Effect of Sand Columns on the Undrained Load Response of Soft Clays

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Abstract: When sand columns are used as vertical drains in soil improvement schemes, the possible reinforcing role that these columns can play in regards to improving the bearing capacity is usually neglected in design. The objective of this paper is to evaluate the degree of improvement in the mechanical properties of soft clays in practical applications involving the use of sand drains or sand columns in clayey soils. For this purpose, 32 isotropically consolidated undrained triaxial tests were performed on normally consolidated kaolin specimens. The parameters that were varied were the diameter of the sand columns, the height of the columns, the type of columns (geotextile encased versus nonencased), and the effective confining pressure. Test results indicated that sand columns improved the undrained strength significantly even for area replacement ratios that were less than 18%. The increase in undrained strength was accompanied by a decrease in pore pressure generation during shear and an increase in Young's modulus. The drained shear strength parameters were found to be relatively unaffected by the sand column reinforcement, except for fully penetrating columns with high area replacement ratios.

DOI: 10.1061/(ASCE)GT.1943-5606.0000328

CE Database subject headings: Soil stabilization; Stone columns; Sand, filter; Geosynthetics; Clays.

Author keywords: Ground improvement; Stone columns; Sand drains; Geotextiles.

Introduction

For structures that involve relatively light to moderate loads that are distributed over large areas, the costs associated with the use of deep foundations to bypass soft clay layers may be prohibitive. In such cases, granular columnar inclusions have been successfully used to improve the mechanical properties of the soft clay. Columnar inclusions generally include: (1) sand drains or sand columns which are used to accelerate the rate of construction of structures or embankments and (2) stone or gravel columns which are associated with vibroreplacement operations which are aimed at replacing a percentage of the soft clay with stiff granular columns.

When sand columns are used as vertical drains to accelerate the rate of construction, the possible positive reinforcing role that these columns can play in regards to improving the bearing capacity is usually neglected in design. While very few researchers have attempted to investigate the role of sand drains in improving the undrained bearing capacity of soft clays, many researchers used sand columns to investigate the role of stone columns in reinforcing soft clays. In the majority of these research studies, the method of construction of the columnar inclusions resembled the construction of sand drains rather than the construction of stone columns. Examples of experimental studies in which sand columns were used to model the behavior of stone columns include the work done by Hughes and Withers (1974), Juran and Guermazi (1988), Juran and Riccobono (1991), Narasimha Rao et al. (1992), Muir Wood et al. (2000), Sivakumar et al. (2004), McKelvey et al. (2004), Ayadat and Hanna (2005), and Black et al. (2006, 2007). In some studies, single sand columns were tested by direct loading of the columns while in others, both single sand columns and column groups (up to four columns per group) were loaded together with the surrounding clay using either model foundations or top plates of typical triaxial cells.

The majority of the studies mentioned above were conducted in large one-dimensional (1D) loading chambers which do not allow for the control of drainage in the soil specimens during loading. Since most of these tests generally entailed partial drainage, the analysis of the test results was restricted to improvements in the general load carrying capacity of the sand column or the clay-sand column hybrid system. Juran and Riccobono (1991), Sivakumar et al. (2004), and Black et al. (2006, 2007) performed tests under full triaxial conditions in which the loading rate and the drainage conditions were controlled during shear. In some tests, the reinforced clay samples were sheared slowly to establish drained conditions, but more generally, soil specimens were sheared undrained. Baumann and Bauer (1974), Alamgir et al. (1996), and Murugesan and Rajagopal (2006) performed finite-element analyses to investigate the effect of granular columns on the load deformation response of reinforced clay.

Results of the experimental and finite-element investigations listed above indicate that the mode of failure of clay specimens that are reinforced with circular single sand columns is characterized by lateral bulging of the sand column particularly in the top four to five diameters along the height of the column. Specimens

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Note. This manuscript was submitted on February 12, 2009; approved on January 17, 2010; published online on January 25, 2010. Discussion period open until February 1, 2011; separate discussions must be submitted for individual papers. This paper is part of the ***Journal of Geotechnical and Geoenvironmental Engineering***, Vol. 136, No. 9, September 1, 2010. ©ASCE, ISSN 1090-0241/2010/9-1263–1277/$25.00.

J. Geotech. Geoenviron. Eng. 2010.136:1263-1277.

with short partially penetrating columns appeared to fail below the reinforced portion of the clay, causing no significant improvement in the load carrying capacity of the specimen. Based on the above observations, several researchers proposed the idea of the "critical column length" that is between four to eight times the diameter of the column beyond which the sand column will not improve the capacity of the clay (Hughes and Withers 1974; Narasimha Rao et al. 1992; Muir Wood et al. 2000; McKelvey et al. 2004).

For fully penetrating sand columns, results from experimental studies indicate that the insertion of sand columns in soft clays increases the load carrying capacity of the soft clays, reduces the settlement, and decreases the generation of excess pore-water pressure during undrained loading. The extent of improvement in the above factors was shown to be dependent on the undrained shear strength of the clay, the angle of internal friction of the column material, and the geometric characteristics of the sand columns (diameter and spacing). Limited results involving tests conducted on both single columns and column groups indicate that for undrained loading, the relative increase in strength due to the presence of sand columns is independent of the column configuration (no column group effect) and is only dependent on the area replacement ratio of the reinforcement (Black et al. 2007).

In some field applications involving sand drains, geosynthetic filter materials are used to separate the sand columns from the surrounding clay. Malarvizhi and Ilamparuthi (2004), Ayadat and Hanna (2005), and Murugesan and Rajagopal (2006) studied the effect of encapsulating sand columns with geofabrics of different strengths and stiffnesses. Although the main focus of these experimental research studies was to investigate the behavior of stone columns and not sand drains, the results obtained indicated that encasing the columns with geotextiles or geogrids provided additional lateral support to the granular column and reduced the bulging of the column during loading, thus increasing the stiffness and bearing capacity of the clay-sand column system.

In this paper, a comprehensive testing program is implemented to assess the impact of sand columns on the undrained load response of soft clays in practical application involving the use of sand drains or sand columns in clayey soils. The experimental program involves performing isotropically consolidated undrained (CU) triaxial tests with pore pressure measurement on soft clay specimens that were reinforced with sand columns. The parameters that were varied in the program were (1) the area replacement ratio, A_c/A_s, defined as the ratio of the cross sectional area of the sand column (A_c) to the cross-sectional area of the specimen (A_s); (2) the column penetration ratio, H_c/H_s, defined as the ratio of the height of the sand column (H_c) to the height of the specimen (H_s); and (3) the confinement of the sand column with a geosynthetic fabric. All tests were conducted at three effective confining pressures to isolate the effect of confinement on the degree of improvement in the mechanical properties of the sand column-clay system (including undrained strength and Young's modulus), but more importantly to characterize and compare the effective Mohr-Coulomb failure envelopes for control clay specimens and specimens that were reinforced with sand columns.

Laboratory Testing Program

In total, 32 isotropically CU triaxial tests were performed on one-dimensionally consolidated kaolin specimens having a diameter of 7.1 cm and a length of 14.2 cm. Tests were conducted on control specimens and specimens that were reinforced with single sand columns having diameters of 2 and 3 cm with column penetration ratios H_c/H_s of 0.5, 0.75, and 1. The 2- and 3-cm diameter columns represent area replacement ratios A_c/A_s of 7.9 and 17.8%, respectively. All sand columns were installed in predrilled holes in the center of the clay specimens and were installed in a nonencased state or with a geosynthetic encasement. All specimens were back pressure saturated using a back pressure of 310 kPa and isotropically consolidated under effective confining pressures of 100 kPa, 150 kPa, or 200 kPa. In all tested specimens, the measured "B" value (Skempton 1954) was greater than 0.96 indicating an adequate degree of saturation. Samples were then sheared in undrained conditions at a strain rate of 1% per hour. All tests were terminated at a maximum axial strain of about 10%. The program of testing is summarized in Table 1.

Specimen Preparation

Kaolin clay powder was mixed with water at a water content of 100% (i.e., 1.8 times its liquid limit) to form a slurry. The slurry was then poured into custom-fabricated consolidometers in preparation for 1D consolidation. A detailed description of the sample preparation and testing procedure is presented below.

One-Dimensional Consolidometers

Four 1D consolidometers were fabricated for the purpose of consolidating the kaolin slurry (Fig. 1). Each consolidometer consisted of a PVC pipe segment with a height of 35 cm, an internal diameter of 7.1 cm, and a wall thickness of 0.1 cm. The PVC pipe segment was cut longitudinally into two-halves to function as a split mold [Fig. 2(b)], thus eliminating the need for extruding the soil sample after consolidation. The two PVC sections were held in place using high-strength duct tape to prevent leakage of slurry and to ensure that lateral strains are negligible during 1D consolidation [Fig. 2(a)]. The advantage behind using a split PVC pipe was to ensure that an undisturbed, relatively soft, and normally consolidated clay specimen could be obtained and removed with minimal disturbance [Fig. 2(c)]. A porous stone and filter paper were used to provide a freely draining boundary at the top and bottom of the soil specimen. At its upper end, the soil specimen was loaded with a system consisting of dead weights similar to those used in 1D consolidation tests. The dead weights were seated on a circular steel plate that transferred the load to the top of the soil specimen through a circular steel rod. A perforated circular steel piston with a diameter of 7.1 cm was fixed to the bottom of the steel rod to act as a loading plate which transmitted the load to the slurry.

