SEISMIC SSI OF ROCKING SHALLOW FOUNDATIONS ON REINFORCED SOIL

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Abstract

During an earthquake event, structures on soft and loose soil pose more complicated seismic behavior compared to similar structures on rock or stiff soil. This is due to various reasons including ground motion amplification in soft soil, kinematic interaction, large deformations at foundation level influencing the eigen value and damping properties of the structure, material and geometric damping of the soil and so on. Several past studies have indicated that the deformations at the foundation base, particularly the rocking of shallow foundation and subsequent energy dissipation may pose beneficial effect on the structure through reducing floor acceleration, column moment, and ductility demands of the structural members. However, adverse consequences, such as excessive permanent and transient settlement, sliding and tilting of the foundations are also associated with rocking shallow foundations. In this background, the present study aims to investigate the utility of geosynthetics as potential improvement of the subsoil to reduce the earthquake-induced settlement of structures.

Geosynthetics such as geogrid and geotextile are cost-effective solutions for ground improvement and are widely used nowadays effectively in several geotechnical application including slope stabilization, reinforcing retaining wall backfills, railway subgrade improvement etc. However, utility of these materials in reduction of seismic settlement and tilting of low-to-medium rise urban buildings have not been explored much. In particular, there is still need to understand the geogrid-sand and geogrid-clay interface behavior during seismic event, the strength and stiffness increase capability of the material, performance of the material at large intensity shaking, strength degradation with time, and proper modeling technique to simulate the behavior of the material.

In the present study, a numerical model of a 3-story steel moment resisting frame structure is developed. The seismic response of the structure on loose Ganga sand with and without geosynthetics layers are examined. The analysis is done in the finite element framework of OpenSees, where a two-dimensional plane-strain approach is adopted. 4-Noded quad elements with nonlinear material model '*PressureDependMultiYield*' are used to model the Ganga sand. To incorporate the soil-structure interaction effects, different footing widths and boundary conditions (compliant and rigid base) are considered. Investigation on the effects of different parameters such as material model, modeling approach, depth of top geosynthetics layer, layer thickness, length of geosynthetics and its interactions with soil provides an insight into seismic design and analysis. The results of the numerical study are compared and validated with a series of shake table experiments conducted on a reduced-scale model of the building at Indian Institute of Technology Kanpur.

Keywords: Soil-structure interaction; Geosynthetics; Continuum Modeling; Dynamic Analysis

1. Introduction

Considerable research attention has been paid to understand the seismic soil structure interaction behaviour on shallow foundation. Amplification in loose and soft soils, kinematic soil structure interaction (SSI), foundation displacement and energy dissipation through the structure and soil, effects the seismic SSI behaviour.Soil is a complex material with nonlinearity; plasticity; and has the ability to change with different loading conditions, surcharge loads, boundaries, water table height, and stresses. Most of the structures, when placed on soft and loose soils, become unstable and can pose a life-threatening problem. Loose soil such as fine sand does not have cohesion and plasticity to hold its particle and are often settles and slides a large amount when subjected to dynamic loads. Settlement caused in sands are irreversible and leads to differential settlement of footings which results in structural failure. Low to medium-rise buildings on these soils experience excess structural drift or permanent settlement of the foundation. Past studies suggest that these problems can have a detrimental effect on soil and structure. In large seismic events, soil nonlinearity plays a major role in mobilization of the bearing capacity of such soils and the corresponding damping characteristics at the soil-structure interface.

Design codes for geotechnical engineering have now been transforming from the limit equilibrium approach to performance-based design procedures. Geo-synthetics such as geogrid and geotextile used for reinforcement have now become a cost and time effective solutions for a number of ground improvement project. Very few studies have been performed on the shallow foundation lying on geosynthetics reinforced loose sand (GRLS). So, there is a need to conduct experiments and develop numerical models for studying the settlement behaviour of reinforced loose soils and its various aspects under seismic loading scenarios. Various researchers use Winkler-based or macro-element modeling approach to model the problem in a simpler way. However, the drawback of the approach is the idealization of the soil continuum with a uniaxial material model and discrete soil reactions that are uncoupled. Case histories of various foundation problems show that stress imposed by the structure is the main cause of settlements in footings that are not evaluated using spring model approach. So, there is a necessity of accurate and reliable constitutive modelling for these pressure-sensitive soils.

