Studies on the Behavior of Single and Group of Geosynthetic Encased Stone Columns

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Abstract: The stone columns (or granular piles) are increasingly being used as ground reinforcement elements for supporting a wide variety of structures including buildings and flexible structures. The stone columns derive their load capacity from the confinement offered by the surrounding soil. In very soft soils this lateral confinement may not be adequate and the formation of the stone column itself may be doubtful. Wrapping the individual stone columns with suitable geosynthetic is one of the ideal forms of improving the performance of stone columns. This type of encasement by geosynthetic makes the stone columns stiffer and stronger. In addition, encasement prevents the lateral squeezing of stones in to the surrounding clay soil and vice versa, preserves drainage function of the stone column and frictional properties of the aggregates. In spite of many advantages, the behavior and the mechanism of the geosynthetic encased stone columns is not thoroughly understood. This paper investigates the qualitative and quantitative improvement of individual load capacity of stone column by encasement through laboratory model tests conducted on stone columns installed in clay bed prepared in controlled condition in a large scale testing tank. The load tests were performed on single as well as group of stone columns with and without encasement. Tests were performed with different geosynthetics for the encasement of stone column. The results from the load tests indicated a clear improvement in the load capacity of the stone column due to encasement. The increase in the axial load capacity depends very much upon the modulus of the encasement and the diameter of the stone column. The increase in the stress concentration on the stone columns due to encasement was also measured in the tests. The results from the tests were used to develop the design guidelines for the design of geosynthetic encasement for the given load and settlement.

DOI: 10.1061/(ASCE)GT.1943-5606.0000187

CE Database subject headings: Stone columns; Soft soils; Clays; Geosynthetics; Soil stabilization; Encasements.

Author keywords: Stone columns; Soft clay; Geosynthetic; Encasement; Ground improvement.

Introduction

The structures constructed on soft clays experience several problems like excessive settlements, large lateral flow of soft clay beneath the structures and loss of global and/or local stability. The popular remedial measure to overcome these problems is to improve the soil with the insertion of stone columns (also called granular piles) in regular grid. The stone columns are nothing but vertical columnar elements formed below the ground level with compacted and uncemented stone fragments or gravels or sand. These load bearing columns usually penetrate through the weak strata. The construction of stone column involves partial replacement or lateral compaction of unsuitable or loose subsurface soils with a compacted vertical column of stone aggregate. The presence of the columns creates a composite material which is stiffer and stronger than the original soil.

Since the pioneering work by Greenwood (1970), there have been much research based on stone columns, reported in the literature (Hughes et al. 1975; Barksdale and Bachus 1983; Priebe 1995). Balaam and Booker (1981), Lee and Pande (1998), and Guetif et al. (2007) have investigated analytically the ground improvement by insertion of stone columns. Bouassida et al. (1995), Rao et al. (1997), Wood et al. (2000), and Maurya et al. (2005) have studied the performance of the group of stone columns. It is very well established in literature that the stone columns derive their load capacity from the lateral confining pressure offered by the surrounding soils. When the stone columns are installed in very soft clays, they may not derive significant load capacity due to the low lateral confinement. McKenna et al. (1975) reported cases where the stone column was not restrained by the surrounding soft clay which lead to excessive bulging and also the soft clay squeezed into the voids of the aggregate. In such situations, the stone column itself may need to be provided with additional confinement for its improved performance. One ideal form of providing such support to stone columns is wrapping the individual stone columns using suitable geosynthetic (Fig. 1). This encasement imparts additional confinement to the stone column and brings in several advantages like increased stiffness of column, preventing the loss of stones into the surrounding soft clay, preserving the drainage and frictional properties of the stone aggregates, etc., as described by Raithel et al. (2002), Alexiew et al. (2005), Brokemper et al. (2006), Murugesan and Rajagopal (2006a,b, 2007a), Kempfert and Gebreselassie (2006), and di Prisco et al. (2006).

The concept of encasing the stone column by wrapping with

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Note. This manuscript was submitted on May 27, 2008; approved on June 29, 2009; published online on July 1, 2009. Discussion period open until June 1, 2010; separate discussions must be submitted for individual papers. This paper is part of the ***Journal of Geotechnical and Geoenvironmental Engineering***, Vol. 136, No. 1, January 1, 2010. ©ASCE, ISSN 1090-0241/2010/1-129–139/$25.00.

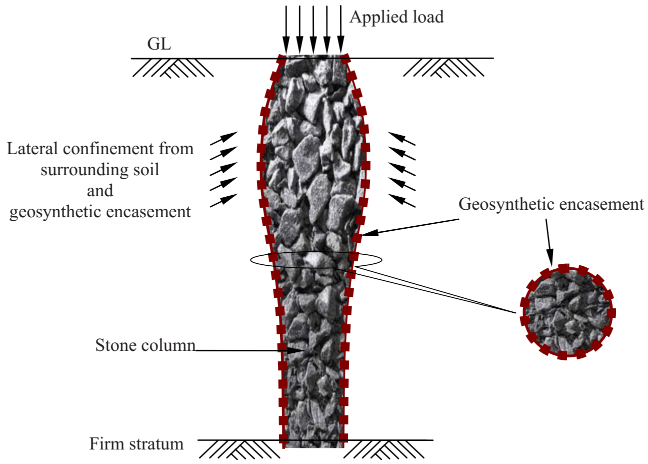

Fig. 1. Schematic of geosynthetic ESC

geotextile was proposed by Van Impe in the year 1985 (Van Impe 1989). Later Bauer and Al-Joulani (1994) have investigated the performance of sleeve reinforced stone columns through laboratory uniaxial and triaxial compression tests. Ayadat and Hanna (2005) have reported the benefits of encasing the stone column installed in collapsible soils. Murugesan and Rajagopal (2006a,b) have evaluated the behavior of ordinary stone columns (OSCs) without encasement and geosynthetic encased stone columns (ESCs) through numerical analyses. They reported that the ESCs are stiffer than the OSCs and were found to be less dependent on the strength of surrounding clay soil for their load capacity. Murugesan and Rajagopal (2007a) have performed laboratory model tests on the stone column installed in a unit cell tank. It was reported that the modulus of the encasement plays major role in strength of the encased column. The effect of encasement was found to decrease with an increase in the diameter of the stone column. The improvement in the load capacity was quantified based on the modulus of the geosynthetic and the diameter. It was also shown that the geosynthetic ESCs were found to act as semi-rigid piles not undergoing any catastrophic failures. Black et al. (2007) examined the performance of small-scale stone columns that were enhanced by jacketing with tubular wire mesh and found that the load-carrying capacity and the settlement performance of stone columns in peat can be improved by this method. Malarvizhi and Ilamparuthi (2007) have compared the performance of stone columns with and without geosynthetic encasement and found the ESCs to be more effective. Murugesan and Rajagopal (2007b, 2009) have reported the results from laboratory model tests on the lateral load capacity of stone columns against shear movements and brought out the qualitative improvements in their shear load capacity.

