves for untreated and treated models

Table 17

Comparison between penetrations of structure inside ground with respect to time of 1-g physical model tests

Time (s)	Penetration inside ground (mm) Untreated model	Treated model-I	Treated model-II	Treated model-III
5	101.6	76.2	3.175	0
10	106.68	91.44	3.175	0
15	107.95	92.075	3.175	0
20	107.95	92.075	3.175	0
25	107.95	92.075	3.175	0
30	107.95	92.075	3.175	0
35	107.95	92.075	3.175	0
40	107.95	92.075	3.175	0
45	107.95	92.075	3.175	0
50	107.95	92.075	3.175	0
55	107.95	92.075	3.175	0
60	107.95	92.075	3.175	0

4. Conclusion

The following are the conclusions being drawn on the basis of final results.

1. The penetration of structure inside the soil is equal to 0.108 m (or 4.25 in) and settlement was occurred for a large depth, i.e. 0.152 m (or 5.975 in), in case of un-treated 1-g physical model because of un-improved ground.
2. After the application of shaking in case of treated 1-g physical model-I with treatment adjacent to structure, penetration of the structure inside the ground and settlement are measured up to the tolerable limits, i.e. 0.092 m (or 3.625 in) and 0.117 m (or 4.609 in), respectively.
3. In case of treated 1-g physical model-II with treatment adjacent to structure, the soil-liquefaction induced penetration of structure inside ground and structural settlement after shaking were equal to, i.e. 0.003175 m (or 0.125 in) and 0.160 m (or 6.3125 in), respectively, and these were relatively less as compared to the penetration and settlement values of treated 1-g physical model-I with treatment adjacent to structure.
4. The retrofitting and dislocation of columns was not occurred due to the provision of cemented-sand fixity plate and columns were found as intact after shaking in case of treated 1-g physical model-II with treatment adjacent to structure.
5. The results of treated 1-g physical model-III with treatment beneath the structure are achieved as very

much effective in order to mitigate the soil-liquefaction induced penetration of structure inside soil and settlement of ground. In this treated model, the penetration of structure inside soil and settlement were negligible, i.e. 0 m and 0.00079 m, respectively.

6. The mitigation of penetration of structure inside soil and settlement of ground in the case of treated 1-g physical model-III with treatment beneath the structure was carried out efficiently. Therefore it is concluded that as compared to other treated 1-g physical models, the treated 1-g physical model-III with treatment beneath the structure was comparatively more efficient against earthquake induced soil-liquefaction associated penetration and settlement.
7. It is concluded that, as the settlement of structure and ground takes place simultaneously, so, the settlement is not that much damaging and hazardous, but, the penetration of structure inside the ground is a huge problem.
8. The treatment provided beneath the structure is capable of preventing the foundation of structure from liquefaction related settlement and penetration of structure inside soil after shaking.

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