An experimental study to increase bearing capacity and reduce bulging of stone columns using geogrid reinforced sand bed

Ashim Dey & Prasenjit Debnath
National Institute of Technology Silchar, Assam, India

ABSTRACT: Stone columns are normally used to improve a soft clay deposit by increasing the bearing capacity and reducing the compressibility. However, the settlement of the soft clay cannot be substantially reduced due to bulging of the stone columns. It is therefore needed to develop a technique so that the expected settlement of the soft clay reinforced with stone columns is greatly reduced under an imposed load. In the present study a series of laboratory model tests were carried out to observe the effect of normal sand bed (NSB) and geogrid-reinforced sand bed (GRSB) resting over a group of geotextile wrapped stone columns floating in soft clay. Significant improvement in the load-carrying capacity and reduction in settlement were observed. Due to placement of geogrid reinforced sand bed, the depth of bulging of the stone columns increases while the bulge diameter reduces. As compared to unreinforced clay, 1.72 fold, 2.83 fold and 5.48 fold increases in bearing capacity were achieved by the provision of only stone columns, stone columns with NSB and stone columns with GRSB, respectively. The critical thicknesses of NSB and GRSB were found to be 0.3 times and 0.2 times the diameter of the footing. The critical diameter of the geogrid layer is 2.5 times the diameter of the footing. The optimum length of the stone column is 6 times the diameter of the column. The observations showed that the stone columns with GRSB exhibit negligible settlement under vertical load

Keywords: Geosynthetics, Unreinforced sand bed, Geogrid reinforced sand bed, Group of stone columns

1 INTRODUCTION

Use of stone columns is considered as one of the ground improvement techniques for a soft soil. Moreover, the stone columns also act as vertical drains and speed up the consolidation process of the surrounding soft clay (Han and Ye 1992). As per IS 15284 Part I (2003) soft clays with undrained shear strength ranging from 7 to 50 kPa can be improved by stone columns. Over the past three decades several researchers conducted a number of tests to confirm that stone columns improve the bearing capacity of the soft clay (Balaam and Booker 1981; Poorooshasb and Meyerhof 1997; Priebe *et al.* 1998; Black *et al.* 2007; Tang *et al.* 2015 etc). It is reported that the bearing capacity can be further improved and the settlement can be further reduced by minimizing bulging of the stone columns. Geosynthetic sheets can be conveniently used horizontally as a reinforced layer in the granular columns (Madhav *et al.* 1994; Wu and Hong 2008). Generally, a cushion of sand bed is placed over the stone columns to distribute the stresses uniformly and to provide a drainage path (Mitchell 1981). Very limited research reported in the literature indicates that this sand layer, when reinforced with planar geosynthetics, can noticeably improve the bearing capacity of the foundation system (Abdullah and Edil 2007; Deb et al., 2011). Most of the above-mentioned experiments used stone columns resting over a hard stratum. Group effect was also not considered. In the present study it is proposed to carry out experimental works on group of geotextile wrapped stone columns floating in the soft clay. It is observed that the bearing capacity increases and settlement reduces with introduction of reinforced soil cement bed over the stone columns.

2 MATERIALS USED

In the current experimental investigation clay, sand, stone aggregate, geogrid and geotextiles were used.

2.1 *Clay*

Clay was collected from paddy fields adjacent to NIT Silchar campus. The gradation curve of the clay is shown in Figure 1. The clay was used as a foundation bed in which the stone columns were constructed. Index properties of the clay (ASTM D4318, 2005 and ASTM D2487, 2006) are shown in Table 1. It is intended to carry out the laboratory tests with a soft clay having undrained cohesion, S_u= 10 kPa. In order to find out the quantity of water required to attain this strength a series of laboratory Unconfined Compression tests (UCS) were conducted on remolded soil samples with different water contents. It was obtained that the water content corresponding to S_u =10 kPa was 32%. At this water content, the bulk unit weight (γ) was 17.20 kN/m^3. It was tried to maintain the water content of 32% for all the tests in this study.