Consolidation of Kaolin Slurry

The slurry was poured into a consolidometer and consolidated under a vertical effective stress of 100 kPa. The clay (with an initial height of 35 cm) was allowed to consolidate under its own weight for a period of 4 h. During 1D consolidation, drainage was allowed from both ends of the sample through the top and bottom porous stones. Dead weights were then added in stages to the top of the sample, with each weight applied for a specified time period according to the loading sequence shown in Table 2. The consolidation time periods presented in Table 2 were estimated based on a trial and error procedure to ensure that: (1) equilibrium

Table 1. Laboratory Testing Program and Results

Test number	Effective confining pressure $(\sigma'_3)_o$ (kPa)	Diameter of sand column (cm)	Area replacement ratio A_c/A_s (%)	Height of sand column (cm)	Column penetration ratio (H_c/H_s)	Column height to diameter ratio (H_c/D_c)	Undrained shear strength (kPa)	Excess pore-water pressure (kPa)	$(E_{sec})_{1\%}$ at 1% axial strain (kPa)	Increase in undrained shear strength (%)	Reduction in excess pore pressure (%)
1	100	0	0	0	0	—	32.3	61.3	4,150	—	—
2		2	7.9	7.1	0.50	3.55	32.5	59.9	4,166	0.6	2.9
3		2	7.9	10.65	0.75	5.32	35.2	57.3	4,220	9.0	6.5
4		2	7.9	14.2	1	7.10	36.6	51.2	4,390	13.3	16.5
5		2 (ESC)	7.9	7.1	0.5	3.55	32.8	58.8	4,187	1.5	4.1
6		2 (ESC)	7.9	10.65	0.75	5.32	37.2	58.0	4,762	15.1	5.4
7		2 (ESC)	7.9	14.2	1	7.10	52.0	58.9	5,132	61.0	3.9
8		3	17.8	10.65	0.75	3.55	38.9	48.9	4,597	20.4	20.2
9		3	17.8	14.2	1	4.73	56.7	42.7	5,853	75.5	30.3
10		3 (ESC)	17.8	7.1	0.5	2.36	36.0	59.2	4,280	11.5	3.5
11		3 (ESC)	17.8	14.2	1	4.73	64.8	42.8	7,150	100.6	30.2
12	150	0	0	0	0	—	42.1	95.1	6,092	—	—
13		2	7.9	10.65	0.75	5.32	48.2	88.9	6,100	14.5	6.5
14		2	7.9	14.2	1	7.10	50.3	87.8	6,368	19.5	7.7
15		2 (ESC)	7.9	7.1	0.5	3.55	42.3	89.4	4,920	0.5	6.0
16		2 (ESC)	7.9	10.65	0.75	5.32	50.9	85.4	6,402	20.8	10.2
17		2 (ESC)	7.9	14.2	1	7.10	58.5	76.8	6,093	39.0	19.2
18		3	17.8	7.1	0.5	2.36	46.0	87.2	6,101	9.3	8.3
19		3	17.8	10.65	0.75	3.55	56.8	78.1	6,697	34.9	17.9
20		3	17.8	14.2	1	4.73	73.9	65.2	8,624	75.5	31.4
21		3 (ESC)	17.8	7.1	0.5	2.36	46.8	92.7	6,574	11.1	2.5
22		3 (ESC)	17.8	14.2	1	4.73	79.4	67.8	8,045	88.7	28.7
23	200	0	0	0	0	—	55.1	130.9	7,637	—	—
24		2	7.9	10.65	0.75	5.32	60.1	120.3	7,904	9.1	8.1
25		2	7.9	14.2	1	7.10	65.8	112.1	7,996	19.4	14.4
26		2 (ESC)	7.9	7.1	0.5	3.55	58.4	119.2	7,788	6.0	8.9
27		2 (ESC)	7.9	10.65	0.75	5.32	61.9	121.1	8,144	12.4	7.5
28		2 (ESC)	7.9	14.2	1	7.10	70.9	115.7	8,228	28.6	11.6
29		3	17.8	10.65	0.75	3.55	71.2	107.8	8,983	29.2	17.6
30		3	17.8	14.2	1	4.73	92.3	89.4	10,103	67.5	31.7
31		3 (ESC)	17.8	7.1	0.5	2.36	58.2	119.7	8,121	5.6	8.5
32		3 (ESC)	17.8	14.2	1	4.73	103.3	86.5	11,407	87.4	33.9

Note: "(ESC)" indicates geosynthetic-encased sand columns and $(E_{sec})_{1\%}$=secant Young's modulus at a strain of 1%.

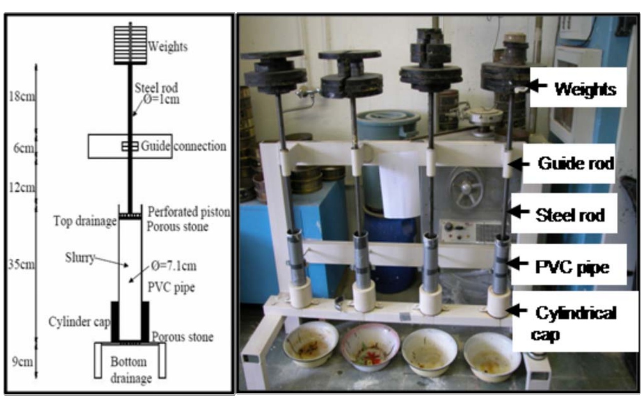

Fig. 1. Custom-fabricated 1D consolidometers

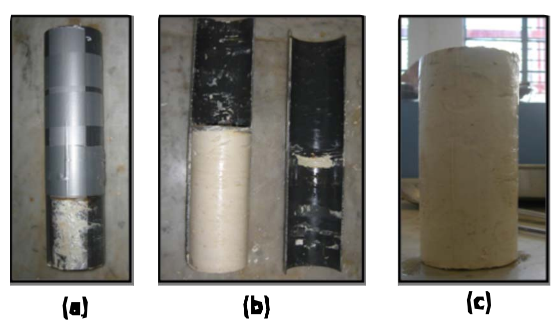

Fig. 2. (a) Sealed PVC split sections; (b) dismantling of PVC sections; and (c) kaolin specimen consolidated to 100 kPa

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Table 2. Loading Sequence during 1D Consolidation

Test information	S1[a]	S2	S3	S4	S5	S6	S7	S8	S9
Applied weight (kg)	0.5	1	2	4	8	12	20	30	40
Applied pressure (kPa)	1.3	2.5	5	10	20	30	50	75	100
Duration (h)	4	4	24	24	24	24	24	24	24

[a]S1 refers to Stage No. 1.

under the final applied load was achieved and (2) no leakage of slurry occurred in the consolidometers. The water content after consolidation was found to be relatively uniform (about 51% with standard deviation of 1%) throughout the depth of the sample. The variations of the water content and the void ratio with depth were determined by slicing a consolidated clay sample into seven pieces and determining the void ratio and water content for each slice. The variation of the void ratio and water content with depth for a typical sample is shown in Fig. 3. The variations are relatively small indicating a relatively uniform degree of consolidation in the sample.

Construction of Sand Columns

After dismantling the cylindrical kaolin specimen from the PVC pipe and trimming it to a final height of 14.2 cm, the specimen was seated in a specially designed auguring apparatus and a hole with a predetermined diameter and height was constructed (Fig. 4). For both encased and nonencased columns, the geosynthetic fabric was used to aid in the preparation of sand columns with different diameters and lengths. This was done by filling the fabric with Ottawa sand in three layers, with every layer being vibrated by means of a specifically designed and manufactured electric vibrator to achieve a relative density of about 44%. After vibration, the encased column was saturated with water to achieve a water content of about 20%. The average bulk density of the wet sand column was about 19.4 $\mathrm{kN/m^3} \pm 0.22\ \mathrm{kN/m^3}$. In clay specimens that were reinforced with encased sand columns, the columns were inserted in the predrilled holes with the fabric wrapped around the column. Fig. 5 shows the sequence of installing encased sand columns in the clay.

The construction of sand columns that were not encased with a geosynthetic fabric included a process that involved freezing (e.g., Sivakumar et al. 2004; Black et al. 2007). In this process, encased sand columns were placed in a freezer for a period of 24 h [Fig. 6(a)]. The fabric was then detached from the frozen sand using a sharp cutter [Fig. 6(b)] and the resulting nonencased sand

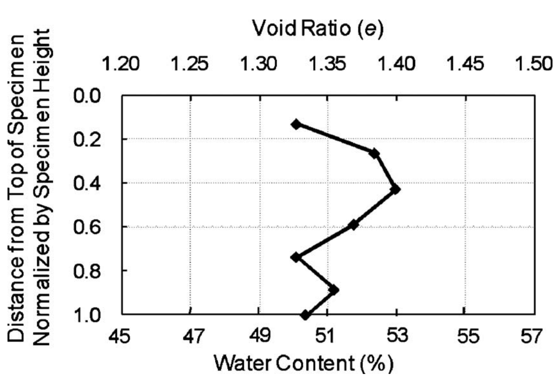

Fig. 3. Variation of water content and void ratio along the height of the sample after 1D consolidation

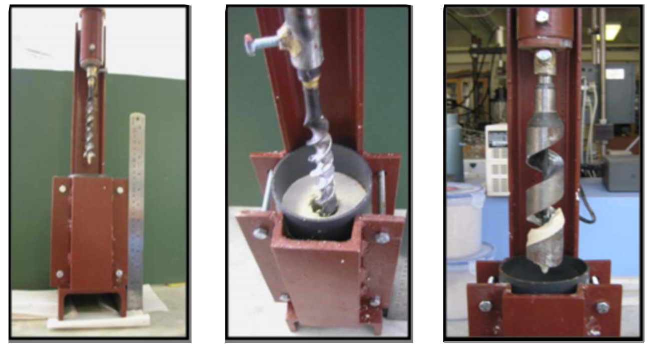

Fig. 4. Drilling holes in kaolin specimens

column was inserted in the predrilled hole (Fig. 7) and left to thaw. The idea behind using a specimen preparation process that involved freezing was to ensure that the mechanical properties of the resulting sand columns were repeatable and uniform across the different samples. The option of forming the sand columns by compaction of sand in the predrilled holes carried with it the risk of obtaining sand columns with variable densities (Sivakumar et al. 2004). Variations in the density of sand columns could result in variations in the Young's modulus and friction angle of Ottawa sand. The use of the freezing method whereby sand particles are compacted by vibration outside the kaolin specimen ensured that the density of the sand column would be uniform and repeatable.