In spite of applying this technique for constructions in soft soils, there are only limited investigations reported in literature to understand the behavior of ESCs. This paper reports the results from a series of laboratory tests performed on ESCs as well as OSCs (without any encasement) installed in clay bed formed in a large scale model test tank. The results mainly quantify the improvements achieved in the load capacity over OSC due to the encasement by geosynthetic. The increase in the stress concentration on the stone column due to encasement was also explored through the load tests on the group of stone columns. Based on the results from the experiments, design guidelines for the design of geosynthetic ESC have been proposed. Design chart has been developed for the selection of geosynthetic for the encasement based on the soil and loading condition in the form of nondimensional parameters relating all dependent variables in the design.

Description of Experiments

Material Properties

The stone aggregates used to form the stone columns were angular granite chips whose properties are listed in Table 1. The peak angle of frictional resistance of stone aggregate determined from direct shear tests (in large shear box with plan dimensions of 300 mm × 300 mm and height of 200 mm) was found to be 41.5° within a normal pressure of 300 kPa.

Four different types of geosynthetics were used to encase the stone columns in the present study, namely woven geotextile, nonwoven geotextile, and soft meshes having two different aperture opening sizes, Soft Grid-1 and Soft Grid-2. The tensile strength properties of these geosynthetics determined from standard wide width tension tests (ASTM 1986) are listed in Table 2. As the geosynthetics were stitched to form the tube for encasing the stone column, the seam strength of the geosynthetic was also determined with geosynthetic specimens having a seam at midlength. The load deformation behavior observed from the junction tensile tests on different geosynthetics is shown in Fig. 2. The failure observed in the seam tensile strength test of woven and nonwoven geotextile was due to the tearing of the seam, whereas in the case of soft grids, the seam was intact and the sample elongated considerably without any failure even at large stain levels.

Table 1. Properties of Stone Aggregate Used for Stone Columns

Serial number	Properties	Value
1	Size range	2 to 10 mm
2	D_{10}, D_{30}, and D_{60}	3.5, 6.2, and 7.1 mm
3	Gradation symbol as per USCS	GP
4	Specific gravity	2.65
5	Peak angle of internal friction	41.5°
6	Dry density	1.6 g/cm^3
7	Relative density	73%
8	Angularity number	7.0
9	Maximum and minimum void ratios	0.9 and 0.5

Table 2. Properties of Geosynthetics Used for the Encasement

Strength properties	Woven geotextile	Nonwoven geotextile	Soft grid-1	Soft grid-2
Ultimate tensile strength (kN/m)	20	6.8	2.5	1.5
Strain at ultimate strength (%)	21	55	52	65
Ultimate tensile strength from tests with seam (kN/m)	4.7	5.1	2	1
Strain at ultimate strength from tests with seam (%)	15	45	50	65
Initial modulus (kN/m) from seam tests	17.5	12	6	2
5% secant modulus (kN/m) from seam tests	15	12	5	2

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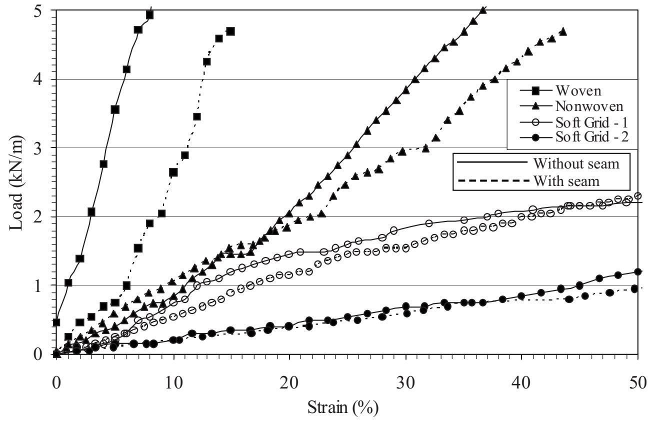

Fig. 2. Tensile load-strain behavior of geosynthetic samples with seam

Table 3. Properties of Clay Soil

Serial number	Properties	Value
1	Liquid limit	49%
2	Plastic limit	17%
3	Plasticity index	32%
4	Specific gravity	2.59
5	Moisture content after consolidation	$47 \pm 1\%$
6	In situ vane shear strength	2.5 kPa
7	Consistency index	0.06
8	Dry unit weight	11.56 kN/m^3
9	CBR value	0.11%
10	USCS classification symbol	CL
11	Degree of saturation	96%
12	In situ void ratio	1.25

Preparation of Clay Bed

The clay bed for the tests was prepared in a large test tank of plan dimensions 1.2 $m \times 1.2$ m and 0.85 m in depth. The clay soil was obtained from a lake bed and was soaked in water for one month before using it in the tests. Initially the wet clay was mixed with water equal to 1.5 times the liquid limit of the soil by kneading thoroughly in a large tank to form slurry free from any lumps and previous stress histories. This slurry was filled in the test tank and allowed to consolidate under a pressure of 10 kPa in a tank by using dead weights. The settlements in the clay bed were monitored using mechanical dial gauges having accuracy of 0.01 mm. Drainage was permitted at the top and bottom of the clay bed by placing 75-mm thick sand layer sandwiched between geotextile layers. The consolidation of the clay bed was continued for a period of 8 to 10 days until the rate of settlement was less than 1 mm/day. This procedure yielded clay beds of uniform moisture content and consistency. After consolidation, undisturbed samples were collected from the clay bed for performing different tests viz. moisture content determination, vane shear strength, degree of saturation, and in situ void ratio. The profile of water content was also determined at different locations within the test bed and found to be almost uniform with $\pm 1\%$ variation. After consolidation of the clay bed, its top surface was trimmed level to a have 600-mm depth of clay soil in the tank. For every load test, the clay bed was prepared afresh in the large tank by this method and the stone columns were installed in it. This method was carefully followed to maintain uniformity of clay bed properties between successive load tests. The properties of the clay bed prepared by this method are listed in Table 3.

Installation of Stone Column

All the load tests were conducted on the stone column installed at the center of the clay bed prepared in the large test tank (Fig. 3). The plan area of the tank is so selected that the loading on the stone column would not be affected by the tank boundaries. The stone columns were installed by displacement method, and were extended down to the bottom of the tank. Hence all the stone columns were of length 0.6 m. A casing pipe having an outer diameter equal to the diameter of the stone column was used to install the stone columns. The casing pipe was pushed into the soil till the bottom of the tank along with a base plate in order to prevent the soil from entering into the casing pipe. When the casing pipe is pulled out, the base plate remains in the soil. In the case of ESCs, the encasement was provided around the casing pipe which also covers the base plate.