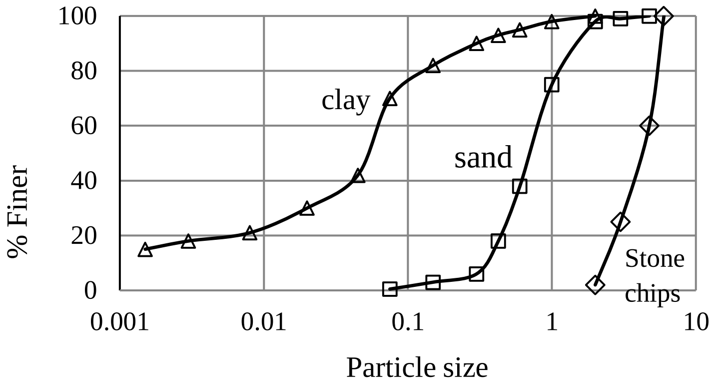


Figure 1. Particle size distribution curves of materials used.

2.2 *Sand*

The sand was used as a blanket over the stone columns and was collected locally from a river. The gradation curve of sand is shown in Figure 1. The sand bed was prepared at 70% relative density for all the tests. Basic properties of sand are presented in Table 1

2.3 *Stone aggregates*

The stone columns were prepared with poorly graded stone aggregates, with particle size ranging from 2 to 6 mm as was used by other researchers (Deb *et al.* 2011; Ali *et al.*, 2012) with a consideration of scale effect. The gradation curve of stone aggregate is shown in Figure 1. The angle of internal friction was obtained through a large-scale direct shear test with a sample size 300mm × 300mm × 150 mm. The stone aggregates were compacted to 70% relative density, same as the placement density of stone column. Basic properties of stone aggregates are listed in Table 1

Table 1**.** Properties of materials used

Property	Clay	Sand	Stone chips
Liquid limit (%)	51		
Plastic limit (%)	22		
Specific gravity	2.62	2.67	2.65
Coefficient of permeability (m/s)	6.77×10^{-10}		
Bulk unit weight (kN/m^3)	17.15 at 32% water content	16.7 At 70% relative density	15.8 At 70% relative density
In situ vane shear strength (kPa)	10.0		
Maximum dry density (kN/m^3)		17.78	16.64
Minimum dry density (kN/m^3)		14.63	14.13
Angle of friction in degree		42 At 70% relative density	46 At 70% relative density
USCS classification system	CH	SP	GP

2.4 *Geosynthetics*

Biaxial geogrid, made of high-density polyethylene was used as a reinforcement layer in the sand bed. The geotextile was used for encasement of the stone columns (Fig. 3.9(b)).

3 TEST SET UP

To prepare the test setup, a steel tank of size 1000 × 1000 × 1000 mm high as shown in Figure 2 was used. The four sides and bottom of the tank were made of 6 mm thick mild steel sheet and were braced laterally with mild steel angles on the outer surface to achieve necessary stiffness against bending during the tests. Initially, the inner surface of the tank wall was coated with a thin film of silicon grease and then covered with a smooth polythene sheet to minimize friction between the soil and the tank wall. Proper overlapping of the polythene sheets was made to avoid loss of water through the sides. In order to maintain a constant density and water content for all the tests, the tank was filled with soft clay in layers, each of 100 mm thick. The soft clay layer was prepared from a known weight of dry clay grinded to fine powder and thoroughly mixed with water corresponding to the water content of 32%. The clay lump was then put in the tank and compacted with a square steel rammer of size 150 mm and weight 10 kg to attain a bulk unit weight around 17.2 kN/m^3. The process was repeated till the test tank was full with clay upto a height of 900 mm. The prepared test bed was covered with a thick plastic sheet and left undisturbed for seven days. Undisturbed soil samples were collected using thin walled cylindrical samplers from different locations of the test bed and their properties were evaluated. Apart from sample collection, small scale vane shear tests were also carried out at different locations. The average moisture content, bulk unit weight and shear strength of the clay, in the test beds, were found to be 32%, 17.20 kN/m^3 and 10 kPa respectively. Their coefficient of variability was in the range of 1.3%.