Properties of Material

The materials used in this experimental study are kaolin clay, Ottawa sand, and a geosynthetic fabric. The properties of these materials are presented in the following sections.

Kaolin Clay

The clay used in the testing program is a kaolin clay with a liquid limit of 55.7%, a plasticity index of 22.4%, and a specific gravity of 2.53. The consolidation properties of the kaolin clay were obtained from 1D consolidation tests that were conducted on a clay specimen with a diameter of 5.08 cm and a height 1.91 cm. The specimen was trimmed from typical specimens which were consolidated from a slurry in 1D prefabricated consolidometers under a vertical effective stress of 100 kPa. Fig. 8 shows the variation of the void ratio versus the effective vertical stress represented in logarithmic scale. The void ratios in Fig. 8 represent the state of the clay specimen 24 h from the onset of loading. The virgin compression (c_c), reloading (c_r), and swelling (c_s) slopes were determined to be 0.41, 0.14, and 0.16, respectively, and the pre-

Fig. 5. Installation of encased sand column

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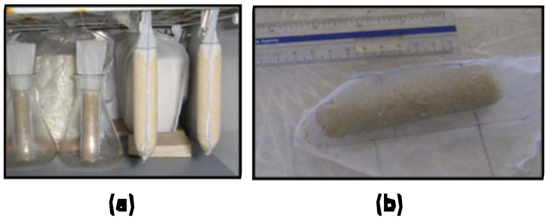

Fig. 6. (a) Frozen sand columns; (b) removal of fabric

consolidation pressure was determined to be 96 kPa using the construction method proposed by Casagrande (1936).

The undrained and effective shear strength properties of the kaolin clay were determined using isotropically CU triaxial tests with pore pressure measurement. Curves showing the variation of the deviatoric stress and the excess pore-water pressure versus axial strain at effective confining pressures of 100, 150, and 200 kPa are presented in Fig. 9(a). For all confining pressures, the deviatoric stress and the pore pressure generally increased with axial strain. Since no maximum deviatoric stresses were reached during the tests, failure was defined at an axial strain of 10%. The deviatoric stresses at failure were 64.6, 84.2, and 110.2 kPa, corresponding to $s_u/(\sigma_3')_o$ ratios of 0.32, 0.28, and 0.27, respectively, where s_u=undrained shear strength and $(\sigma_3')_o$=initial effective confining pressure. These $s_u/(\sigma_3')_o$ ratios are typical of normally consolidated kaolin clay specimens that are sheared in undrained conditions (Lin and Penumadu 2005). The pore-water pressures at failure were 61.3, 95.1, and 130.9 kPa at effective confining pressures of 100, 150, and 200 kPa, respectively, corresponding to Skempton pore pressure parameters "A_f" of 0.95, 1.12, and 1.19, respectively (Skempton 1954).

Deviatoric stresses and pore pressure at failure were used to determine the Mohr-Coulomb effective strength envelope for the control clay specimens [Fig. 10(a)]. The apparent cohesion (c') and the effective friction angle (φ') were determined to be 0 kPa and 26.3°, respectively. These effective shear strength parameters are in line with results reported by Lin and Penumadu (2005) for normally consolidated kaolin specimens.

Ottawa Sand

Ottawa sand which classifies as poorly graded sand (SP) according to the Unified Soil Classification System was used to construct the sand columns. The physical and mechanical properties for Ottawa sand are presented in Table 3. The shear strength of the sand was determined using isotropically CU triaxial tests that were conducted on specimens with a height of 14.2 cm and a diameter of 7.1 cm at effective confining pressures of 100, 150, and 200 kPa. The specimens were prepared at a dry density of 16.2 kN/m^3 corresponding to a relative density of 44%, and a void ratio of 0.60. This density corresponds to the dry density of

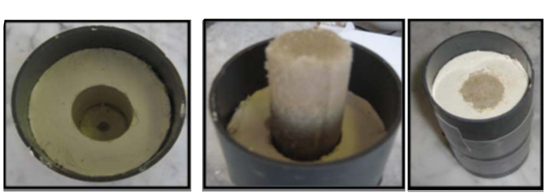

Fig. 7. Installation of nonencased sand columns

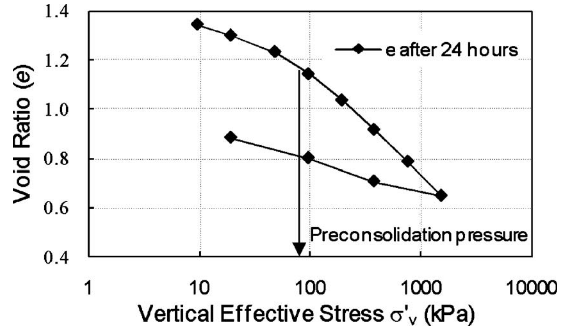

Fig. 8. e-log σ_v' for normally consolidated kaolin clay

the sand columns used in the testing program. The variations of deviatoric stresses and excess pore pressure with axial strain are shown in Fig. 9(b), while the effective shear strength envelope is shown in Fig. 10(b). Results in Fig. 9(b) indicate that the sand at this relative density has a tendency to dilate, thus resulting in the generation of negative pore pressures during undrained shear. The measured effective friction angle of the sand is equal to 33° and the apparent cohesion is zero.

Geosynthetic Fabric

The selection of the geosynthetic fabric was made based on several criteria. The fabric had to (1) ensure a moderate lateral support for the sand column during loading; (2) provide proper drainage of pore water during isotropic consolidation; and (3) prevent the mixing of the sand column material with the surrounding clay. Tensile strength tests were conducted on specimens of the selected fabric to measure the tensile strength and Young's modulus of the fabric in dry and soaked conditions and along the strong and weak fabric orientations. Young's modulus was calculated based on the stress corresponding to a tensile

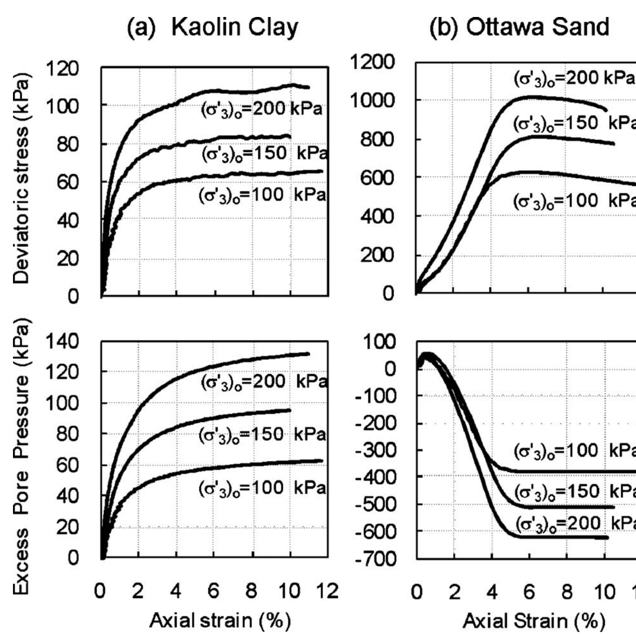

Fig. 9. Deviatoric stress and excess pore-water pressure versus axial strain for kaolin clay and Ottawa sand

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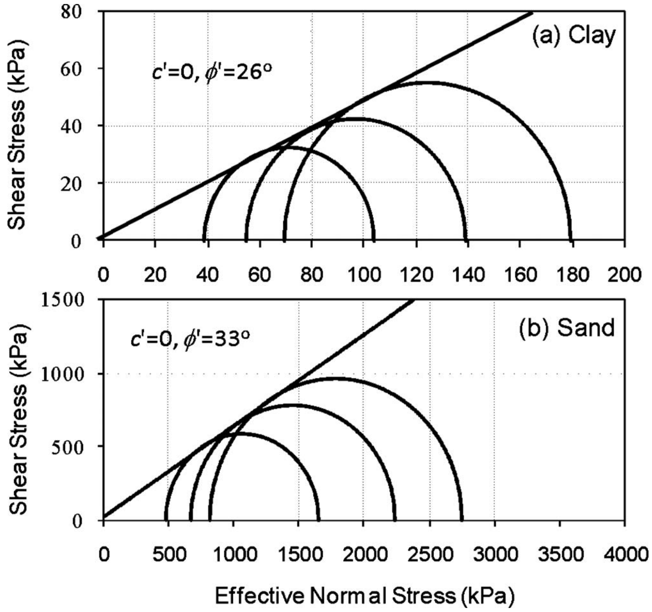

Fig. 10. Mohr-Coulomb effective stress failure envelopes for (a) kaolin clay; (b) Ottawa sand

Table 3. Physical and Mechanical Properties of Ottawa Sand

Soil property	Value
D_{10} (mm)	0.22
D_{30} (mm)	0.3
D_{60} (mm)	0.5
Coefficient of uniformity (D_{60}/D_{10})	2.3
Coefficient of curvature $(D_{30})^2/(D^*_{60}D_{10})$	0.82
Maximum and minimum void ratios (e_{max}, e_{min})	(0.75,0.49)
Specific gravity	2.65
Angle of internal friction (φ)°	33

strain of 1%. Each test was repeated twice to confirm the test results and to obtain representative average values for the tensile strength and Young's modulus (Table 4). Soaking the fabric led to a 25% reduction in the tensile strength, which was determined to be about 5.8 and 3 MPa for strong and weak fabric orientations, respectively. It should be noted that the orientation of the strong fabric in the actual triaxial tests was in the horizontal direction.