The casing pipe along with the geosynthetic encasement was slowly pushed in to the clay bed vertically at the specified location in the clay surface in the tank until it reaches the bottom of the tank. Only static force was manually applied to push the casing pipe gently into the soil so as to minimize the disturbance in the clay soil that may change the properties of the clay after reinforcement. The displaced clay during the installation of stone column was taken out and the surface of the soil was trimmed level. The quantity of the stone aggregate required to form the stone column was premeasured and charged in to the casing pipe in layers of 50 mm thickness. The stone aggregate was moistened before charging into the casing pipe in order to prevent it from absorbing the moisture from the surrounding clay soil. After placing each layer of stone aggregate, the casing pipe was lifted up gently (leaving the base plate and the geosynthetic encasement intact) to a height such that a minimum overlap of 15 mm between the bottom of the casing pipe and the stone fill within the casing pipe was always maintained. This is to prevent intrusion of surrounding clay soil into the stone column or neck formation in the geosynthetic in the case of ESCs due to the lateral thrust of the surrounding clay. Immediately after lifting the casing pipe, the stone aggregate was compacted with a tamping rod (10-mm diameter and 1-m long) with 25 numbers of blows falling freely

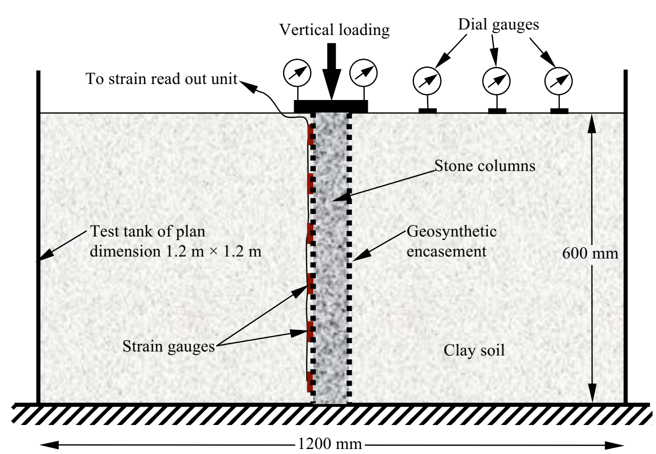

Fig. 3. Load test on single stone column in large test tank—schematic

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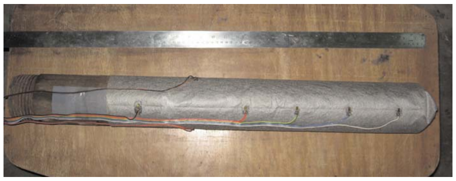

Fig. 4. Strain gauges fixed on the geosynthetic encasement

from a height of 250 mm. This method of compaction gave a dry density of 1.6 g/cm^3 to the aggregate column. Each layer of aggregate was compacted using the same number of blows. The procedure was repeated until the entire height of the stone column was formed. The top surface of the stone column was leveled and any protruding geosynthetic was cut and trimmed flush with the surrounding clay surface.

In one of the load tests, the hoop strain developed at various levels in the geosynthetic encasement was measured using strain gauges. Strain gauges along with the connecting wires were fixed to the geosynthetic at different locations along the height of the stone column as shown in Fig. 4. The strain gauges were fixed on the geotextile just after wrapping it around the casing pipe (before installation in the clay soil). The strain gauges were of Hottinger Baldwin Messtechnik GmbH made, Germany having 3-mm grid size and a resistance of 120 Ω with a gauge factor of 2.14. The strains were measured by connecting them in quarter bridge circuit with dummy strain gauges fixed on a similar surface out side the test apparatus to compensate for the effects of temperature, resistance of lead wire, etc. The strain gauges were oriented along the circumferential direction of the encasement to measure the hoop strain developed in the geosynthetic. Among the geosynthetics used, the nonwoven geotextile was more extensible and possesses suitable surface for gluing the strain gauges. Hence the strain measurements were made only in the case of stone columns with nonwoven geotextile encasement. The strain gauges and the connecting terminals were properly covered with flexible putty to prevent the direct contact of strain gages with stones or clay soil. The inner surface of the geosynthetic at the strain gauge locations was covered with Teflon tape to prevent the moisture from reaching the strain gauges through the geotextile. The glue and the putty were soft enough not to cause any localized changes in the strength properties of the geotextile.

Load Tests on Stone Columns

The stone columns thus formed in the clay bed were subjected to vertical loading at the top of the column through a loading plate displaced at a constant strain rate of 1.2 mm/min. The loading plate was displaced by a jack fixed on a reaction loading frame. The loading plates used in the tests were circular having a diameter twice that of the stone column so that the stone column along with contributory soil area of stone column is loaded (Fig. 3). In each case, the loading plate will cover the equivalent circular effective area concentrically. This method was adopted after the guidelines given in IS 15284 [Indian Standard (IS) 2003]. The loads corresponding to different displacements (in the stone column) were measured through a precalibrated proving ring (having accuracy of 0.8 N). As the loading is quick it is essentially undrained loading which simulates the loading condition immediately after the construction.

Four series of tests were conducted by varying the diameter of the stone column. First series of tests were performed on the clay bed without any stone column. Second series of tests were performed on OSCs without any encasement. Third series of tests were performed on ESCs with different diameters and type of geosynthetic encasement. Fourth series of tests were performed on groups of OSCs as well as ESCs. The group of stone columns was formed with 75-mm diameter stone columns. Table 4 gives the complete program of the load tests conducted in the present study.

Parametric Studies

Load tests were conducted on OSC and the geosynthetic ESCs to directly compare their relative performance. As the confinement effect is highly dependent on the diameter of the stone column, three different diameters of stone columns viz. 50, 75, and 100 mm were considered for the present study. The other parameter considered was the modulus of the geosynthetic used for the encasement; this was varied by using different types of geosynthetics for the encasement. In order to investigate the presence of neighboring stone columns, load tests were performed on a group of stone columns installed in the clay bed prepared in the large test tank. Group of 12 numbers of 75-mm diameter stone columns were installed in a triangular grid at a center to center spacing of 150 mm (twice the diameter). The loading was applied through a circular loading plate of diameter such that it just inscribes the central three stone columns among the 12 stone columns, as shown in Fig. 5(a). The diameter of the loading plate was 248.2 mm for the present case. This pattern of loading on the group of stone column was followed as per the guidelines given in IS 15284 [Indian Standard (IS) 2003]. The loading was applied by controlled displacement at a rate of 1.2 mm/min. The load tests on the group of stone columns were performed on the OSCs and stone columns encased in woven and nonwoven geotextile.