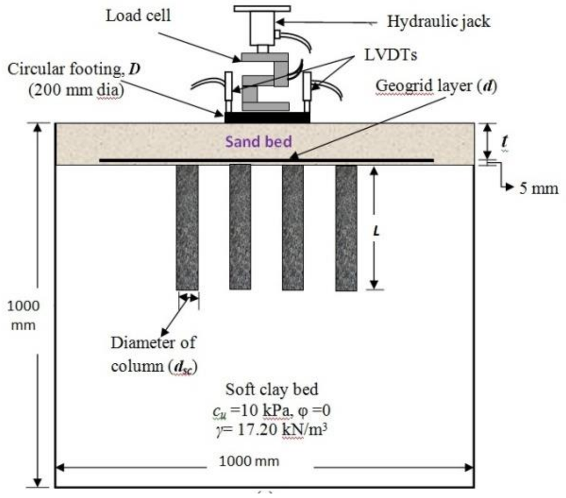
All columns for the group tests were constructed in the clay bed. A helical steel auger of diameter 50 mm was used to scoop out the clay from the inner part of the steel pipe. To minimise suction effect, maximum 50 mm thickness of soil was removed at a time. Boring was continued till the total penetration depth became 300 mm. Once the boring was completed, the geotextile encasement in the form of a cylinder of internal diameter 50 mm was inserted with the help of a circular wooden rod of 45 mm diameter. Figure 2 (b) shows the plan of the proposed group of stone column arrangement. It is to be mentioned that this arrangement depicts the behavior of the central three columns shown within a circle in Figure 2 (b). As per IS 15284 Part I (2003), at least 12 columns are required for a 3 column-group test in order to simulate the field condition of the intervening soil. In the present study, the columns were arranged in a triangular pattern with a spacing of 2.5 times the diameter of the columns. The area replacement ratio was found to be 14.5% for this arrangement. The required weight of stone aggregate corresponding to 70% relative density was obtained from the volume of the hole considering 10% increase in diameter during compaction. Total weight of the stone was divided into six equal parts and the column was constructed in six layers with proper compaction by a circular steel tamper of 25 mm diameter and mass 1.0 kg with 30 blows from 200 mm drop, leading to a density of 15.8 kN/m^3 corresponding to 70% relative density.

The sand bed was prepared in layers by compacting dry sand to a relative density of 70%. A 5 mm thick sand layer was placed over the clay bed; a circular geogrid layer was placed on the sand layer at the center of the group stone columns (Fig. 2a); finally, additional sand as per the required thickness was placed over the geogrid layer.

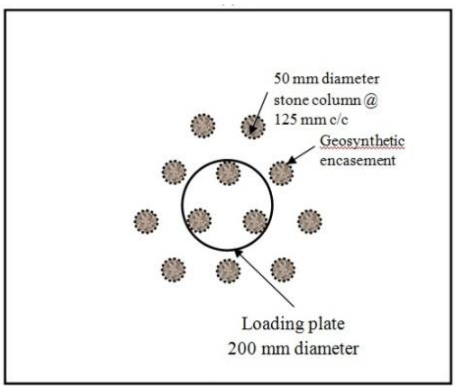
The footing used was made of a rigid steel plate of diameter (D) 200 mm and thickness 15 mm. The footing was placed at the centre of the stone column arrangement.

4 TEST PROCEDURE

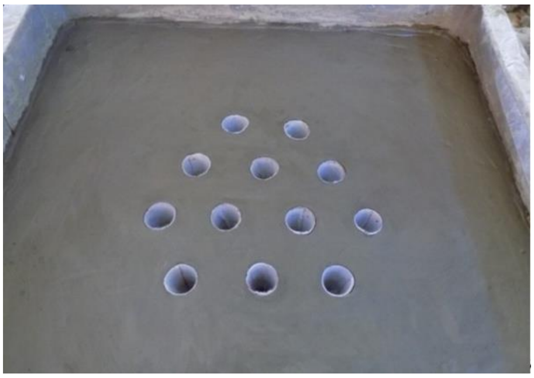
In all the tests, reaction loading with a hydraulic jack was applied through a load cell placed over the footing. The capacity of the load cell was 100 kN. The load was applied in equal increments and each load increment was maintained constant until the footing settlement was stabilized and no significant change in settlement (i.e., < 0.02 mm/min) was observed. For all the experiments short-term loading test was conducted. The settlement during each load increment was observed through two LVDTs (Linear Variable Differential Transducers) with a least count of 0.01 mm placed at diametrically opposite ends on the footing. For recording the LVDT and the load cell data a twelve-channel portable data acquisition system was used. In all the tests, the load was applied until the total settlement reached a value 20% of the footing diameter. After the load test, without disturbing the columns, thin cement slurry was poured under gravity in the three central stone columns to study their bulging and lateral deformations. Table 2 shows a summary of the tests conducted for this study.