Test Results and Analysis

The automated triaxial test setup "TruePath" by Geotac was used to conduct CU tests with pore pressure measurement on control and reinforced clay specimens which were saturated under a back pressure of 310 kPa. The samples were then isotropically consolidated under effective confining pressures of 100 kPa, 150 kPa, or 200 kPa and sheared in undrained conditions at a strain rate of 1%, while measuring pore-water pressure at the bottom of the samples. Although the tests were conducted in "globally" undrained conditions (water could not move out of the specimen), local changes in water content in both the sand column and the surrounding clay are possible due to the possible flow of water between the sand and the clay during shearing. This interaction between the sand column and the clay may lead to local volume changes during the tests. Throughout the tests, the total confining pressure was kept constant as the vertical stress was increased in compression.

Mode of Failure

The mode of failure was characterized by "bulging" of the clay specimens. Clay samples that were reinforced with fully penetrating sand columns showed minimal and uniform signs of bulging along the height of the sample [Fig. 11(a)], whereas bulging for partially penetrating columns was significant and concentrated at the lower portions of the specimen [Figs. 11(b and c)]. These observations agree with findings from previous studies (Hughes and Withers 1974; Sivakumar et al. 2004) which indicate that for partially penetrating columns of short lengths, the stresses at the base of the column generally exceed the bearing capacity of the soil leading to a premature bearing capacity failure in the unreinforced lower portion of the specimen.

To investigate the mode of failure of the sand columns, specimens which were reinforced with sand columns of different penetration depths were split along their vertical axes to expose the columns and the surrounding clay (Fig. 11). The sections shown in Fig. 11 indicate that although the clay specimen as a whole bulged during undrained shear, the column itself did not exhibit any noticeable bulging along its length. This result is also in line with experimental results that were published in other studies in which the reinforced clay specimen was loaded using "area loading" rather than direct loading of the sand column (Sivakumar et al. 2004; Ambily and Gandhi 2007).

An examination of the mode of failure of kaolin specimens that were reinforced with fully and partially penetrating encased

Table 4. Results of Pullout Tests on Geotextile Fabrics

Test number	Fabric orientation	Dry fabric	Soaked fabric	Tensile force (N)	Average tensile strength (kPa)	Secant Young's modulus at 1% strain (kPa)
1	Along strong fabric orientation	X		61	5,770	35,400
2		X		66		
3			X	47	4,410	22,600
4			X	50		
5	Along weak fabric orientation	X		31	3,000	14,300
6		X		35		
7			X	22	2,300	10,000
8			X	28		

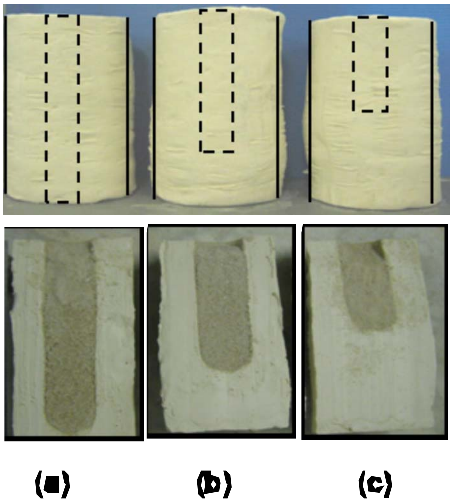

Fig. 11. Modes of failure of clay specimens (upper part) and sand columns (lower part) reinforced with (a) fully penetrating column; (b) column with penetration ratio of 0.75; and (c) column with penetration ratio of 0.5

sand columns indicated that the presence of the fabric reduced the degree of bulging significantly compared to control specimens and specimens that were reinforced with nonencased columns (Fig. 12).

Analysis of Test Results

The results presented in Table 1 were analyzed to investigate the effect of relevant parameters which include the height of the sand column, the area replacement ratio, the confining pressure, and the presence of the geosynthetic fabric on the improvement in the undrained shear strength, undrained Young's modulus, and effective strength parameters of the clay-sand column system. It should be noted that in all the discussion presented below, it was assumed that the sand column and the surrounding clay act as a single element with homogeneous distributions of stresses and strains, and no attempt was made to measure pore-water pressure in the sand column and the clay separately and at different depths in the soil specimens. Limitations in the available testing equipment and instrumentation restricted the objectives of the current study to evaluating the effect of the sand columns on the load response of the combined soil mass in lieu of attempting to isolate the proportions of the stresses that are resisted by the sand and the clay separately. It is believed that this approach could be applicable to practical field applications involving the reinforcement of soft clay layers with medium dense sand columns at relatively small area replacement ratios (less than 18%). The improved mechanical properties of the combined soil mass could be used in conventional bearing capacity models to predict the load response of foundation systems that are supported by soft clay layers that are reinforced with sand columns.

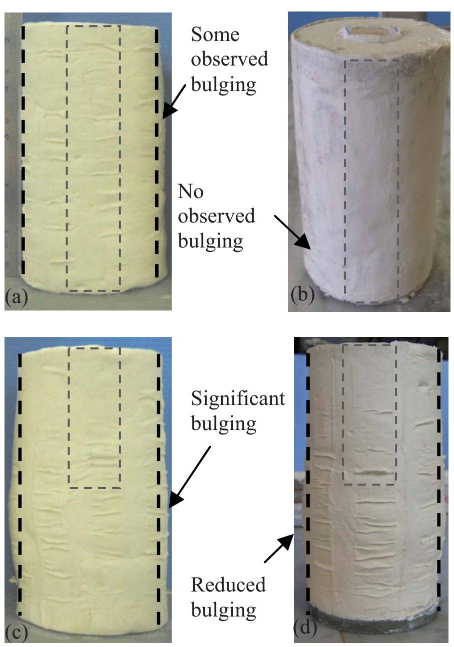

Fig. 12. Modes of failure of clay specimens with (a) fully penetrating unreinforced column; (b) fully penetrating reinforced column; (c) unreinforced column with penetration ratio of 0.5; and (d) reinforced column with penetration ratio of 0.5

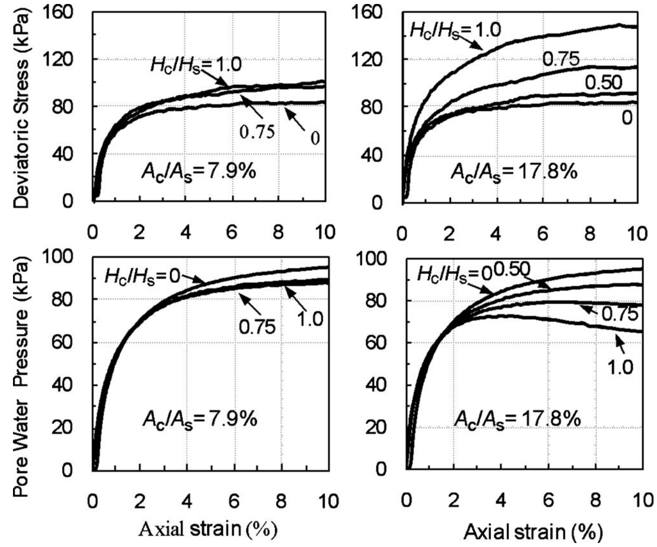

Fig. 13. Deviatoric stress and excess pore-water pressure versus axial strain for kaolin specimens reinforced with nonencased sand columns $[(\sigma_3')_o = 150\ \text{kPa}]$

Stress-Strain Behavior

Curves showing the variation in the deviatoric stress and the excess pore-water pressure with axial strain for a confining pressure of 150 kPa are shown in Figs. 13 and 14 for clay specimens that are reinforced with nonencased and encased sand columns, respectively. These results are indicative of tests conducted at other confining pressures. With the possible exception of tests with $H_c/H_s = 1.0$ and 0.75 for $A_c/A_s = 17.8\%$, in all the other cases tested, pore-water pressure increased as the deviatoric stresses reached their maximum values at axial strains of about 5–6%. For samples reinforced with nonencased sand columns, results in Fig. 13 indicate a consistent increase in the deviatoric stress and a consistent decrease in the pore-water pressure at failure, as the column penetration ratio and area replacement ratio increased. These results are in line with the experimental test results reported in Black et al. (2007), Sivakumar et al. (2004), McKelvey

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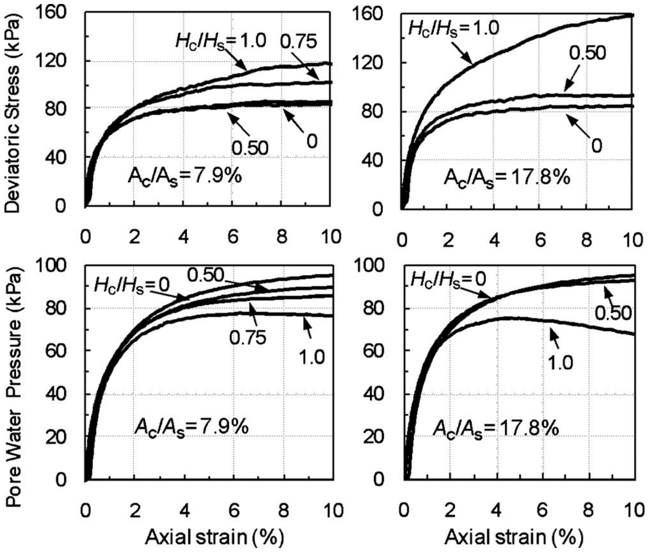

Fig. 14. Deviatoric stress and excess pore-water pressure versus axial strain for kaolin specimens reinforced with encased sand columns $[(\sigma_3')_o=150\ \text{kPa}]$

et al. (2004), and Muir Wood et al. (2000). It should be anticipated here that the decrease in excess pore-water pressure during shear in the tests conducted in this study can be attributed to the higher stiffness and to the dilatational tendency of the sand columns, whose effects are expected to become more significant as the area replacement ratios and the column penetration ratios increase.