Fukuda et. al (1987) [1] carried out experimental tests on square footing. Reinforcement material used is polymer grid. Study has been done for u/B ratio and h/B to find the optimal depth of first layer of reinforcement and optimal spacing. Omar et al (1992) [2] did lab tests on square and strip footing on sand of relative density 70%. Reinforcement used here is geogrid BX-100 Tensar. u/B has been found optimum for ratio of 1. Yetimoglu et al. (1994) [3] took rectangular footing with uniaxial geogrid layed at 0.3B depth for one layer and 0.2-0.4 times width for multilayer. The effective zone has been found upto depth of 0.25B. Adams and Collin (1997) [4] made lab tests on square footing on 60,70 and 88% relative density of sand. Stiff biaxial geogrid has been used at 0.25-1.5B. Seismic slope behavior was studied by Lin and Wang (2006) [5] in shake table using uniform medium sand in a plexiglas sided box. They found substantial soil amplification when soil nonlinear behaviour was observed and slope failure surface was circular. Yang et al. (2013) [6] carried out a series of dynamic centrifuge tests up to 50g. Input ground acceleration and frequency effects were observed on slopes. They found out that soil amplification factor increases non-uniformly throughout the height of the geotextile reinforced slope and depends on the frequency of the excitations. A summary of optimum parameters used in different studies is tabulated in Table 1.

The present study focuses on studying the soil-structure interaction behavior of shallow foundations resting on geosynthetic-reinforced sand deposit. The study comprises of a series of shake table testing coupled with nonlinear finite element analysis.

Table 1 Optimum parameters for geogrid reinforced soil

Authors	Type of Study	Shape of Footing	Reinforcement Used	N	b/B	u/B	h/B	d/B
Binquet and Lee (1975a)[7]& Guido et al. (1985) [8]	Experimental	strip	geogrid	2-4	2-3	<0.67		1.25
Yetimoglu et al. (1994) [3]	Experimental	rectangular	uniaxial geogrid		4.5	0.25	0.2	1.5B
Adams & Collin (1997) [4]	Experimental	square	stiff biaxial geogrid	3		0.5	0.25-1.5	
Mosallanezhad et al. (2007) [9]	Lab tests	square	Grid-Anchor	4	4.5	0.25	0.25	
Chung &Cascante (2007) [10]	Experimental & numerical	square	fibreglass and aluminium meshes		3.5	0.3	0.3	1
Latha&Somwanshi 2009 [11]	Experimental & numerical	square	uniaxial and biaxial geogrid		4			2
Keskin and Laman (2012) [12]	Experimental	strip	geogrid	1-4	1	0.25-1.5	0.25-.1.5	
Shadmand et al. (2017) [13]	Experimental	Square	Geocell	1-2	5	0.1-0.3	0.1	
Arab et al. (2017) [14]	Review & Numerical	square	Geogrid	1-4	4	0.5	0.5	
Present study	Experimental &numerical	Square	Geogrid	3	3	0.2	0.3	

2. Experimental setup

A series of experiments is performed on the shallow footing resting on reinforced sand bed to understand the effect of geosynthetic layers on settlement reduction and bearing capacity enhancement. Followed is a detailed description of the soil bed preparation, footing model and geogrid selection.

2.1 Soil

Table 2. shows the properties of Ganga sand used in the test and numerical simulations. Through the well-established rainfall pouring method, the sand is poured inside the container by hand hopper to obtain the desired relative density. Loose Ganga sand is calibrated with the height of fall method in a rigid box. The box was filled with sand and weighed to get the corresponding relative densities. It was determined, for a height of fall of 15 cm, the relative density of 40% is achieved and 15cm height of fall is used for the present study. Jornada dynamic penetrometer (Herrick and Jones (2002)[23)] has been used to check the uniformity of the sand bed for the relative density.

2.2 Structure

The square footing of 1m x1m is modelled for each column of a SMRF office building. The prototype building is in Zone III and is designed as per IS800:1998[15] and as per IS 1893:2000[16]. The footing of size 2cm x 2cm with mass of 105 gm is designed using the similitude laws on the 3-story prototype footing

(Vivek and Raychowdhury, 2016 [17]). The footing is scaledtoa prototype to model ratio of 50. The model footingsimulates the isolated footing of the 3-story structure with a scaled mass, load and dimension considering similitude laws by Iai (1989)[18].

2.3 Geogrid

Geogrid used for the test is StrataGridTM BX60. It is a soil-reinforced polyester (PET) geogrid. Such geogrids are built with a coating of high molecular weight and high tenacity knitted polyester fabrics. Geogrid is having an aperture size of 8mm x 10mm and the tensile strength of 64 kN/m in both the machine and cross machine direction. Following the literature search and review (Table 1), the depth of top layer to footing width (u/B) was kept to be 0.2, other layer to width ratio is kept to be 0.3, geogrid length to width ratio is kept to be 3. The rubber membrane is marked at different height for the placement of geogrid during the filling of sand.