(a) Sectional elevation



(b) Arrangement of group of three stone columns



(c) Geotextile placement in the boreholes

(d) Placement of geogrid over 5 mm thick sand

Figure 2. Test set up for group of stone columns

Table 3.5 Summary of the experimental programme with OSC and VESC.

Test series	Type of reinforcement	Details of parameters investigated
1	Unreinforced clay bed	–
2	Clay+OSC	L=300 mm, d_{sc}= 50 mm, S = 2.5×d_{sc}
3	Clay+OSC+USB	Variable parameters: t/D = 0.1, 0.15, 0.2, 0.3, 0.4, 0.5 Constant parameters: L/d_{sc} =6, S/d_{sc} =2.5
4	Clay+OSC+GRSB	Variable parameters: t/D = 0.1, 0.2, 0.3 Constant parameters: L/d_{sc} =6, S/d_{sc} =2.5, d/D=4
5	Clay+OSC+GRSB	Variable parameters: d/D = 1.5, 2.0, 2.5, 3.0, 4.0 Constant parameters: L/d_{sc} =6, S/d_{sc} =2.5, t/D=0.2

Note: L = length of stone column; d_{sc} = diameter of stone column; S = spacing of stone columns; d = diameter of geogrid layer; D = diameter of footing; t=thickness of sand bed; OSC= ordinary stone column; VESC= vertically encased stone column; USB = unreinforced sand bed; GRSB= geogrid reinforced sand bed.

5 SCALING CONSIDERATION

The main drawback of all the laboratory experiments is the scaling effect. A similitude ratio is defined to express the dimensional scaling. It is the ratio of any linear dimension of the prototype to the equivalent dimension of the model. For a clayey soil having a very low permeability, the similitude ratio should be as small as possible. Typically, prototype stone columns have a diameter (d_p) ranging from 0.6 to 1.0 m. Thus, considering the diameter of the stone column in the model tests as 50 mm, the similitude ratio becomes 12 to 20. Again in the prototype stone columns, l/d_p ratio varies between 5 and 20 (Shahu and Reddy, 2011). Considering the l/d ratio to be 6 in the model tests, the column length for 50 mm diameter test columns will be 300 mm. Typically, the particle size of stone aggregates (d_s) varies between 25–50 mm in the prototype stone columns (for d_p = 0.6–1.0 m). It is considered that in the prototype stone columns d_p/d_s ratio varies between 12 and 40 (Wood et al. 2000). The particle size of the stone in the model stone columns was kept as 2 to 6 mm corresponding to d_p/d_s ratio ranging between 9 and 25. It is therefore considered that the scale effects are eliminated in the present study.

Considering that the failure wedge in the foundation bed extends over a distance of about 2 to 2.5 times the footing width, away from its centre. In the present study, the dimension of the test tank was 1000 mm x 1000 mm, where as the diameter of the footing was 200 mm. Thus, the slip planes are not likely to be intercepted by the tank walls.

6 EXPERIMENTAL OBSERVATIONS AND DISCUSSIONS

6.1 *Influence of thickness of USB and GRSB*

Influence of thickness of USB (t) on the pressure-settlement response of stone columns has been studied. Six different thicknesses of sand beds, for example t = 0.1D, 0.15D, 0.2D, 0.3D, 0.4D and 0.5D have been considered. The pressure-settlement responses for the USB of different thicknesses are shown in Figure 3. Pressure corresponding to 20% settlement is considered as the ultimate load carrying capacity for comparison of different results. It is interesting to notice that the load-carrying capacity is increased by 32.28% as the thickness of USB is increased from 0.1 to 0.3D, and an additional 6% only when the thickness of USB is increased from 0.3 to 0.5D. Hence, the optimum thickness of the USB may be considered as 0.3 times the diameter of the footing (i.e., 0.3D)

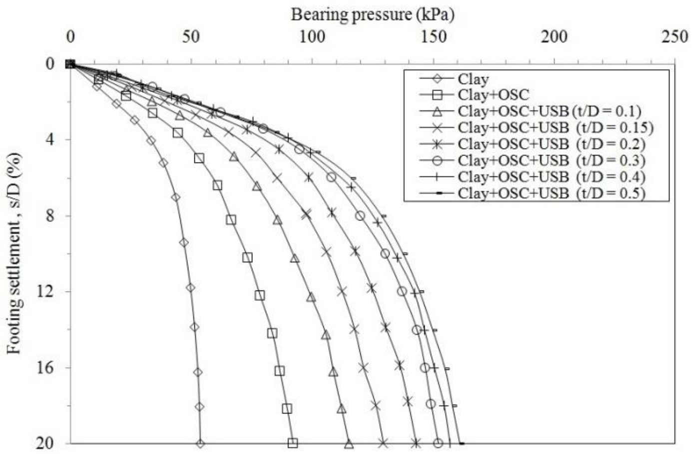


Figure 3 Bearing pressure against settlement for ordinary clay, clay with stone column and clay with stone column overlaid by sand bed of different thicknesses