For specimens reinforced with geotextile-encased columns, the general trends of the stress-strain and the pore pressure variation curves were similar to the trend of nonencased columns except for the fact that the deviatoric stress continued to increase even after an axial strain of 10% was achieved, indicating a strain-hardening behavior. This behavior was exhibited for fully penetrating columns only. This strain-hardening behavior was accompanied by a slight drop in the pore-water pressure (more pronounced for the high area replacement ratio). This behavior for encased specimens can be attributed to the presence of the geotextile fabric which allowed the specimens to continue to carry additional loads at high strains by providing additional lateral confinement to the sand column and possibly increasing the tendency of the sand column to dilate during shear.

Effect of Sand Columns on Undrained Shear Strength

For fully penetrating columns that were tested at effective confining pressures of 100, 150, and 200 kPa, results in Table 1 indicate that for area replacement ratios of 7.9 and 17.8%, the increase in undrained shear strength due to nonencased sand columns ranged from 13 to 19.5% and from 67.5 to 75%, respectively. For columns with a penetration ratio of 0.75, the corresponding increases in undrained strength ranged from 8.6 to 14.5% and from 20.1 to 34.9%, respectively. For the relatively small area replacement ratios used in this study, these increases in the undrained shear strength can be considered to be substantial and may be attributed to the replacement of a portion of the soft and compressible clay by a stronger and stiffer sand column. In addition, the increase in undrained strength of the composite may have been affected by a

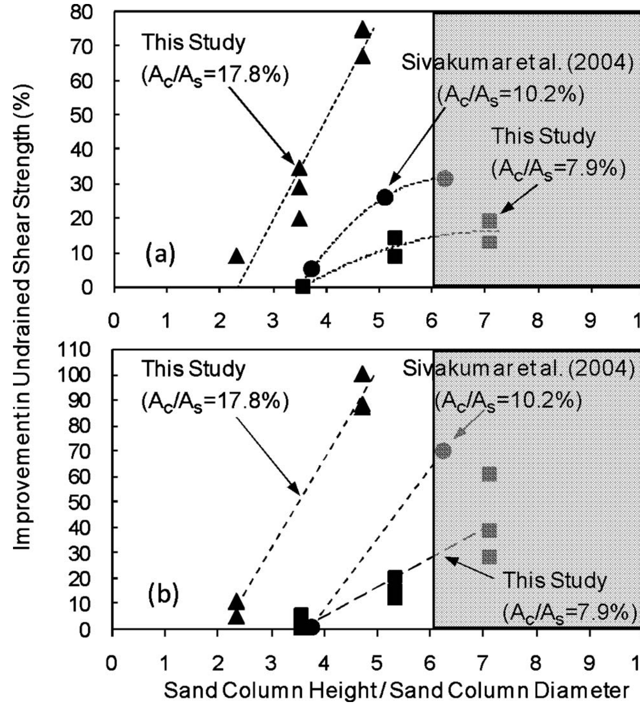

Fig. 15. Effect of ratio of column height to diameter on undrained shear strength for (a) nonencased columns; (b) encased columns

possible decrease in the water content of the clay as pore water flows out from the clay to the sand column during shear. The extent of the possible change in water content of the clay could not be quantified in the current testing program since these changes are expected to be relatively small and localized in various locations in the clay matrix.

An analysis of the results in Table 1 for test specimens that were reinforced with nonencased sand columns indicates that the relative increase in the undrained strength when comparing partially penetrating columns ($H_c/H_s=0.75$) to fully penetrating columns was much more pronounced in the case where the area replacement ratio was 17.8% (increase from 20.1 to 75% for 100 kPa confining pressure) compared to the case where the area replacement ratio was equal to 7.8% (increase from 8.6 to 13.0% for 100 kPa confining pressure). This disproportionate increase in strength indicates that the improvement in undrained shear strength may not only be a function of the column penetration ratio H_c/H_s, but also of the ratio of the column height to the column diameter. This dependence has been studied by other researchers (example Narasimha Rao et al. 1992) who proposed the idea of the critical column length beyond which the column will not have any positive effect on improvements in capacity. Narasimha Rao et al. (1992) suggested that a column length greater than five column diameters may no longer participate in increasing the load carrying capacity of soft cohesive clays.

To investigate the possible dependency of the increase in undrained strength on the ratio of the column height to column diameter, the relative increase in undrained strength was plotted in Fig. 15(a) versus the ratio of the column height to column diameter for all the tests conducted in this study on samples that were reinforced with nonencased columns and for all confining pressures. Average trend lines or curves were plotted through the data for the two area replacement ratios used. In addition, data points from tests conducted by Sivakumar et al. (2004) and which show improvements in undrained strength due to the inclusion of

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3.2-cm diameter sand columns in kaolin clay are also plotted on the same figure for comparison.

The data in Fig. 15(a) support the hypothesis of a critical column length beyond which the increase in undrained shear strength becomes relatively negligible. This critical column length can be approximated as $6D_c$ (D_c=diameter of the sand column) as indicated by the gray area in Fig. 15(a). The larger increase in strength that was observed when the height of the sand column was increased from 10.65 cm (H_c/H_s=0.75) to 14.2 cm (H_c/H_s=1) for specimens that were reinforced with an area replacement ratio of 17.8% is therefore to be ascribed to the nonlinear trend of the improvement associated with the existence of a critical column length. In these tests, the maximum ratio of the column height to column diameter was indeed smaller than 4.7 for the fully penetrating column, a ratio which is in turn smaller than the estimated critical column ratio. In comparison, the ratio of the column height to column diameter was equal to 7.1 and 5.3, respectively, for the fully penetrating and partially penetrating columns (H_c/H_s=0.75) for specimens reinforced with an area replacement ratio of 7.9%. The improvement in undrained shear strength in these specimens leveled out at a maximum improvement of about 17%. It should be noted that conducting a test with 3-cm diameter columns, in which the ratio of the column height to diameter is greater than 6 was impossible given the size limitations of the triaxial cell that was used in this study.

For encased fully penetrating columns, results in Table 1 indicate that for area replacement ratios of 7.9 and 17.8% the increase in the undrained shear strength ranged from 28.7 to 61% and from 87.5 to 100%, respectively. For an area replacement ratio of 7.9% and a small column height (H_c/D_c=3.5), the insertion of encased sand columns did not cause any noticeable increase in the undrained shear strength of the clay (only 2.7% average increase); however, increasing the column height to diameter ratio to 5.3 and 7.1 leads to an average increase in the undrained shear strength of 16.1 and 42.9%, respectively. For an area replacement ratio of 7.9%, these results indicate that the concept of the critical column length which was confirmed by the data in Fig. 15(a) for samples that were reinforced with nonencased columns becomes invalid for encased columns [Fig. 15(b)]. However, this observation could not be validated for tests with area replacement ratios of 10.2 and 17.8% since there was not enough data to determine whether a limit in the improvement of the undrained shear strength exists or not.

Effect of Confining Pressure on Degree of Improvement in Undrained Strength

The effect of confining pressure on the degree of improvement in undrained shear strength due to the presence of fully penetrating sand columns is illustrated in Fig. 16. Results in Fig. 16 indicate that for specimens that were reinforced with nonencased sand columns, the improvement in undrained shear strength was relatively independent of the effective confining pressure. This result is important because it indicates that for normally consolidated clay layers that exhibit an increase in the undrained shear strength with depth, the presence of nonencased sand columns will increase the undrained shear strength of the clay with the same percentage irrespective of the embedment depth in the clay. On the other hand, the results of encased columns shown in Fig. 16 indicate that as the effective confining pressure increased, the degree of improvement in the undrained shear strength decreased. This effect was more pronounced for samples reinforced with

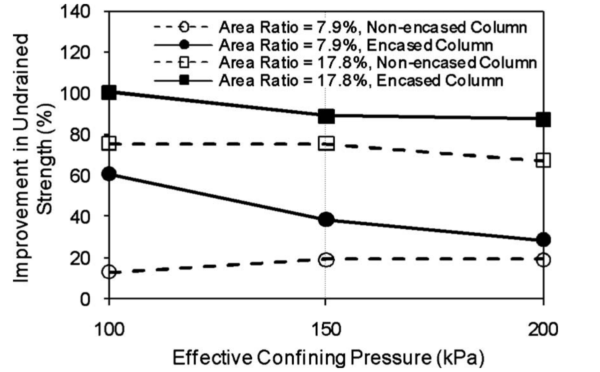

Fig. 16. Variation of improvement of undrained shear strength with confining pressure

fully penetrating 2-cm diameter encased columns where the improvement decreased from 61% at an effective confining pressure of 100 kPa, to 39% at an effective confining pressure of 150 kPa, to 28.7% at an effective confining pressure of 200 kPa.