In order to measure the pressure transmitted to the stone column area and clay soil area, pressure cells (having accuracy of 0.1

Table 4. Program of Load Tests on Stone Columns

Test description		Diameter of the stone column 50 mm	75 mm	100 mm	Total number of tests
Loading on clay bed		√	√	√	3
Test on OSC		√	√	√	3
Tests on ESCs with different geosynthetic for encasement	Nonwoven geotextile	√	√	√	3
	Woven geotextile	√	√	√	3
	Soft grid–1	—	√	—	1
	Soft grid–2	—	√	—	1
Load tests on group of stone columns			√		3

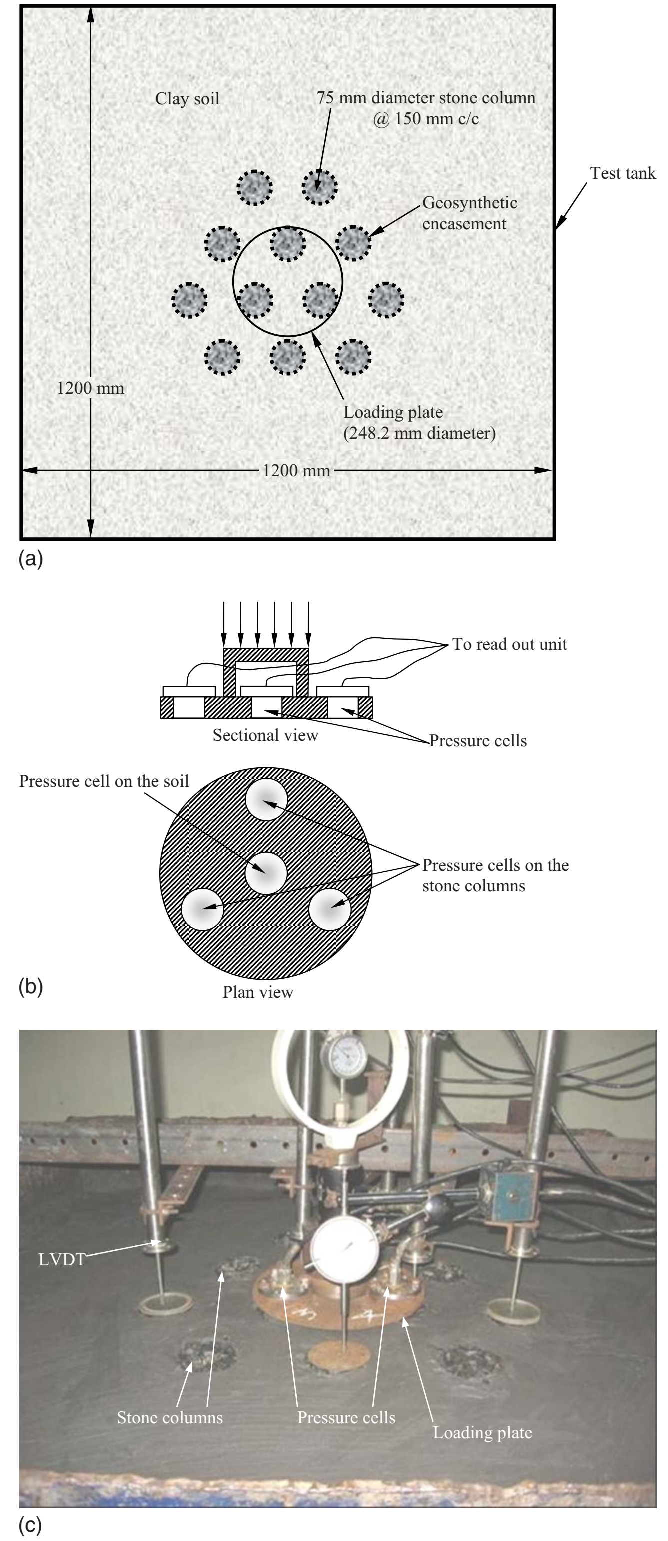

Fig. 5. Load Tests on a group of stone columns. (a) Plan view of a group of stone columns (area replacement ratio among group=0.23); (b) schematic of the loading plate fitted with pressure cells; and (c) photographic view of load test.

kPa) were fitted to the loading plate. Four number of pressure cells were fixed in the loading plate in such a way that the pressure sensing surface of the pressure cells were flush with the loading surface of the loading plate. The three pressure cells were positioned to lie directly in contact with the stone columns in order to measure the pressure developed in the stone columns. Another pressure cell was positioned at the center of the loading plate to measure the pressure developed in the clay soil [Figs. 5(b and c)]. From these tests the stress concentration factors on the individual stone columns and the stress ratio between the stone column and the intervening clay soil were observed. The load test on the group of stone column also led to quantify the improved bearing capacity of the clay bed due to the inclusion of stone columns.

Results and Discussions

Effect of Geosynthetic Encasement

The pressure-settlement responses observed from load tests on clay bed, single OSCs and single ESCs of different diameters encased in nonwoven and woven geotextile are shown in Figs. 6 and 7, respectively. These results were obtained by loading with a loading plate of double the diameter of the column or the equivalent area in the case of clay soil. This was done so as to determine the improved bearing capacity of the clay bed due to the stone column inclusions. The loading on the OSCs shows a clear failure indicating ultimate load (there is no significant raise in the pressure carried by the stone column beyond 10-mm settlement), while the ESCs did not show any signs of failure even at large settlement levels. The pressure on the ESCs corresponding to 10-mm settlements is found to be three to five times greater than that on the OSCs. The ESCs behaved like semirigid flexible piles. The failure of ESCs was not observed even at a settlement of 50 mm (i.e., 8.33% of the column length) in the present studies. In the case of ESCs, the compression of the stone column was mainly due to the radial elongation of the geosynthetic encasement caused by the bulging of the stone aggregates.

Theoretical Predictions

The maximum load that can be applied on the OSC treated clay bed was obtained by using the method given in IS:15284 [Indian Standard (IS) 2003]. The total load that a stone column can bear is the sum of (1) load capacity of the stone column resulting from the resistance offered by the surrounding soil against its lateral deformation (bulging) under axial load and (2) direct bearing sup-

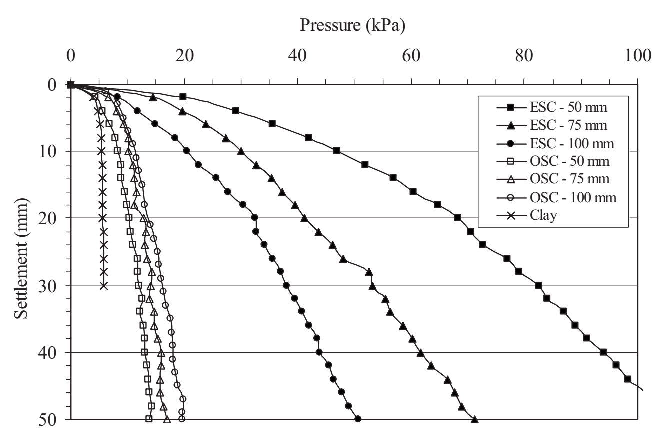

Fig. 6. Pressure-settlement responses of single OSCs and ESCs (nonwoven)

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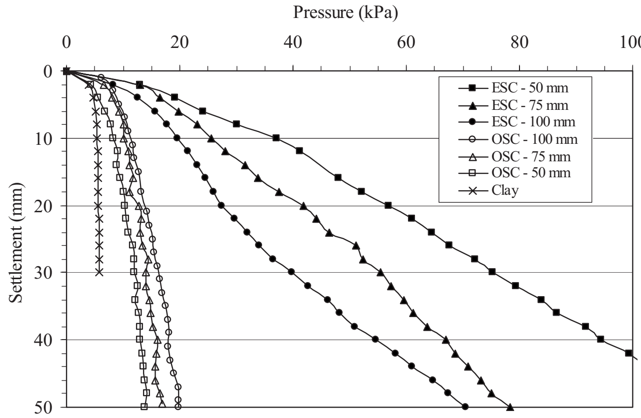

Fig. 7. Pressure-settlement responses of single OSCs and ESCs (woven)

port provided by the intervening soil between the columns. The surcharge effect on the load capacity of the stone columns was taken as 0, as there was no surcharge.