2.4 Instrumentation Details

A small laminar box consisting of five flexible lamina of size 30 cm x 30 cm x 27 mm rests on six roller bearing (three on each side) guided through channels are mounted on the top of the shaker (Figure 1). The shaker is connected with a power amplifier and input waveform generator (Figure 2) which provides the input sinusoidal acceleration. One accelerometer is attached to each of the five lamina and one at the base for the input acceleration. The rubber membrane is made in the form of the cube shape which fits into the laminar box and is attached to the wooden base with araldite adhesive. Figure 2 describes the step by step process of conducting the shaker test. The laminar box with shaker are placed at a label position and checked for horizontal levelling of the setup. The laminar box is lined with rubber membrane which prevents the soil to spill off from the gaps between lamina. The silicon rubber membrane (0.3 mm thick) is glued to each other sides using synthetic rubber glue and is prepared such that it fit in the flexible shear box. The base of the rubber membrane is fixed with araldite such that it does not peel off or get other noise during the process of testing. The accelerometers with data acquisition system are installed at the setup location. The input acceleration is provided through the power amplifier. A total of three dynamic experiments (unreinforced, singly reinforced and doubly reinforced) are carried out for each case. The accelerometers A1-A5 are located at the outside of the lamina box set-up are used to record the acceleration time history responses of the Ganga sand along the depth.

Table 2 Soil Properties

Soil Properties	Ganga Sand	
Specific Gravity	Gs	2.67
% Soil	% Sand	98.19
	% Fines	1.81
	D_{10}	0.15 mm
Effective particle size.	D_{30}	0.195 mm
	D_{60}	0.25 mm
Coefficient of uniformity	Cu	1.667
Coefficient of curvature	Cc	1.014
Maximum void ratio	e_{max}	0.99
Minimum void ratio	e_{min}	0.70
Unit weight	ϒ	14.07 kN/m^3
Friction Angle	Φ	32.5^o
Shear modulus	G_{soil}	8.18 MPa
Poisson's ratio	N	0.2

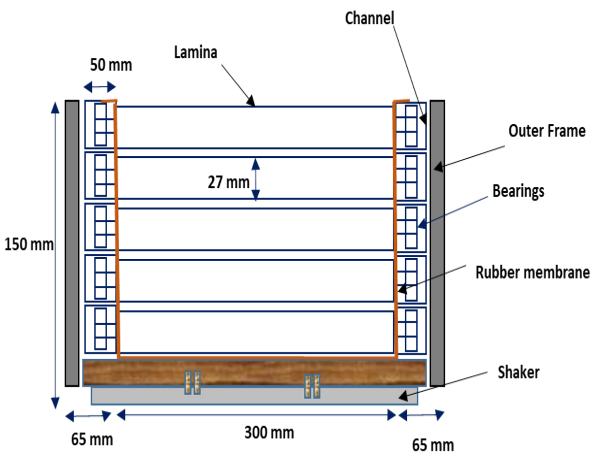

Figure 1 Schematic diagram of the laminar box used in the experiments.

2.5 Input Loading details

The motion applied to the soil foundation system is in the form of harmonic excitations and the amplitude is provided through a sensitive DC coupled power amplifier to the aps electrodynamics shaker. The ground motion selected is of frequency range 2-5 Hz and 0.05g to 0.2g (Table 3). Five ground motions are applied and the tests were repeated for each ground motion for three of the cases. So, A total of 15 tests are conducted to find the settlement enhancement and soil amplification factor at different depth for unreinforced (UR), singly reinforced (SR) and doubly reinforced (DR) soil. The excitations which were used for the shaker test are having the frequency and peak acceleration amplitude of 2 Hz 0.05g, 2 Hz 0.1g, 5Hz 0.05g, 5Hz 0.1g and 5Hz 0.2g. Figure 3 shows the typical applied input motion and corresponding top lamina acceleration time history of 5Hz 0.2g for all the 3 cases. For the data processing, the acceleration time recordings are filtered using a 4th order low pass Butterworth filter with a cut off frequency of 0.05 Hz- 20 Hz.

Figure 2 (a) Test setup with accelerometers and rubber membrane (b) rainfall pouring technique with geogrid maintaining the height of fall for a relative density (c) Settlement measuring dial gauge with holder and magnet on the frame (d) The aluminium light weight strip to hold the dial gauge while performing the test (e) DAQ system, Power amplifier and waveform generator to provide input frequency and acceleration amplitude into the shaker.