This decrease in the degree of improvement in the undrained shear strength with effective confining pressure can be attributed to the fact that the contribution of the geotextile fabric to any strength increase is not expected to be affected significantly by confining pressure. Since the undrained strength of both the sand and the normally consolidated clay is expected to increase with effective confining pressure (Terzaghi et al. 1996), the relative contribution of the geotextile to the undrained strength of the composite is expected to become less significant as the effective confining pressure increases. The same trend is also seen for specimens that were reinforced with fully penetrating 3-cm diameter encased columns, but at a smaller scale. The percentage of improvement in the undrained shear strength decreased from 100.6 to 87.5% as the effective confining pressure increased from 100 to 200 kPa. In this case, the smaller drop is due to the fact that the presence of the 3-cm diameter sand column is the main contributor to the improvement in strength, while the geotextile has a minor contribution (the improvement was already 75% for nonencased sand columns).

Effect of Sand Columns on Pore Pressure Generation

An analysis of the results in Table 1 indicates that for samples that were reinforced with fully penetrating nonencased sand columns with area replacement ratios of 7.9 and 17.8%, the average reduction in the excess pore-water pressure at different effective confining pressures was 12.9 and 31.3%, respectively. The reduction in the generation of excess pore-water pressure during undrained loading is likely due to the dilatational tendency and the higher stiffness of the sand columns. For partially penetrating columns with H_c/H_s=0.75, the average reductions of excess pore-water pressure at different effective confining pressures were reduced to about 7 and 17% for area replacement ratios of 7.9 and 17.8%, respectively. Hence, the insertion of sand columns reduces the excess pore-water pressure generation during undrained loading, and their effectiveness in reducing the water pressure increases with increasing the column height and area replacement ratio. The insertion of fully penetrating encased sand column with area replacement ratios of 7.9 and 17.8% leads to an average reduction of 11.6 and 30.9% in the excess pore-water pressure, respectively.

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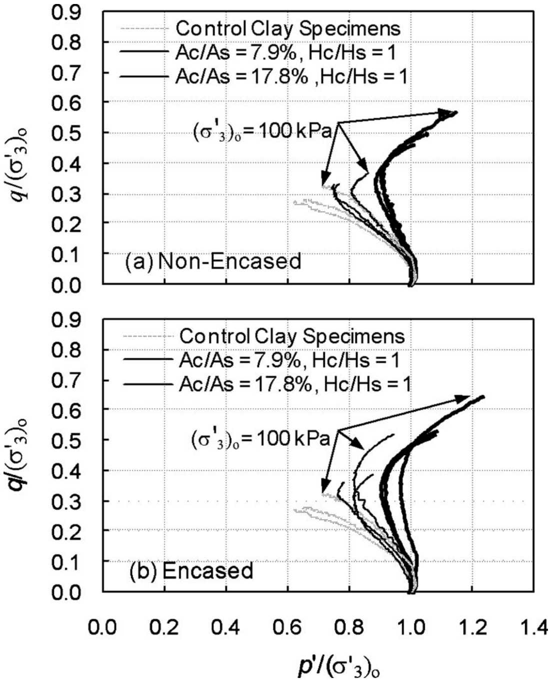

Fig. 17. Normalized stress paths for control clay specimens and specimens that were reinforced with fully penetrating columns

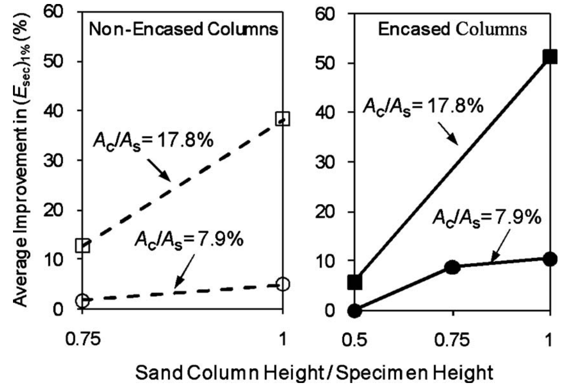

Fig. 18. Effect of encasement and column height penetration ratio on the average degree of improvement in $(E_{\mathrm{sec}})_{1\%}$

These percentages are very similar to the values reported for the nonencased columns indicating that the geosynthetic fabric did not play a role in decreasing pore pressure generation.

To better illustrate the effect of sand columns on the degree of improvement in undrained shear strength and its possible relation to the generation of pore pressures during shear normalized stress paths were analyzed for tests involving fully penetrating nonencased [Fig. 17(a)] and encased [Fig. 17(b)] sand columns for the three effective confining pressures used in this study. The normalized stress paths were plotted in Fig. 17 in terms of the invariants $q/(\sigma_3')_o$ and $p'/(\sigma_3')_o$, where q and p' are defined based on the "MIT School" (Lambe and Whitman 1969) as $q=(\sigma_1'-\sigma_3')/2$ and $p'=(\sigma_1'+\sigma_3')/2$, while $(\sigma_3')_o$=effective confining pressure used in the triaxial tests. The value of the normalized shear stress at failure $q/(\sigma_3')_o$ would correspond to the normalized undrained shear strength $s_u/(\sigma_3')_o$, while $p'/(\sigma_3')_o$ will reflect the generation of pore pressure during shear. The curves in Fig. 17 indicate that the undrained shear strength of the clay-sand column composite and the excess pore pressure at failure are closely related. For example, for a constant initial stress state, the smaller the value of the pore pressure at failure (indicated by a shift of the stress paths to the right), the larger the value of the undrained shear strength.

The normalized behavior exhibited in Fig. 17(a) reinforces the conclusion that confining pressure did not have a significant effect on the degree of improvement in undrained shear strength for specimens that were reinforced with nonencased sand columns. A slight deviation from normalized behavior was exhibited in the control specimen that was sheared at an effective confining pressure of 100 kPa. This deviation was also observed for clay specimens that were reinforced with 2- and 3-cm sand columns and sheared at the same confining pressure of 100 kPa. On the other hand, the data presented in Fig. 17(b) for encased sand columns show a markable effect of confining pressure on the undrained shear strength, which is in line with the behavior exhibited in Fig. 16.

Effect of Sand Columns on the Undrained Young's Modulus

A secant undrained Young's modulus $(E_{\mathrm{sec}})_{1\%}$ defined at an axial strain of 1% was calculated for each test by dividing the deviatoric stress measured at an axial strain of 1% by the corresponding strain. Results of the calculated values of $(E_{\mathrm{sec}})_{1\%}$ are presented in Table 1. In addition, a plot showing the average improvement in $(E_{\mathrm{sec}})_{1\%}$ for tests conducted using both encased and nonencased columns are presented in Fig. 18 for comparison. The average improvement in the secant undrained Young's modulus $(E_{\mathrm{sec}})_{1\%}$ represents the average of the relative improvements in $(E_{\mathrm{sec}})_{1\%}$ obtained for effective confining pressures of 100, 150, and 200 kPa, respectively.

Results in Table 1 and Fig. 18 indicate that the insertion of fully penetrating nonencased sand columns in the soft clay increased $(E_{\mathrm{sec}})_{1\%}$ of the unreinforced kaolin specimens by an average of 5 and 38.3% for area replacement ratios of 7.9 and 17.8%, respectively. The encasement of sand columns with a geosynthetic fabric increased the average improvement in $(E_{\mathrm{sec}})_{1\%}$ from 5 to 10.5%, and from 38.3 to 51.2%, for area replacement ratios of 7.9 and 17.8%, respectively. On the other hand, the average improvement in $(E_{\mathrm{sec}})_{1\%}$ of the reinforced clay was raised from 1.8 to 8.8% due to encasing the sand columns with geotextile fabric for an area replacement ratio of 7.9% and a column penetration ratio of 0.75.

The behavior of soils in general and of normally consolidated kaolin in particular, is likely to be highly nonlinear at all levels of strain and load directions (Al-Tabbaa 1987). To investigate the dependency of the undrained secant Young's modulus on strain, the variation of E_{sec} with the strain level at an effective confining pressure of 200 kPa is plotted in Fig. 19 for the control clay specimen, the Ottawa sand specimen, and the clay specimens that were reinforced with encased and nonencased sand columns. Results of reinforced clay specimens correspond to the case of fully penetrating sand columns with an area replacement ratio of 17.8%. The behavior observed in Fig. 19 is indicative of the behavior of specimens tested at different effective confining pressures.