Pressure-settlement responses for the GRSB of different thicknesses are shown in Figure 4. To study the influence of the thickness of the GRSB, the diameter of the geogrid layer was chosen as 4D. Three different thicknesses of sand beds, t = 0.1D, 0.2D and 0.3D have been chosen to study the optimum thickness of the sand bed. It is observed that the load-carrying capacity is increased by 12.88% as the thickness of the GRSB is increased from 0.1 to 0.2D, however, when the thickness of the GRSB is increased from 0.2 to 0.3D, an additional 2.22% increase in load-carrying capacity is observed. Hence, the optimum thickness of the GRSB is considered as 0.2 times the diameter of the footing (i.e., 0.2D). As compared to USB of thickness 0.1D, the percentage increase in bearing capacity can be achieved by 133.30%, 163.36% and 169.22% with an increase in thickness of the GRSB from 0.1, 0.2 and 0.3D, respectively.

6.2 *Influence of diameter of geogrid layer*

Typical footing pressure-settlement responses for the GRSB having different diameters of geogrid are presented in Figure 5. The optimum thickness of the GRSB (i.e., 0.2D) has been considered to study the influence of geogrid reinforcement diameter. Five different diameters of geogrid, for example d = 1.5D, 2D, 2.5D, 3D and 4D have been chosen to obtain the optimum diameter of the geogrid. It is interesting to note that as the diameter of the geogrid increases from 1.5D to 2.5D, a considerable increase in bearing capacity and reduction in the settlement is observed. However, with further increase in diameter from 2.5*D* to 4*D*, the relative improvement is very less. Hence, the optimum diameter of the geogrid is considered as 2.5 times the diameter of the footing (i.e., 2.5*D*).