The maximum pressure on OSCs, i.e., limiting axial stress (σ_v) on the stone column is given by Eq. (1) [Hughes et al. 1975; Indian Standard (IS) 2003]

$$\sigma_v = (\sigma_{ro} + 4c_u)Kp_{\text{col}} \quad (1)$$

In the above, σ_{ro} is the initial effective radial stress computed at an average depth of twice the diameter of the column. $Kp_{\text{col}} = \tan^2(45+\phi/2)$, where φ is the angle of internal friction of the stone aggregate. The limiting axial stress on the column was estimated by assuming a K_o of 1.0 for the soft soil and using the properties of the soft clay and the aggregate reported earlier in the paper. The bearing support offered by the clay soil in contact with the loading plate was obtained as $q_{\text{safe}} = c_u \times N_c / \text{FS}$ [the bearing capacity factor N_c was conservatively taken as 5.14 as the friction angle of soil was zero and the factor of safety (FS) is considered as 2.5].

The total limiting load on the stone column was computed and was compared with the maximum load (corresponding to settlement of 50 mm) obtained from the experiments for different diameters of the OSC. The comparison between the experimental results and the analytical values matched very well as shown in Fig. 8. This consistent comparison for all the diameters indicates

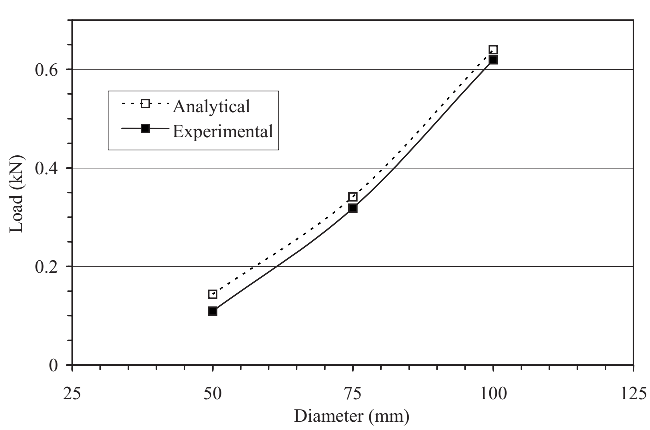

Fig. 8. Comparison of analytical and experimental results from tests on single OSCs

the uniformity in the preparation of clay bed and the stone columns in the tests. Some of the tests were repeated once again in order to verify the consistency of the data. The comparison was found to be excellent with less than 5% variation in the results between the different tests.

A simple analytical model based on the hoop tension theory was used to predict the vertical pressure on the ESCs (Van Impe 1989; Ayadat and Hanna 2005). It is proven that the bulging of the stone columns occurs predominantly in the top portion over a height equal to about 4 times the diameter (Greenwood 1970; Hughes et al. 1975). Accordingly, the predominant vertical strains (caused by the radial bulging) in the stone columns could be assumed (for simplification) to be due to the compression in this zone of the column. Assuming the stone aggregate to deform without volume changes, the hoop strain ε_c (circumferential strain) in the geosynthetic is calculated from the vertical strain (i.e., axial strain ε_a) using the relation (Henkel and Gilbert 1952)

$$\varepsilon_c = \frac{1 - \sqrt{1 - \varepsilon_a}}{\sqrt{1 - \varepsilon_a}} \quad (2)$$

The vertical strain in the column was estimated based on the measured surface displacements divided by a column height equal to four times the diameter. The tensile load corresponding to this strain in the geosynthetic is obtained from the load strain response from the wide width tension tests performed on the seamed geosynthetic. Knowing the tensile load (T) in the geosynthetic, the additional lateral confining stress p_c (hoop compression) exerted by the geosynthetic is calculated as

$$p_c = \frac{2T}{d} \quad (3)$$

in which d is the diameter of the column. As the diameter increases, the additional confining pressure decreases as evident from Eq. (3). The vertical stress on the ESC can be calculated using the relation

$$\sigma_v = (\sigma_{ro} + 4c_u + p_c) \times Kp_{\text{col}} \quad (4)$$

The vertical stress on the ESCs corresponding to a vertical settlement of 50 mm for the different stone columns was calculated and compared with those obtained from the experimental results for the case of nonwoven geotextile (Fig. 9(a)). A reasonably good agreement is obtained between the two as could be seen from the figure. The experimental results from ESCs were also compared with the predictions by Eq. 11 of Ayadat and Hanna (2005) by considering a value of α equal to 0.15 (which corresponds to modulus of stone column 41,600 kPa). The results of the present experiments reasonably match with their predictions as shown in Fig. 9(b).

Influence of the Diameter of the Stone Column

It could be seen in Figs. 6 and 7 that the responses of the OSC of different diameters are almost the same. The ESCs have developed much higher pressures compared to the OSCs. The pressures developed in the ESC decrease with increase in diameter of the column. This is in line with the findings by Murugesan and Rajagopal (2006a,b, 2007a) based on numerical and laboratory model tests on stone columns installed in unit cells. The pressure-settlement response of the columns encased in woven geotextiles have also shown similar trend of decreasing pressures with increasing diameter. Eq. (3) shows that the additional confining

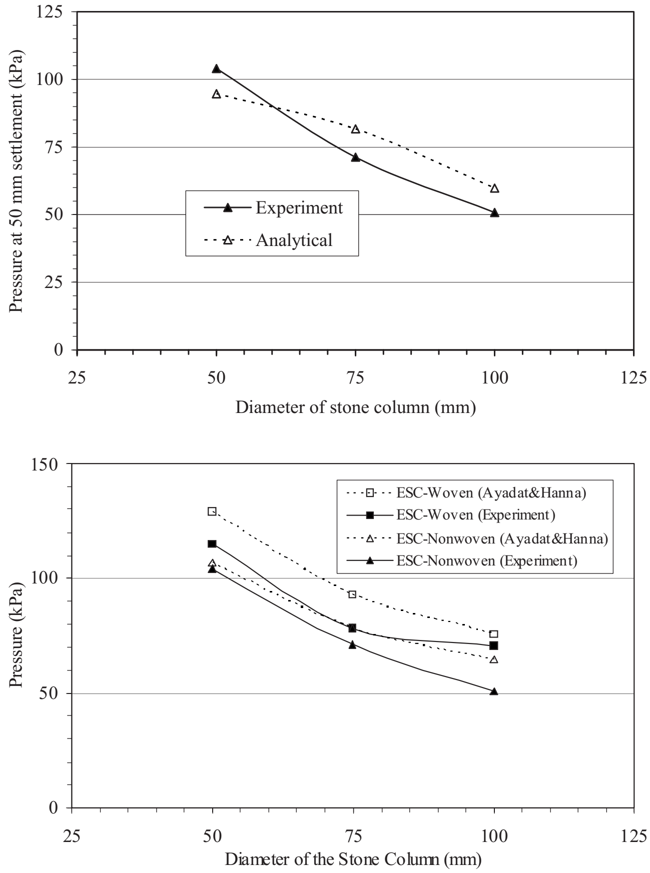

Fig. 9. Comparison of experimental results with analytical and other theoretical results. (a) Comparison of analytical and experimental results for ESCs; (b) comparison of experimental results for ESCs with Ayadat and Hanna (2005).

pressure due to the encasement is inversely proportional to the diameter of the stone column. Hence the effect of encasement decreases with increase in diameter of ESCs.