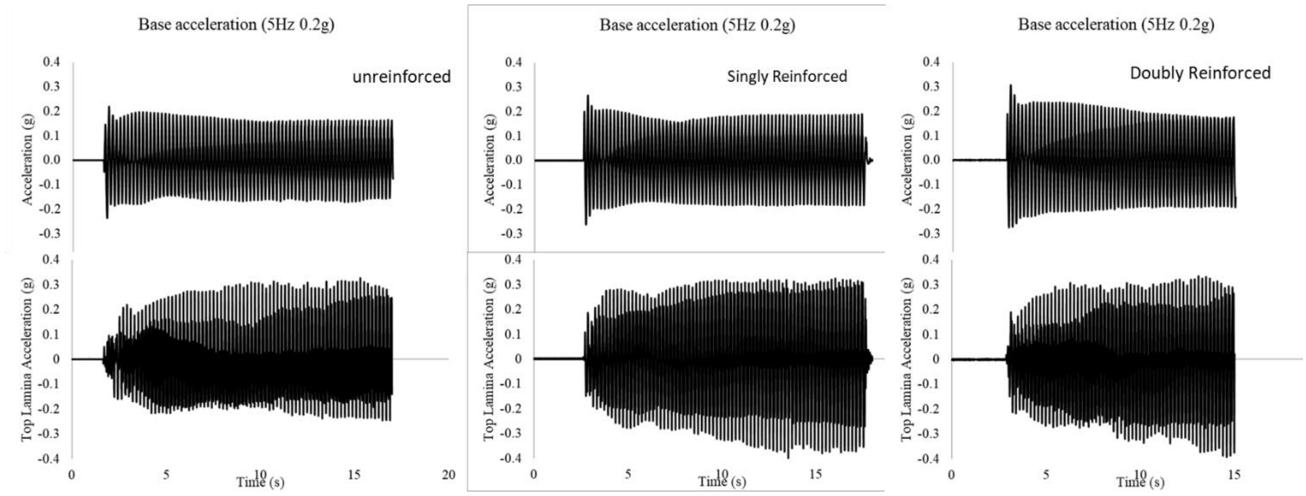

Figure 3 Input motion and the op lamina acceleration for the 5Hz 0.2g excitation for Unreinforced (UR) Singly Reinforced (SR) and Doubly reinforced (DR) tests

3. Experimental Results

3.1 Vertical Settlement

The energy dissipative capability of the dynamically loaded foundation results from the mobilization of capacity through the deformations of the foundation. This is done through the mechanism of foundation sliding, rocking and settlement. The vertical settlement of the footing is measured through a dial gauge which was mounted on the frame by magnet. The dial gauge is supported by a hinge-slide mechanism, developed by a lightweight aluminum strip which hold its position during the cyclic tests to permit free settlement, sliding and rotation to the footing. The settlement of the footing was 2.64 mm for the unreinforced case, 1.84mm for singly reinforced and 1.49 mm for doubly reinforced for the minimum excitation intensity motion of 2Hz 0.05g (Figure 4).

Settlement of the footing increases with the excitation frequency and the peak acceleration amplitude. The total settlement computed after each test depends on the number of loading cycles while the settlement rate was not influenced by the no. of cycles. The isolated footing behaves like a SDOF system, where the uplift of footing and foundation settlement mobilizes the system rocking so there are some variations in the results of 2Hz 0.1g, and 5 Hz 0.1g. In these cases, the settlement and uplift of the footing was observed simultaneously during the experiments.

The foundation with singly and doubly reinforced sand bed shows significant reduction in settlement and increase of bearing capacity. The footings which rest on the unreinforced sand fails or topples in high intensity motions while geogrid interface with singly and doubly reinforced sand allows the foundation to stand still with less vertical settlement. The maximum settlement observed was 10.45 mm, 7.45 mm and 8.11 mm for the UR, SR, DR case. The maximum % change in the singly reinforced is observed to be 51% for the 5Hz 0.05g excitation and 33.18% in doubly reinforced in the 5Hz 0.2g.

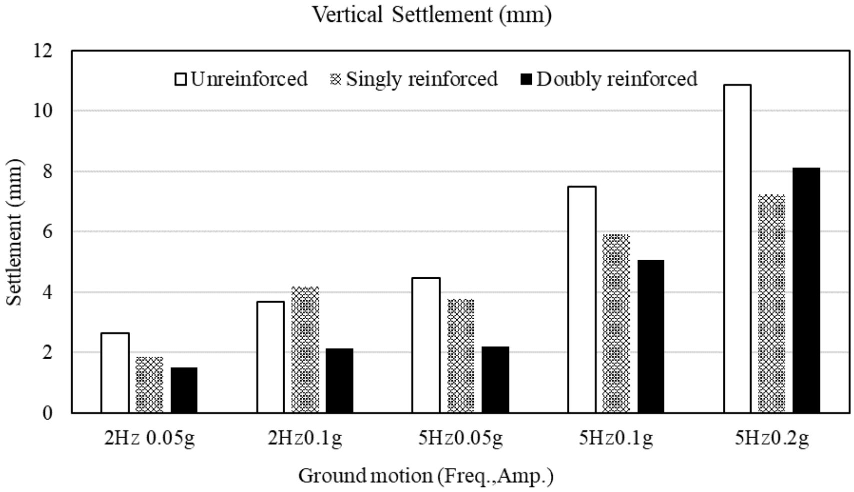

Figure 4 Vertical settlement for the sinusoidal motions for unreinforced, singly reinforced and doubly reinforced case.