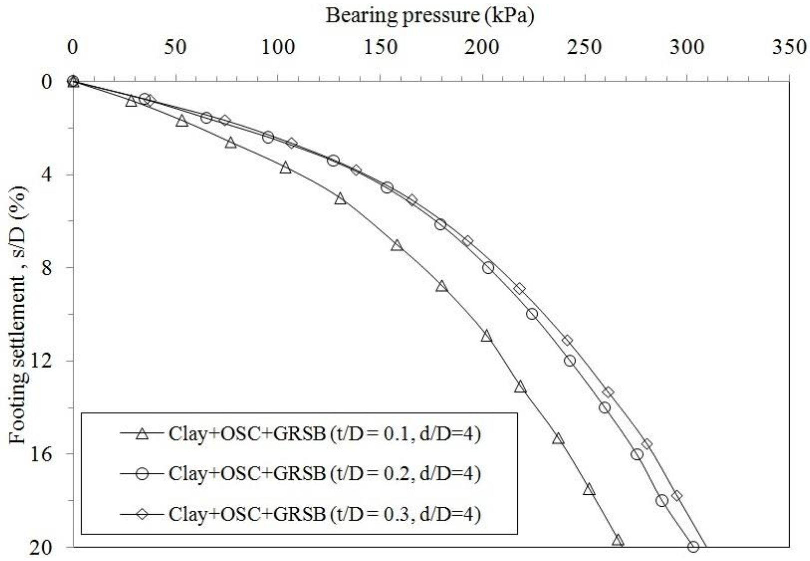


Figure 4. Effect of thickness of geogrid reinforced sand bed on bearing pressure

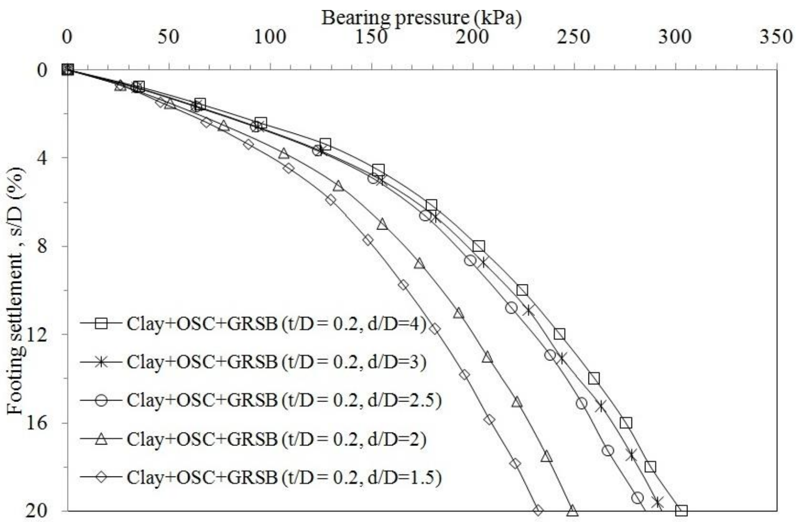


Figure 5. Effect of thickness of geogrid reinforced sand bed on bearing pressure

6.3 *Influence of various combinations of reinforcements*

Figure 6 shows the pressure-settlement characteristics of the footing for different combinations of soil and reinforcement like clay bed alone; clay bed improved by stone columns only; clay bed improved by stone columns with USB and GRSB. The optimum diameter (i.e. 2.5*D*) of the geogrid layer obtained from the model test has been taken. Corresponding to 20% settlement, the load intensity for the clay bed alone is obtained as 53.82 kPa. Again, corresponding to this load intensity reduction in the settlement for the soil improved by the stone columns alone is 74.68%; the soil improved by the stone columns with USB and GRSB are 89.05% and 93.03%, respectively. Again, corresponding to 20% settlement, the load intensity for the clay bed improved by the stone columns is obtained as 92.11 kPa. At this load intensity, reduction in the settlement with the application of USB and GRSB are 78.36% and 87.13% respectively. Thus, it can be said that the effectiveness of geogrid reinforcement is increased with an increase in the pressure intensity.

The increase in the bearing capacity is quantified through a nondimensional improvement factor, defined as the ratio of bearing pressure of the reinforced to that of the unreinforced clay bed at equal footing settlement (*s/D* %). The variation of improvement factors corresponding to a footing settlement for different reinforcement combinations is shown in Figure 7. Numerically an improvement of 1.72 fold in load carrying capacity of normal clay bed is observed when the clay bed is improved with stone columns alone; 2.83 fold with stone columns with 60 mm thick USB and 5.30 fold with stone columns with 40 mm thick GRSB. Hence it can be said that the stone columns along with GRSB composite are a superior form of reinforcement that can give a better performance than the conventional ones.

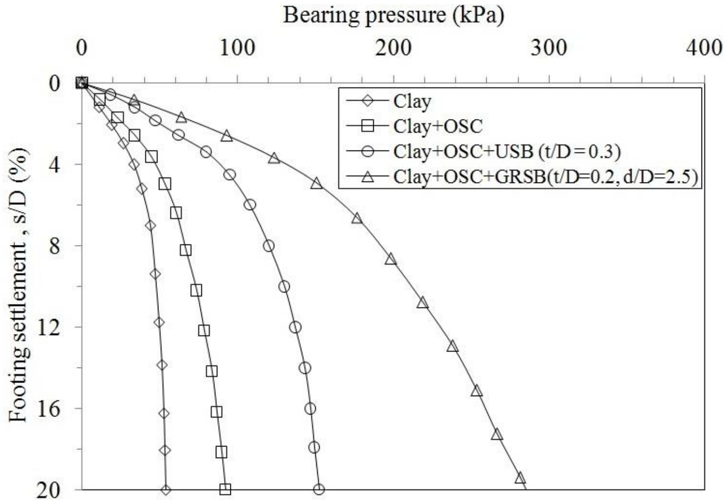


Figure 6. Comparison of bearing capacity for different combinations of stone columns and sand bed