The curves in Fig. 19 provide an indicative measure of the relative undrained secant Young's moduli in the reinforced specimen. An attempt was made to predict the variation of E_{sec} of the reinforced clay specimen with strain using the curves representing E_{sec} of the control specimens and the sand specimens, and taking into consideration the area replacement ratio of 17.8%. This could

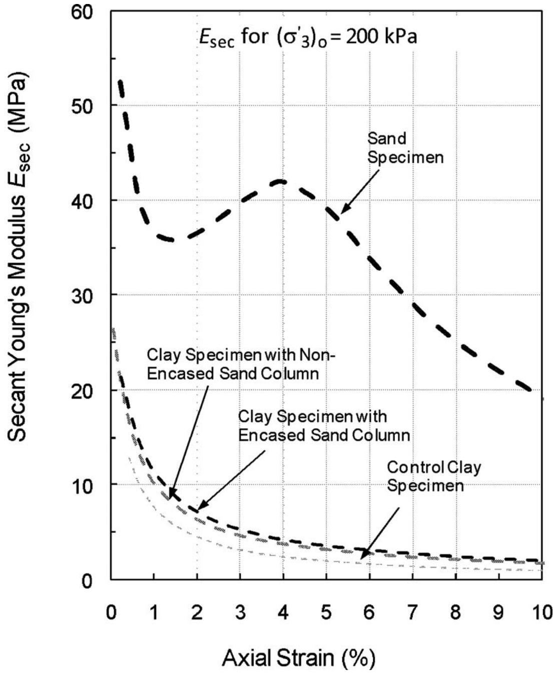

Fig. 19. Variation of E_{sec} with strain for control clay specimens, sand specimens, and reinforced clay specimens [A_c/A_s=17.8%, H_c/H_s=1.0, and $(\sigma'_3)_o$=200 kPa]

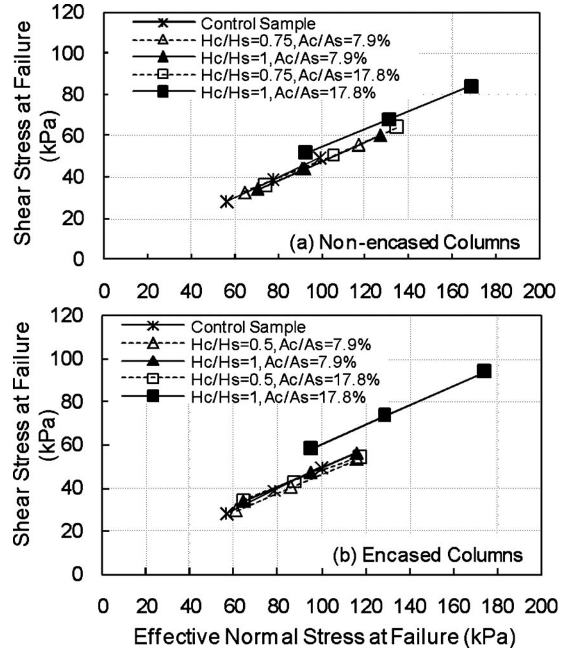

Fig. 20. Effective failure envelopes for unreinforced and reinforced kaolin specimens

be achieved using the equilibrium equation $\sigma A=\sigma_c A_c+\sigma_s A_s$ presented by Baumann and Bauer (1974), where σ, σ_c, and σ_s =total average stress acting on the soil specimen, the total stress acting on the sand column, and the total stress acting on the surrounding clay, respectively, and A=cross-sectional area of the specimen. For example, consider the stress state in the reinforced clay specimen at a strain of 1% for specimens tested at an effective confining pressure of 200 kPa. From Fig. 19, the stress in the sand column σ_c, and the stress in the surrounding clay σ_s, can be predicted at a strain of 1% using $(E_{sec})_{1\%}$ of the sand specimen (36.4 MPa) and the control clay specimen (7.63 MPa), respectively as σ_c=364 kPa and σ_s=76.3 kPa. For an area replacement ratio of 17.9%, the stress acting on the composite specimen can be calculated as σ=127 kPa based on the equilibrium equation presented above (Baumann and Bauer 1974). The calculated stress on the composite specimen corresponds to an $(E_{sec})_{1\%}$ of 12.7 MPa. The corresponding measured $(E_{sec})_{1\%}$ for the reinforced clay specimen can be obtained from Fig. 19 as 10 MPa. The observed discrepancy between the measured and predicted $(E_{sec})_{1\%}$ is expected given the interaction that may occur in the composite specimen between the sand column and the surrounding clay, and which cannot be reflected in the behavior observed for the control clay specimen and the sand specimen, separately.

Effect of Sand Columns on the Effective Shear Strength Parameters

Figs. 20(a and b) show the effective Mohr-Coulomb envelopes corresponding to each combination of area replacement ratio and column penetration ratio analyzed in the present study, for non-encased and encased columns, respectively. The resulting shear strength parameters c' and φ' are summarized in Table 5.

As indicated by the data shown in Table 5, the insertion of sand columns with different heights and diameters did not lead to a noticeable change/increase in the effective friction angle (φ') of the composite soil mass. In fact, the friction angle decreased slightly (by about 2.7°) in some cases, but was generally accompanied by an increase in the apparent cohesion (c'). The increase in the apparent cohesion (c') was significant for specimens reinforced with fully penetrating columns at an area replacement ratio of 17.8%, where c' increased from 0 to 11.9 kPa. As can be seen from Fig. 20(a), only the effective failure envelope for specimens with fully penetrating columns with a diameter of 3 cm (H_c/H_s =1) extended above the control effective failure envelope, while the other effective failure envelopes (H_c/H_s=0.75 and 1 for A_c/A_s=7.9%, and H_c/H_s=0.75 for A_c/A_s=17.8%) almost coincided with the control effective failure envelope. Hence, for bearing capacity problems involving long-term conditions, the insertion of sand columns with a friction angle of about 33° in clay layers having similar properties to the clay tested in this study may have a positive effect on the effective shear strength parameters of the reinforced clay, but this positive effect is only recognizable for relatively high area replacement ratios.

Table 5. Effective Shear Stress Failure Parameters

Column diameter (cm)	Column penetration ratio	c' (kPa)	φ' degrees)
0	0	0.0	26.3
2	0.75	4.4	23.7
2	1	1.0	25.3
2 (ESC)	0.5	3.3	23.2
2 (ESC)	0.75	5.8	23.6
2 (ESC)	1	22.3	18.4
3	0.75	0.0	25.9
3	1	11.9	23.6
3 (ESC)	0.5	9.5	21.0
3 (ESC)	1	15.1	24.3

For samples that were reinforced with the small area replacement ratio (7.9%), the presence of the geosynthetic fabric increased the apparent cohesion significantly compared to samples with non encased columns [Fig. 20(b)]. As a result, the Mohr-Coulomb effective stress failure envelope was significantly higher compared to the nonencased case, even though the effective friction angle slightly decreased. For samples reinforced with the larger area replacement ratio (17.8%), the presence of geotextile fabric slightly increased the apparent cohesion (c') compared to samples that were reinforced with nonencased sand columns. Since the effective friction angle was relatively unchanged by the geotextile encasement, the Mohr-Coulomb effective stress failure envelope for samples that were reinforced with encased columns was parallel to, but slightly higher, than the envelope of samples with nonencased columns.

Despite the major simplifying assumption which stipulates that the sand column and the surrounding clay are assumed to act as a single element with homogeneous distributions of stresses and strains, the results presented in Table 5 and Fig. 20 are significant in that they represent the first attempt to measure the effect of sand columns on the effective shear strength parameters of soft clays. The data collected in this paper show that although sand columns consistently increased the undrained shear strength for almost all the tests conducted in this study, the same conclusion could not be made with regards to the effective shear strength parameters which remained approximately constant except for cases with $A_c/A_s=17.9\%$ and $H_c/H_s=1$. This behavior can be partially explained by the fact that the reduction in pore-water pressure due to the insertion of sand columns increased the effective stresses in the reinforced clay compared to the control samples. Although this increase in effective stress was accompanied with an increase in the deviatoric stress at failure (shear stress), the increase in shear stress was not large enough to cause any significant increase in the effective shear strength parameters for the reinforced specimens. The only exception was the case with the largest area replacement ratio and fully penetrating sand columns.

Application of the Laboratory Test Data to Field Behavior

The main objective of this paper was to investigate the role that sand columns play in improving the bearing capacity and deformation response of soft normally consolidated clays. Since short-term stability conditions generally govern the design of foundation systems that are supported on soft normally consolidated clay, a laboratory testing program that is based on "undrained" triaxial tests was designed and implemented to achieve the desired objective. However, several possible implications and limitations have to be addressed before the results obtained in the current study could be extended to field applications.

Sand columns in the field are expected to act as drains that will exhibit drained behavior when sheared. In the triaxial tests conducted in this study, no global drainage was allowed through the sand columns (only localized drainage could occur between the sand columns and surrounding clay), which represents an extreme condition in the field. This discrepancy in the drainage conditions has mainly two conflicting implications. The first implication is that the sand columns in the laboratory testing program could exhibit generation of negative pore-water pressure, a behavior which may not be applicable to field conditions, except in extremely fast loading conditions that are not likely to occur in typical applications involving sand columns. The second implication is that the clay in the laboratory testing program will not exhibit unrestricted drainage through the sand columns, thus eliminating the possibility for pore pressure to dissipate freely through the sand columns. This behavior is also not applicable to practical field conditions in which the clay is expected to exhibit partial dissipation of pore pressure under typical construction loads. The two implications mentioned above are expected to have opposite effects with regards to improvements in the undrained shear strength of the clay-sand column composite.

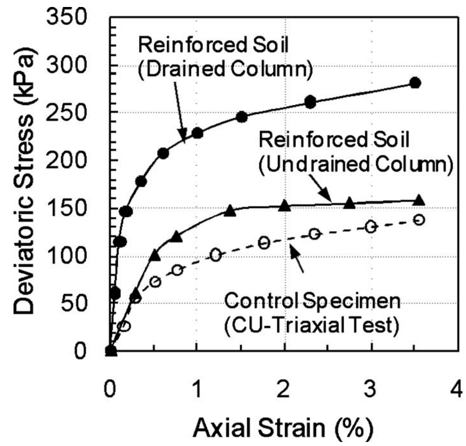

Fig. 21. Stress-strain curves from tests conducted at an area replacement ratio of 4% (data from Juran and Guermazi 1988)

In addition to the above two implications, the shear strength of a reinforced clay specimen in drained or undrained conditions can vary on the basis of the relative loads carried by the sand and clay, which in turn depend on the sand/clay relative stiffness at various levels of strains. The sand/clay relative stiffness can be different in the field depending on partially or fully penetrating columns and drained/undrained/partially drained conditions. Consequently, the extension of the conclusions of the current study to the field conditions could be conservative or not.