3.2 Amplification factor

The wave motion amplification factor at any depth z of soil deposit on the laminar shear box can be expressed as:

$$A_F(z) = \frac{(\ddot{u}_{soil}(z))_{\max}}{(\ddot{u}_{input}(z))_{\max}} \tag{1}$$

where $A_F(z)$ is the amplification factor, $\ddot{u}_{soil}(z))_{\max}$ and $\ddot{u}_{input}(z))_{\max}$ are the soil and input wave accelerations at any depth z respectively. Soil amplification is affected by the fluctuations in the input acceleration. Soil amplification factor of the geogrid reinforced soil domain depends on non-uniform distribution and also on the amplification and attenuation responses which increase with the height. Amplification for the top lamina is less as the soil settles from the top to the second lamina. Noteworthy amplification was observed at low exciting energy and as the frequency and amplitude is increased, the soil of the top lamina settles quickly and leads to de-amplification. (Figure 5). Also, amplification get reduced as the test is repeated for the same fill of sand.

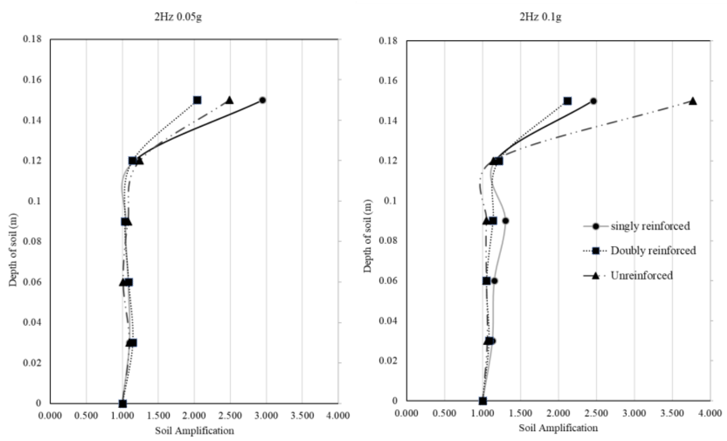

Figure 5 Amplification factor with depth corresponding to ground motions for UR, SR and DR case.

4. Numerical Model

A numerical model with continuous quad element is used for modelling loose sand with PressureDependMultiYield' (PDMY) ND-Material, proposed by Yang et al. (2003)[21], that characterizes the stress-strain relationship at gauss-points. "PDMY" is an elastic-plastic material that simulates the properties and responses of pressure-sensitive sand material under normal loading conditions. It includes dilatancy and non-flow liquefaction effects during monotonous or cyclic loading. The mesh element size was kept to be 0.5 in x and 0.1 in y direction, due to limitation of geogrid depth of top layer ratio u/B and h/B ratio of 0.2 and 0.3 the length of geogrid was in a ratio of b/B of 3. The footing 1 m x 1m x 0.1 m is modelled with elasticbeamcolumn element. The footing material is mild steel with young's modulus of 2e11 and poison's ratio of 0.3. Footing nodes are connected to soil nodes using equalDOF in both x and y direction. A self-generated generic code is written to define the mesh geometry; geosynthetics its layer; interfaces between the structure and soil and interaction between soil and geosynthetics by using interfaces like direct bonding using equalDOF, and using interfaces like zerolengthcontact2d (node to node contact) element (node to surface or node to node contact) based on the Mohr-Coulomb frictional law

$$T = C + N \tan(\phi) \quad (1)$$

where K_n and K_t are stiffness on normal and tangential direction and phi is the angle of failure between interfaces. Contact element are based on Kuhn tucker equation of gap and hook elements. K_n and K_t need not to be too high to induce numerical instability into the problem and has to be sufficiently large enough to allow tolerable penetration of nodes into each other. Geosyntheticshas been modelled as linear 2d truss element. The geosynthetics nodes are connected with zero lengthcontactelements with the soil with K_n =50.2MPa/m K_t =5.1MPa/m. Table 4 below shows the eigen value analysis for the unreinforced and reinforced case of the model.

Staged analysis procedures have been performed for static and transient analysis in OpenSees. The steps for the analysis are as follows. Firstly, the soil mesh is generated with all the parameters that match with the loose sand at field conditions, like soil profile height, the weight of soil by soil density, element size, soil domain length. Constitutive models like the one which is used for analysis require parameters like unit weight shear modulus, Poisson's ratio, friction angle, phase transformation angle, peak shear strain, dilation constants, and liquefaction constant.

Secondly, the boundary condition for lateral and base boundaries is specified: - standard fixities have been provided for defining the boundaries i.e., the base has been fixed in both x and y direction, and side is fixed for only in the x-direction to provide vertical displacement during the elastic geostatic stage. This is done to generate the same initial state of stress as the soil was subjected to when no structure was present. Eigenvalues analysis is performed to know the time period associated with the soil.