Influence of Modulus of Geosynthetic Encasement

Fig. 10 shows a comparison of pressure-settlement responses of 75-mm diameter stone columns with different types of encasements. It can be observed that the stiffness of the ESCs increases with the increase in the secant modulus of the geosynthetic used for encasement. The improved performance due to the encasement can be attributed to the enhancement of overall stiffness of the columns due to larger confining stresses developed in the stone columns. The vertical pressure (load on the loading plate divided by the area of the plate) corresponding to 50-mm settlement in the stone column encased in various geosynthetic is plotted against the 5% secant modulus of the geosynthetic (Fig. 11). From the figure it could be seen that the increase in the modulus of geosynthetic results in increase of vertical pressure on the column. The hoop stresses in the geosynthetic lead to increase in confining pressures in the stone columns as described by Van Impe (1989), Alexiew et al. (2005), Murugesan and Rajagopal (2006a), etc. Hence, geosynthetics with higher modulus will induce larger confining pressures leading to stiffer and stronger response of the stone columns.

Hoop Strain Variation in the Encasement

The hoop strains developed along the height of the stone columns encased with nonwoven geotextiles were measured using strain gauges fixed at 100-mm vertical intervals. The hoop strains developed in 75-mm diameter stone column is shown in Fig. 12. It is observed that the hoop strains are higher near the top of the stone column where predominant bulging takes place. The hoop strains are observed to decrease with depth in the stone column. This is because of lesser straining of stone columns at deeper depths. It could be observed that the magnitudes of hoop strains developed in the encasement are less than 0.5%. The strains developed in the strain gauges were within the elastic limit during unloading the strain gauges could recover some strains as evidenced in Fig. 13. Due to the harsh environment around the stain gauges, the Strain Gauge 3 became nonfunctional during the test. The pattern of hoop strain variation is similar to that reported by Murugesan and Rajagopal (2006a,b) based on numerical simulations. The low hoop strains measured during the tests are thought to be due to the straining occurring in the seam rather than in the parent material. Efforts were made to measure the strain at the seam by fixing the strain gauges at the seam. However, the efforts were not successful because of early separation of strain gauges from the seam.

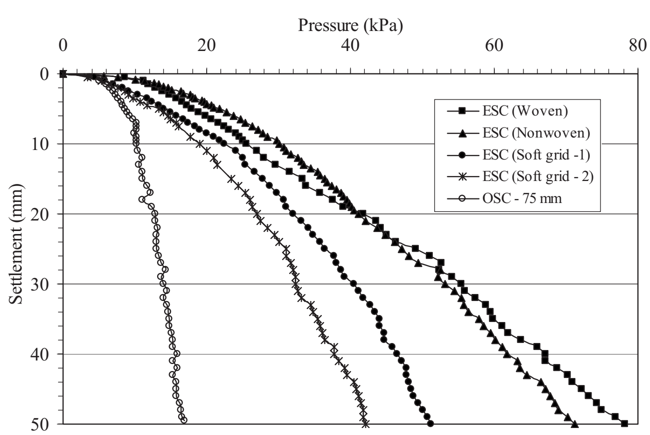

Fig. 10. Responses of 75-mm diameter stone columns encased in different types of geosynthetics

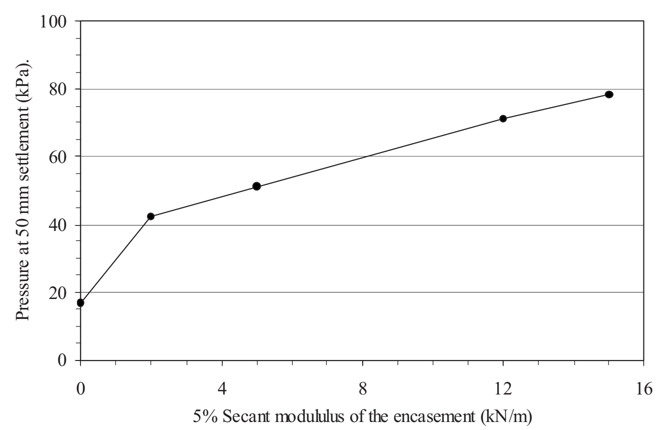

Fig. 11. Influence of the modulus of encasement on the performance of the encased column (75-mm diameter)

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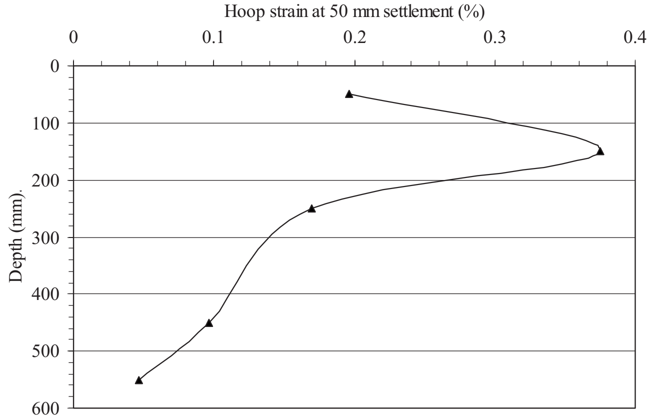

Fig. 12. Hoop strain variation with depth in 75-mm diameter ESC (nonwoven) (hoop stains corresponding to 50-mm settlement in the column)

Results from Load Tests on Group of Stone Columns

The stress concentration factor on the stone column is defined as the ratio between the pressure on the stone column and the total pressure applied over the stone column treated soil. This aspect was experimentally investigated through load tests on group of stone columns installed in the large test tank as described in the Parametric Studies section. Twelve 75-mm diameter stone columns were installed in a triangular pattern at a spacing of 150-mm center to center which forms an area replacement ratio of 0.227. The central three stone columns were loaded through a 10-mm thick loading plate having a diameter of 248.2 mm which just inscribes the three stone columns as shown in Fig. 5(a). The pressures on the stone column and the soft clay soil were measured using pressure cells fitted to the loading plate. The load tests on the group of stone columns were performed on the OSC, and stone column encased in woven and nonwoven geotextile. From these tests the improved performance of the ESCs over OSCs was also quantified.