Juran and Guermazi (1988) implemented an experimental research program using a modified triaxial cell to investigate the effect of partial drainage of a soft silty soil that was reinforced by compacted river-sand columns. The river sand was prepared at a relative density of 80% (compared to 44% for the Ottawa sand used in the current study) and had a friction angle of 38° (compared to 33° for Ottawa sand). Relatively small area replacement ratios (about 4 and 16%) were used in the test program. To investigate partial drainage, Juran and Guermazi (1988) conducted two series of tests. In the first series, the reinforced soil was sheared while allowing drainage of the sand columns (partially drained tests), while in the second series both the sand columns and the surrounding soil were not allowed to drain (undrained test). It should be noted that the samples were instrumented with four pore pressure cells that measured the pore pressure in the sand column and the surrounding soil, and two load cells to measure the total applied load and the load supported by the soft soil.

Results reported by Juran and Guermazi (1988) indicated that the drainage of the column had a significant effect on the stress-strain response of the reinforced soil. Data collected from tests conducted on specimens with an area replacement ratio of 4% for both drained and undrained column cases were digitized from Juran and Guermazi (1988) and replotted in Fig. 21. In spite of the small replacement ratio used in the tests (area ratio=4%), the drained column significantly improved the resistance of the rein-

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forced soil to the applied strain. The observed improvement was attributed by the writers to the coupled effect of reinforcement and drainage. Moreover, results pertaining to the load carried by the sand columns indicated that the maximum load carried by the "drained" column was about twice that carried by the undrained column. The stress concentration ratio, defined as the ratio of the stress in the sand column to the stress in the surrounding soil, was equal to about 6 for samples that were reinforced with the drained column compared to 3 for samples that were reinforced with undrained columns (Juran and Guermazi 1988).

The results reported by Juran and Guermazi (1988) seem to be the only results available in the literature whereby good quality instrumented triaxial tests were performed on identical soil specimens that were reinforced with identical sand columns while allowing drainage of the sand column in one specimen and prohibiting drainage of the sand column in the other. The results obtained indicate that although dense sand columns may increase the undrained shear strength of reinforced soil due to a tendency to generate negative pore-water pressure during undrained shear, drainage of the sand columns during shear may produce a larger increase in the undrained shear strength of the reinforced soil due to inevitable pore pressure dissipation in the surrounding soil. More importantly, Juran and Guermazi (1988) stated that their results indicated that the tendency of the dense sand specimen to dilate and produce negative pore-water pressure during a typical CU Triaxial test was not observed at the same magnitude in the sand columns of the reinforced soil. They attribute this observation to the fact that in the loading test on the reinforced soil specimen, the excess pore-water pressure generated in the sand column was controlled by water flow from the surrounding contracting soft soil. This water flow from the surrounding soil to the sand column inhibits the generation of negative pore pressures during shear.

Although further studies may be needed to confirm the results obtained by Juran and Guermazi (1988), these results indicate that allowing drainage in sand columns (which is the practical case in the field) is expected to improve the undrained shear performance of the composite soil system compared to the case where the columns are undrained. If this is true, it may indicate that the results presented in this paper and which correspond to an extreme undrained condition could be conservative when extended to the field behavior in which the columns are expected to act as drains. However, additional experimental studies will have to be conducted in the future to confirm these conclusions.

It should be noted that the tests presented in this paper were performed on isotropically consolidated clay specimens. In the field, normally consolidated clays are associated with anisotropic conditions of loading. Recently, an effort has been made by several researchers (e.g., Black et al. 2006) to design and construct sophisticated equipment that would allow for maintaining Ko conditions during consolidation and applying an independent foundation load to the reinforced clay specimen within a triaxial framework. The results of such studies could provide information on the effect of anisotropy on the load response of clays that are reinforced with sand columns.

Finally, the installation of sand columns in the field could cause smearing between the sand column and the surrounding clay. Smearing could affect the process of consolidation and pore pressure dissipation for the reinforced clay. Since all the tests conducted in this paper are undrained tests, any smearing that may have occurred due to the installation of sand columns is not expected to have any effect on the test results and analysis. In addition, the installation methods used to construct sand columns in the field may result in variations in the relative density of sand columns. Such variations in density did not exist in the laboratory specimens that were constructed and tested in this study. Variability in density of sand columns is a source of uncertainty that has to be incorporated in prediction methods used to design clay systems that are reinforced with sand columns in the field.

Conclusions

Based on the results of isotropically consolidated triaxial tests in compression, the following conclusions can be drawn with regards to the effect of sand columns on the load response of soft clay:

1. Reinforcing normally consolidated soft kaolin specimens with sand columns increased the undrained Young's modulus of the reinforced clay. For fully penetrating nonencased sand columns, the improvement in the secant undrained Young's modulus of the reinforced clay increased by about sixfolds when increasing the area replacement ratio from 7.9% (5% improvement) to 17.8% (38.3% improvement). Furthermore, the presence of a geosynthetic fabric resulted in an additional increase in the secant Young's modulus.
2. The presence of fully penetrating encased and nonencased sand columns reduced appreciably the generation of excess pore-water pressure during undrained shearing. For area replacement ratios of 7.9 and 17.8%, the average reduction in the excess pore-water pressure at different confining pressures was 12.9 and 31.3%, respectively.
3. For fully penetrating nonencased sand columns and for area replacement ratios of 7.9 and 17.8%, the increase in undrained shear strength ranged from 13 to 19.5% and from 67.5 to 75%, respectively. These increases are substantial given the relatively small area ratios used. For the smaller area ratio used (7.9%), the average increase in the undrained shear strength due to encasing the sand column with geotextile fabric was raised by a factor of 2.5. However, for columns with the higher area replacement ratio of 17.8%, the addition of lateral confinement to the fully penetrating sand columns didn't affect significantly the average increase in the undrained shear strength of the reinforced clay (increase from 72.8 to 92.2%).
4. For kaolin specimens that were reinforced with non encased sand columns, the test results did not show any clear indication of the effect of confining pressure on the improvement in the undrained strength. However, specimens that were reinforced with encased columns exhibited a clear tendency for the improvement in the undrained shear strength to decrease as the confining pressure increased.
5. The data collected for samples that were reinforced with nonencased sand columns support the hypothesis of a critical column length corresponding to about six column diameters, beyond which the increase in undrained shear strength due to the presence of the sand columns becomes negligible. However, for samples with encased columns, this observation could not be validated for tests with area replacement ratios of 10.2 and 17.8% since there was not enough data to determine whether a limit in the improvement of the undrained shear strength exists or not.
6. Generally, the effective friction angle (φ') and the apparent cohesion (c') of clay specimens that were reinforced with nonencased sand columns were not significantly affected by the presence of the sand column. However, for samples that

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were reinforced with fully penetrating sand columns with an area ratio of 17.8%, c' increased from 0 kPa (for unreinforced specimen) to 12 kPa. As a result, it can be concluded based on the data that were collected in this study that reinforcing soft normally consolidated clays with sand columns with a friction angle of about 33°, will not have a significant impact on the effective shear strength parameters of the reinforced clay, except if fully penetrating columns with relatively high area ratios (greater than 17%) were used to reinforce the clay.

7. The encasement of sand columns with a geotextile fabric improved the apparent cohesion of the composite, particularly for small area replacement ratios (A_c/A_s=7.9%) and fully penetrating columns. However, the increase in c' was accompanied by a reduction in the effective angle of friction. For an area replacement ratio of 17.8%, the increase in c' was not as significant.

It should be noted that all the tests conducted in this study involved the use of single sand columns with a relative density of about 44% at area replacement ratios that are less than 18%. Since the improvement in the mechanical properties of soft clay-sand column systems in comparison to the mechanical properties of the unreinforced clay is expected to be dependent on the density (shear strength of the sand) and the area replacement ratio, future research studies should investigate the use of sand columns with higher densities and larger diameters. In addition, future studies should investigate the behavior of clay specimens that are reinforced with groups of sand columns under triaxial conditions. The objective of such a testing program is to validate the limited test results that are available in the literature and which indicate that for undrained loading, the relative increase in the undrained strength of the clay due to the presence of sand columns is independent of the column configuration (no column group effect) and is only dependent on the area replacement ratio of the reinforcement (Black et al. 2007). This would require the use of large-scale triaxial cells that would allow for the testing of clay samples with relatively large diameters. Finally, the effect of confining sand columns with geosynthetic material on the mechanical properties of the clay can be further studied by encasing the sand columns with material having different stiffnesses and tensile strengths.

Acknowledgments

The writers would like to acknowledge the support of the University Research Board (URB) at the American University of Beirut (AUB) for funding this research program.

Notation

The following symbols are used in this paper:

A = cross-sectional area of test specimen;
A_c = cross-sectional area of sand column;
A_f = Skempton's pore pressure parameter;
A_s = cross-sectional area of test specimen;
B = Skempton's pore pressure parameter;
c' = apparent cohesion;
c_c = slope of the virgin compression segment of the consolidation curve;
c_r = slope of the reloading segment of the consolidation curve;
c_s = slope of the swelling segment of the consolidation curve;
D_c = diameter of sand column;
E_{sec} = secant Young's modulus;
$(E_{\text{sec}})_{1\%}$ = secant Young's modulus at 1% axial strain;
e = void ratio of clay;
H_c = height of sand column;
H_s = height of test specimen;
p' = $(\sigma_1'+\sigma_3')/2$;
q = $(\sigma_1'-\sigma_3')/2$;
s_u = undrained shear strength of clay;
σ = total axial stress acting on soil specimen;
σ_c = total axial stress acting on sand column;
σ_s = total axial stress acting on clay surrounding sand column;
σ_v' = vertical effective stress;
$(\sigma_3')_o$ = effective confining pressure prior to shearing; and
φ' = effective friction angle.

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