Thirdly, the soil is set to first run in the elastic stage, and then the material is updated to plastic to know whether it satisfies elastoplastic equilibrium. Once the equilibrium is attained, the structure with its footing and geosynthetics layers is added to the soil surface. Gravity loads are applied to the structure in an increasing linear pattern. The model is analyzed to satisfy the static equilibrium.

For the seismic analysis step, the horizontal boundaries fixities are removed and are replaced by the corresponding reactions that we get from the static equilibrium stage. Boundary conditions which prevail during the different stage are: -

For Continuum reflective base (CRB) boundary (Figure6 (b)): - Side boundaries are fixed in both the direction and base boundaries are allowed to move in ground shaking direction i.e., in the x-direction, the fixities are removed. Reflective base boundaries provide the best results for input motion as displacement time history as compared to velocity and acceleration time history.

For Continuum Viscous base (CVB) (Figure 6 (c)): - Fixities at side boundaries are removed in the x-direction and are replaced corresponding by viscous dashpots that represent the soil free field. The vertical fixities are provided in the y-direction to simulate the laminar box conditions. Base boundary fixities are removed in the horizontal direction and are replaced corresponding by viscous dashpots that represent the elastic rock. As dashpots are involved for simulating the absorbent boundary condition, the force is calculated using the equation

$$f(t) = C_d Velocity(t)$$

Where C_d is the dashpot coefficient, f(t) is the force-time history, and Velocity(t) is the velocity-time history. So, we require a velocity-time history profile for the viscous compliant base analysis.

Table 3 Details for the experiments

Test No.	Reinforcement conditions	Frequency (Hz)	Peak Acceleration, Amax (g)
HM1	•Unreinforced • Singly Reinforced • Doubly Reinforced	2	0.05
HM2		2	0.1
HM3		5	0.05
HM4		5	0.1
HM5		5	0.2

Table 4 Eigen Value Analysis

	Modes	Time Period Tn (secs)	Frequency (Hz)
Unreinforced	1	0.289	3.464
	2	0.288	3.468
Geogrid Reinforced	1	0.324	3.082
	2	0.317	3.152

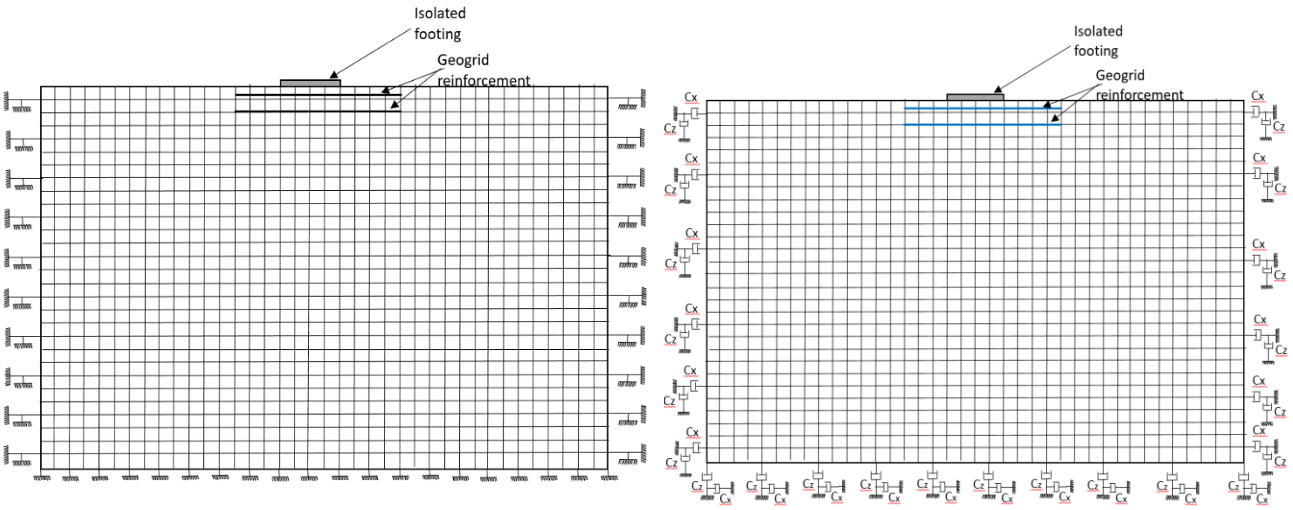

Figure 6 (a) Continuum Rigid Boundary (CRB) model (N=2) (b) Continuum Viscous Boundary (CRB) model (N=2)