Fig. 14 shows the pressure-settlement responses from the load test on the group of stone columns with and without encasement. The response of the group of ESCs encased in the woven geotextile shows a linear behavior without any significant sign of failure even for a settlement of up to 40 mm. On the other hand, the group of OSCs has shown a clear sign of failure in their pressure-

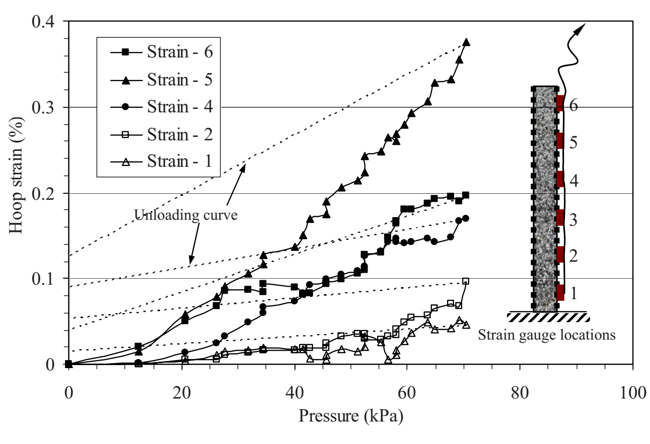

Fig. 13. Hoop strain in the encasement versus applied pressure on 75-mm diameter nonwoven ESC

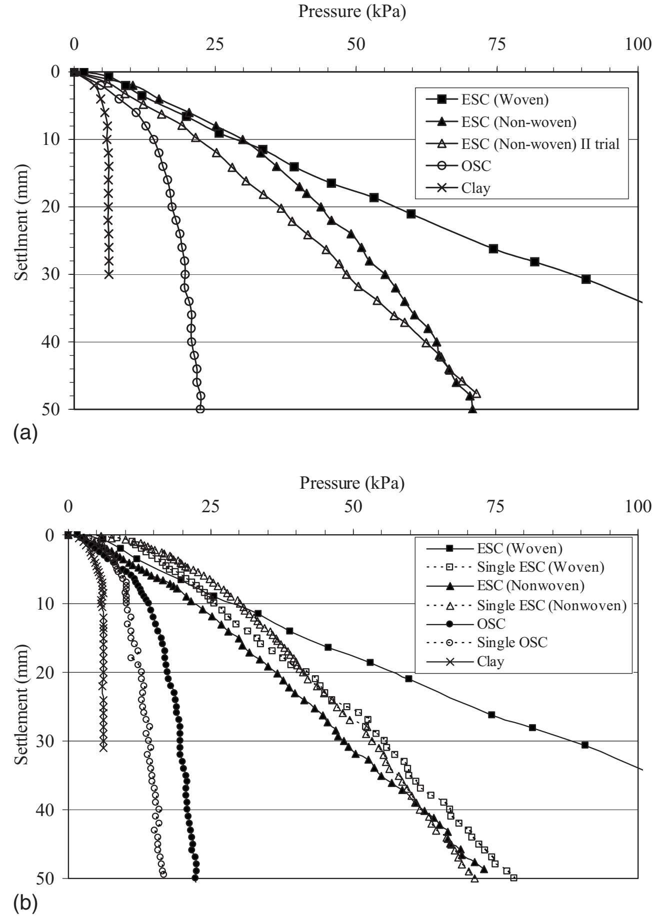

Fig. 14. (a) Pressure-settlement responses of a group of stone columns (75-mm diameter); (b) comparison of pressure-settlement responses of a group of stone columns and the corresponding responses from tests on isolated columns

settlement response. This may be due to the excessive bulging of the stone column due to loading. In the case of ESC with nonwoven encasement, the response is softer than that of the woven geotextile encasement owing to the lesser modulus of the geotextile material itself. There is a reasonably good match between the repeated tests. This shows the consistency in different tests. By comparing the responses of OSCs and the ESCs it could be observed that the bearing capacity increases by three to five times due to the encasement. In general, the ESCs show strain hardening behavior whereas the OSCs show plastic failure beyond a certain load limit.

The pressure on the individual stone column was measured by the pressure cells attached to the loading plate. The stress concentration factor on the stone column is calculated as the ratio between the pressures on the stone column and the total pressure on the loading plate. The stress concentration factor versus the settlement in the loading plate is shown in Figs. 15(a and b). From the figures it could be observed that the stress concentration factor on the ESC is about five. The ratio of the stress transferred to the clay soil is only about 0.1 to 0.6 times that of the total pressure on the loading plate. Moreover it could be observed from the Fig. 15(b) that the stress on the intervening clay soil between the ESCs is less than that corresponding to that with OSCs. This indicates that the ESCs behave similar to semi rigid columnar elements, which carry higher percentage of total load and transferring smaller fraction to the surrounding soil. The stress concentration

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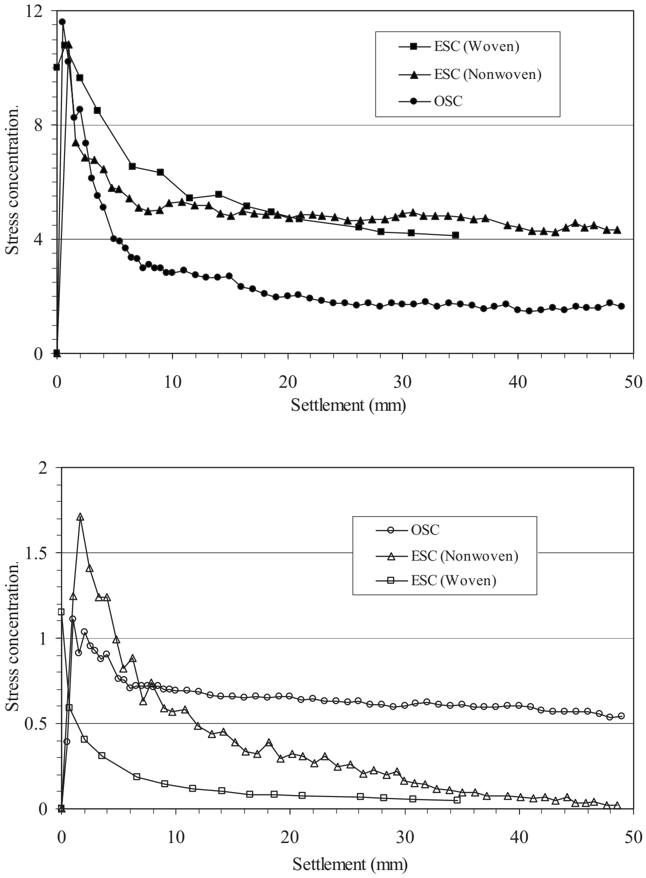

Fig. 15. (a) Stress concentration on the stone columns with settlement (load test on group of columns—75-mm diameter); (b) stress concentration on the clay surface with settlement (load test on group of columns—75-mm diameter)

factor is only 2 for the OSCs after the final mobilization of the system at higher settlements, which occurs at settlements of about 2.5 to 3.0% of the column length. The stress transferred to the intervening clay soil between OSCs is higher compared to that on the clay soil between ESCs. The peaks in the stress concentration curves at initial pressure levels were thought to be due to the quick loading through strain controlled displacement of the loading plate.