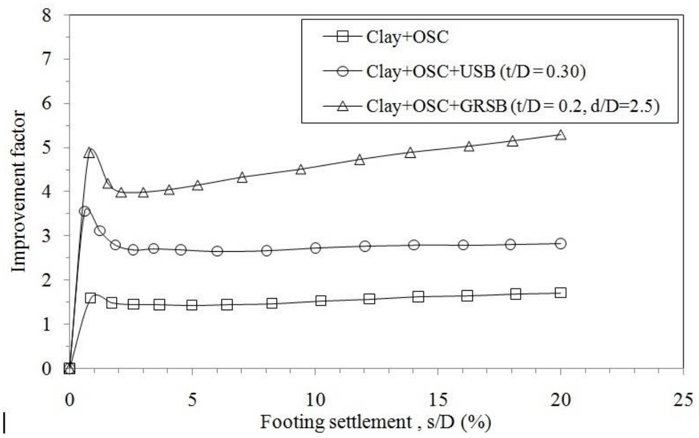


Figure 7. Variation of improvement factor for various reinforced conditions

6.4 *Bulging of stone columns*

Figure 8 shows the bulging and lateral deformation behaviour of the central three stone columns under various conditions. It is observed that the lateral deformation of the columns is mostly outward. It has also been observed that failure is a combination of lateral deformation and bulging of stone columns group. A similar pattern of column deformation was reported by Wood *et al.* (2000) and Ghazavi and Afshar (2013). When the soft clay is improved by stone columns alone, a maximum bulging of 7 mm is found at a depth of 120 mm from the top of the stone columns. However, the maximum bulging is reduced to 4.5 mm and deformation depth is increased to 128.5 mm when the stone columns are overlaid by USB. For stone columns overlaid by GRSB, the maximum bulging is further reduced to 2.0 mm and deformation depth is further increased to 148.5 mm. Hence, maximum deformation occurs at a depth of 2.4, 2.57 and 2.97 times the diameter of the column when the clay bed is reinforced by stone columns alone, stone columns with USB and stone columns with GRSB respectively. As compared to the stone column reinforced clay bed, 35.72% and 71.43% reduction in maximum bulging diameter are observed when the stone columns are combined with USB and GRSB respectively

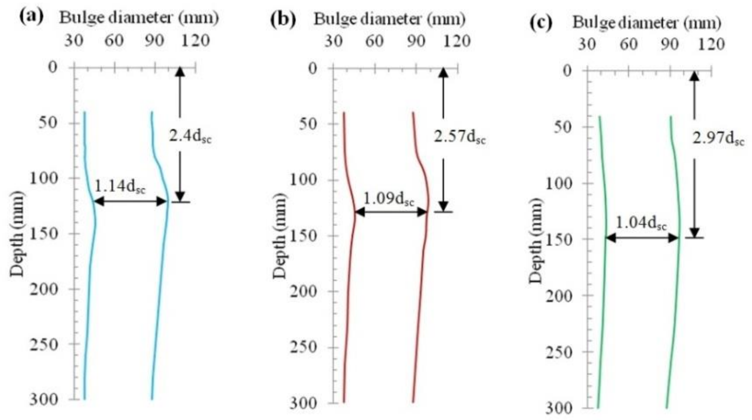


Figure 8. Bulging of stone column when soft clay is improved with (a) group of stone columns, (b) group of stone columns with 60 mm USB (c) group of stone columns with 40 mm GRSB

7 CONCLUSIONS

The work reported herein investigates the performance of USB and GRSB on the increase in the bearing capacity of a group of OSC and VESC floating in soft clay. The following conclusions are drawn from the present study:

- As compared to unreinforced clay, 1.72 fold, 2.83 fold and 5.30 fold increase in bearing capacity can be achieved with the provision of stone columns alone, stone columns with USB and stone columns with GRSB, respectively.
- The critical thickness of USB and GRSB can be taken equal to 0.3 times and 0.2 times the diameter of the footing, beyond which, any further increase in thickness of the sand bed bearing capacity of the stone columns composite foundation bed is marginal. The critical diameter of the geogrid layer is 2.5 times the diameter of the footing.
- When the soft clay is reinforced by stone columns alone, stone columns with USB and stone columns with GRSB, maximum deformation occurs at a depth of 2.4, 2.57 and 2.97 times the diameter of the column, respectively. As compared to a clay bed reinforced with stone columns only, 35.72% and 71.43% reduction in maximum bulge diameter and 7.08% and 23.75% increase in depth of location of maximum deformation of the stone column are observed for stone columns coupled with USB and GRSB, respectively.
- The improvement factor of stone columns with GRSB shows an increasing trend with an increase in footing settlement.
- The optimum length of a group of stone columns coupled with GRSB is 6 times the diameter of the column (i.e. $6d_{sc}$) for an area replacement ratio equal to 14.5%. For a higher length, a marginal improvement in the bearing capacity is observed.

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