5. Comparison between experiment and analysis

Figure 7 and 8 show the behavior of the footing on reinforced bed predicted from two different numerical models as mentioned earlier as well as from the experiments. Figure 7 shows the peak settlement of footing due to peak acceleration amplitude (g) of frequency 5 Hz for the unreinforced, singly and doubly reinforced case. It is also observed that, with the rise in the excitation frequency and amplitude the settlement rate increases manifold. The bearing capacity is substantially mobilized with high soil plasticization. The double-layer geogrid in loose sand is able to control the settlement for dynamic excitations to a minimum of 21%. For the prototype model in OpenSees, the same trend is observed as in the experiments for singly and doubly reinforced case; the maximum settlement is observed to be 16.43 mm, 14.03 mm, 13.05 mm for UR, SR and DR case of the CRB model. Figure 8 shows the settlement ratio with number of geogrid layer with different excitation energy. CVB model is able to predict the responses while CRB model overpredicts the response.

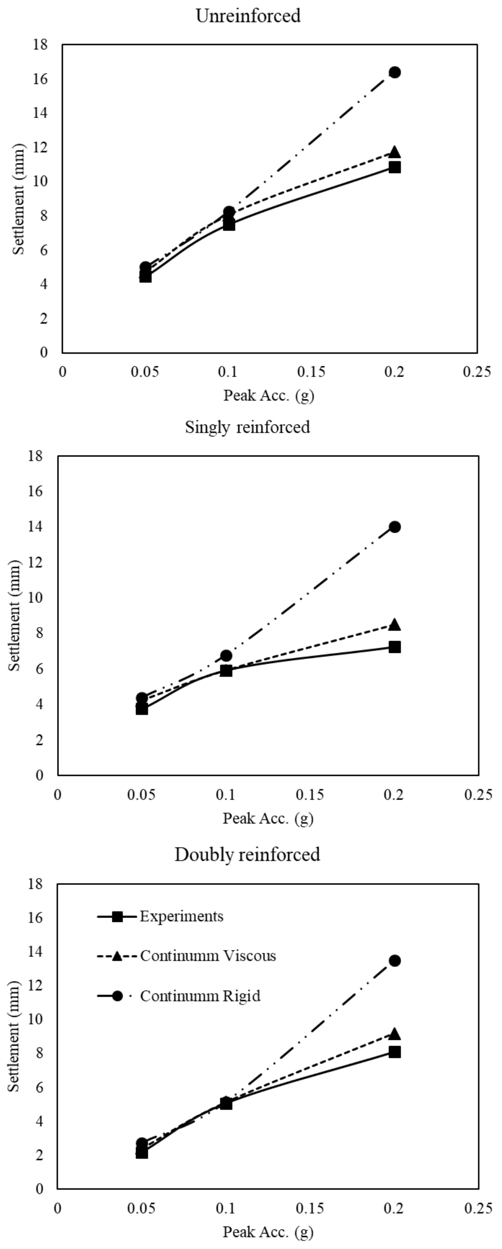

Figure 7 Maximum Settlement of footing with peak acceleration amplitude variations for the UR, SR, DR case

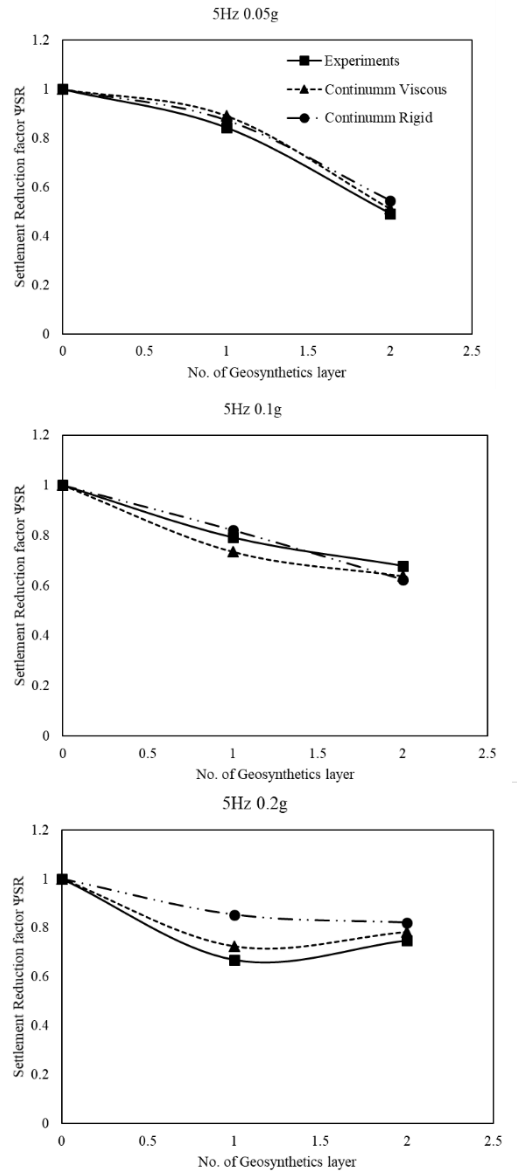

Figure 8 Settlement ratio with the no. of reinforced geogrid layer with different excitation energy.