Guidelines for the Design of ESCs

Based on the results from the current research work (strain controlled load tests), the following guidelines are developed for the design of geosynthetic ESCs. The bearing support from the soft soil is conservatively ignored in this methodology as the ESCs are specially suited for the case of extremely soft soils (with shear strengths of less than 20 kPa).

1. For the given pressure loading p_o from the structure, suitable spacing (s) and diameter (d) of the stone columns are chosen. A typical unit cell consisting of a stone column and the contributing surrounding soil is considered among the grid of stone columns. The load on the unit cell area is assumed to be carried fully by the stone column alone in the unit cell.

   Load on the stone column=applied pressure $p_o \times$ Area of the unit cell A. Area of the unit cell $A = \pi \times (0.525s)^2$: for triangular grid; $= \pi \times (0.564s)^2$: for square grid where s is the spacing of the stone column. Load on stone column=load on the unit cell=$p_o \times A$. $\therefore$ Pressure on the stone column=load on the unit cell/area of the stone column A_c.
2. The limiting stress on an OSC without encasement is computed by using the Eq. (1).
3. The additional confinement p_c required is calculated as

$$p_c = \frac{(p_o - \sigma_v)}{K p_{\text{col}}} \tag{5}$$

4. The corresponding hoop tension force in the encasement (T) can be estimated as

$$T = \frac{p_c d}{2} \tag{6}$$

5. The hoop strain ε_c in the encasement corresponding to the permissible settlement (δ) in the stone column is computed using Eq. (2), in which ε_a is the axial strain in the stone column. This value can be evaluated from the surface settlement of the stone column treated ground δ (i.e., permissible settlement)

$$\varepsilon_a = \frac{\delta}{4 \times d} \tag{7}$$

   The effect of the surface loads was found to cause strains over a height of four times the diameter of the stone column as discussed in "Theoretical Predictions" section.
6. A suitable geosynthetic that can develop the long term allowable design tensile strength, T at a strain level of ε_c can be chosen for the encasement.

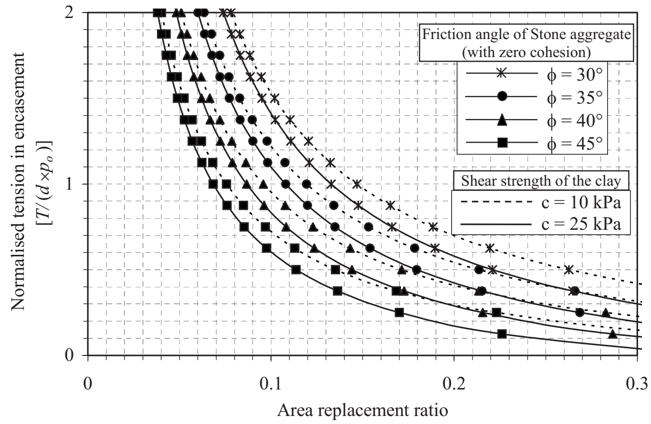

Fig. 16. Design charts for the geosynthetic ESCs

Design Charts

Based on the above design procedure developed for the ESCs, design charts have been prepared in nondimensional form applicable for a range of realistic soil parameters (Fig. 16). The area replacement ratio in this figure is calculated from the spacing, s and diameter of the stone column, d as

$$\text{Area ratio} = 0.907 \times \left(\frac{d}{s}\right)^2 \quad \text{for triangular grid} \tag{8}$$

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$$\text{Area ratio} = 0.786 \times \left(\frac{d}{s}\right)^2 \text{ for square grid} \qquad (9)$$

For designing geosynthetic ESC, the design chart given in Fig. 16 can be used with following steps:

1. For the assumed spacing and diameter of the stone column, the area ratio is calculated.
2. For the properties of clay soil (c), friction angle of the stone aggregate (φ) and the area ratio, normalized tensile force required for the encasement is read from the chart. For other soil properties, linear interpolation may be used.
3. From the normalized tension in the geosynthetic encasement, the tensile strength of the geosynthetic required is estimated for the maximum applied pressure.
4. From the settlement criterion, the axial strain in the stone column is calculated. The hoop strain corresponding to this axial strain is computed.

A suitable geosynthetic having long term allowable design tensile strength (including due allowance for the corresponding long-term stain after creep) is then selected for the encasement.

Conclusions

The results of the testing program give some important insight into the performance of the ESCs. The trends obtained in these laboratory tests are in good agreement with the results reported in the literature. The major conclusions that can be drawn from this research work are as follows:

1. The load capacity and stiffness of the stone column can be increased by all-round encasement by geosynthetic. The ESCs exhibit stiffer and stronger response. They do not exhibit significant strain softening response beyond the peak load. On the other hand, the OSCs exhibit softer response with significant strain softening.
2. The benefit of encasement decreases with increase in the diameter of the stone columns due to reduction of hoop tension in the encasement. The performance of ESCs of smaller diameters is superior to that of larger diameter stone columns for the same encasement because of mobilization of higher confining stresses in smaller diameter stone columns. The higher confining stresses in the column leads to higher stiffness of smaller diameter encased columns.
3. The elastic modulus of the geosynthetic encasement plays an important role in enhancing the capacity and stiffness of the encased columns. The confining pressures generated in the stone columns are higher for higher modulus of encasements.
4. The hoop strains in the geosynthetic encasement are highest near the top surface and decrease with depth. The pattern of variation of hoop tension forces closely follows that of the bulging of the column.
5. The encased columns have much higher stress concentration compared to that of ordinary columns. The stress concentration on the stone column increases with increase in the modulus of the encasement. This result shows that the encased columns act like semirigid piles.

Acknowledgments

The writers are thankful to the Ministry of Human Resources Development, Government of India, New Delhi for the financial assistance to the project titled "Investigations on Modern Technologies for Construction of Road/Rail Embankments on Soft Clay Soils" at IIT Madras. The writers acknowledge the help rendered by Mr. M. Gunasekaran at various stages during this work. The writers are thankful to the anonymous reviewers for their meticulous review and suggestions for improving the presentation in the paper.

Notation

The following symbols are used in this paper:

A = area of unit cell considered;
A_c = area of cross section of the stone column;
c_u = cohesion of undrained shear strength of the clay soil;
d = diameter of the stone column;
Kp_{col} = coefficient of passive earth pressure for the stone aggregate;
N_c = bearing capacity factor;
p_c = confining pressure by the geosynthetic encasement;
T = force developed in the geosynthetic;
δ = permissible settlement in the stone column;
ε_a = axial strain in the stone column;
ε_c = circumferential strain in the geosynthetic encasement;
σ_{ro} = initial effective radial pressure at depth of twice the diameter of the column; and
σ_v = vertical pressure.

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