6. Conclusions

Use of geosynthetic layers to reinforce and improve the performance of the footing on the dry sand bed is evident from settlement reduction point of view. Single-layer geosynthetics seems to be the optimum case for settlement reduction purpose. The finite element numerical modelling predicts the experimentally observed behavior reasonably well. The continuum viscous model (CVB) is able to simulate the response of the soil-foundation system in a better way compared to the continuum rigid boundary (CRB) model.

7. References

[1] Fukuda, N. … Sutoh, Y.(1987): Foundation Improvement by Polymer Grid Reinforcement. 365–368.

[2] Omar, M.T. … Wright, M.A.(1992): *A Comparison of the Ultimate Bearing Capacity of Square and Strip Foundations on Geogrid Reinforced Sand*. In: Proceedings of Numerical Models in Geomechanics. p.p: A.A. Balkema, 967–976.

[3] Yetimoglu T., W. … Saglamer, A.(1994): Bearing Capacity of Rectangular Footing on Geogrid-Reinforced Sand. *J. Geotech. Eng.*, **120**, 2083–2099.

[4] Adams, M.T. and Collin, J.G.(1997): Large Model Spread Footing Load Tests on Geosynthetic Reinforced Soil Foundations. *Geotech. Geo-environmental Eng.*, **123**, 66–72.

[5] Lin, M. and Wang, K. (2006.: Seismic slope behavior in a large-scale shaking table model test. **86**, no date, 118–133.

[6] Yang, K.H. … Kencana, E.Y.(2013): Acceleration-amplified responses of geosynthetic-reinforced soil structures with a wide range of input ground accelerations. *Geotech. Spec. Publ.*, (231 GSP), 1178–1187.

[7] Binquet, J., and Lee, K..(1976): Bearing capacity tests on reinforced earth slabs. 11F, 16R. *Int. J. Rock Mech. Min. Sci. Geomech. Abstr.*, **13** (3), 31.

[8] Guido, V.A. … Sullivan, M.J.(1985): *Bearing Capacity of a Geotextile Reinforced Foundation*. In: Proceedings of the 11th International Conference on Soil Mechanics and Foundation Engineering. A.A. Balkema, the Netherlands: Vol.3, 1777–1780.

[9] Mosallanezhad, M. … Ghahramani, A.(2007): Experimental study of bearing capacity of granular soils, reinforced with Innovative Grid- Anchored system. *Geotech. Geol. Eng.*, **25** (1), 123–137.

[10] Chung, W. and Cascante, G.(2007): Experimental and Numerical Study of Soil-Reinforcement Effects on the Low-Strain Stiffness and Bearing Capacity of Shallow Foundations. *Geotech. Geol. Eng*, **25**, 265–281.

[11] Latha G. M. & Somwanshi, A.(2009): Bearing Capacity of Square Footings on Geosynthetic Reinforced Sand. *Geotext. Geomembranes*, **27**, 281–294.

[12] Keskin, S.M. and Laman, M.(2012): Model studies of bearing capacity of strip footing on sand slope. *KSCE J. Civ. Eng.*, **17** (4), 699–711.

[13] Shadmand, A. and Ghazavi, M.(2017): Load settlement characterstics of large-scale square footing on sand reinforced with opening geocell reinforcement. *Geotextile and geomembranes*, **46**, 319–326.

[14] Arab, M.G. … Tahmaz, A.(2017): Numerical analysis of shallow foundations on geogrid reinforced soil. *MATEC Web Conf.*, **120**, 1–14.

[15] Standard, I.(2007): *IS 800-2007: GENERAL CONSTRUCTION IN STEEL — CODE OF PRACTICE*.

[16] IS:1893(2002): Criteria for Earthquake Resistant Design of Structures. *Indian Stand.*, 1–44.

[17] Vivek, B. and Raychowdhury, P.(2017): Influence of SSI on period and damping of buildings supported by shallow foundations on cohesionless soil. *Int. J. Geomech.*, **17** (8), 1–14.

[18] Iai, S.(1989): Similitude for shaking table tests on soil-structure-fluid model in 1g gravitational field. *Soils Found.*, **29** (1), 105–118.

[19] Herrick, J.E. and Jones, T.L.(2002): A dynamic cone penetrometer for measuring soil penetration resistance. *Soil Sci. Soc. Am. J.*, **66** (4), 1320–1324.

[20] Lu, J. … Law, K.H.(2004): Computational modeling of nonlinear soil-structure interaction on parallel computers. *13th World Conf. Earthq. Eng.*, 530.