Ground Improvement
Volume 168 Issue GI4

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ice | proceedings

Proceedings of the Institution of Civil Engineers
Ground Improvement 168 November 2015 Issue GI4
Pages 265–281 http://dx.doi.org/10.1680/grim.13.00049
Paper 1300049
Received 15/08/2013 Accepted 2/01/2014
Published online 07/03/2014
Keywords: columns/foundations/geotechnical engineering

Triaxial response of clays reinforced with granular columns

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The response of soft clays that are reinforced with granular columns is expected to be affected by the rate of loading and drainage conditions. The objective of this paper is to bracket the range of loading conditions in the field by analysing the results from 114 drained and undrained triaxial tests that were compiled in a state-of-the-knowledge database. For a given reinforcement condition, the percentage improvement in load-carrying capacity was found to be higher for undrained conditions than for drained conditions. The variation in the magnitude of the percentage improvement in strength with area replacement ratio indicated that, for drained conditions, there could be an upper bound area replacement ratio beyond which the benefits of increasing the area ratio become economically unjustifiable. The database was also utilised to test the validity of the concept of the critical column length. For both drained and undrained tests, results showed clear evidence that the rate of the percentage improvement in strength decreases significantly as the ratio of the column height to diameter increases beyond values of 5·0–6·0 for undrained tests and 7·0–10·0 for drained tests.

Notation

A_s	area replacement ratio
C_u	undrained shear strength
D_c	diameter of the column
D_R	relative density
D_s	diameter of the sample
H_c	height of the column
H_s	height of the sample
σ'_3	effective confining pressure
ϕ'	effective friction angle

1. Introduction and background

The increasing use of granular columns in reinforcing soft clay deposits has led to a pressing need for understanding and modelling the response of reinforced clay systems under various loading and drainage conditions. A sound understanding of the response is critical for the proper development of practical and reliable design procedures for these systems.

Despite the large number of published experimental, analytical and numerical research studies (e.g. Najjar, 2013) on the behaviour of clays that are reinforced with granular inclusions, there is still a gap in the literature with regard to the effects of rate of loading and drainage conditions on the response of these systems. For example, the majority of published experimental studies that were used as a basis for understanding the behaviour of reinforced clay systems are generally based on 1***g*** tests that are conducted in one-dimensional loading chambers (Ambily and Gandhi, 2007; Ayadat and Hanna, 2005; Cimentada *et al.*, 2011; Fattah *et al.*, 2011; Gniel and Bouazza, 2009; Hughes and Withers, 1974; Malarvizhi and Ilamparuthi, 2004; McKelvey *et al.*, 2004; Muir Wood *et al.*, 2000; Murugesan and Rajagopal, 2008; Murugesan and Rajagopal, 2010; Narasimha *et al.*, 1992; Shahu and Reddy, 2011). A thorough examination of these studies indicates that the rates at which loads were applied to the tested samples differed significantly and ranged from very quick to very slow. Since 1***g*** tests in one-dimensional loading chambers do not allow for the control of drainage and the generation of pore-water pressure, the actual drainage conditions during these tests cannot be known, and the findings from such experimental programmes may not be easily generalised or linked to a given loading and drainage condition in the field.

The limitations of 1***g*** model tests were recognised by many researchers who resorted to testing soft clay specimens that were reinforced with sand/gravel columns under triaxial conditions where the stress state, the drainage conditions and the loading rate could be controlled. Examples of studies where reinforced clays were tested under undrained conditions include Sivakumar *et al.* (2004), Black *et al.* (2007), Najjar *et al.* (2010) and Bou Lattouf (2013), while examples of studies where reinforced clays were tested under fully drained conditions include the work by Black *et al.* (2006, 2011), Sivakumar *et al.* (2011), Maalouf (2012) and Bou Lattouf (2013).

Current design procedures for bearing capacity problems involving foundations on soft clay deposits that are reinforced with sand/gravel columns lack a systematic approach for quantifying the effect of drainage and accounting for it in design. For example, the method of Hughes and Withers (1974), which is commonly used to predict the ultimate bearing capacity of a single stone column, assumes that the column will act as fully drained during loading and that the capacity of the column is governed by the limiting confining radial pressure on the column, which is evaluated as a function of the undrained shear strength of the clay. However, Hughes and Withers (1974) state clearly that in practical applications, drainage and consolidation of the clayey soil surrounding the column will occur, leading to increases in the radial stiffness. This potential increase in the capacity of a column due to partial drainage of the surrounding clay could be significant and is currently not adequately quantified.

The above discussion pertains to the analysis of single columns. In practical applications involving foundations/embankments on clays that are reinforced with sand/stone columns, the clay surrounding the column will support part of the applied stress. The importance of the contribution of the surrounding clay to the bearing capacity of the clay–column system is a function of the area replacement ratio, with the contribution of the clay becoming less as the area replacement ratio increases. Results from a field study conducted by Stuedlein and Holtz (2012) on vibro-compacted piers and tamped piers in clays led the authors to conclude that the role of the matrix soil in a group loading could be more important than the effects of installation and column type and composition, even for area replacement ratios as high as 30%. These results further reinforce the importance of accurately predicting the response of the matrix clay, which is expected to be strongly affected by the drainage conditions of the clay.

The objective of the work presented in this paper is to investigate the behaviour of clay–sand column composites at the two extreme loading conditions represented by the undrained and the fully drained cases. To achieve this objective, an effort was launched to compile the largest ‘state-of-the-knowledge’ database of available high-quality triaxial tests on soft clays that were reinforced with sand/gravel columns. The database is important for quantifying the difference in the load response of the clay–column system under drained and undrained drainage conditions in an attempt to bracket the range of loading conditions that could exist in practical field applications involving the use of sand/stone columns in soft clays.

Efforts to quantify the difference in the load response between drained and undrained conditions have been limited because of the limited number of tests that could be conducted in any given study. The newly assembled database will provide a unique opportunity to study the impact of the loading rate and drainage conditions on the observed response. The study could

(*a*) provide insight regarding possible modifications that could be made to the approaches used in predicting the capacity of clays that are reinforced with granular columns
(*b*) fill a gap in the current knowledge on the load rate effect on the behaviour of reinforced soft clays
(*c*) provide a lower and upper bound for the shear strength of these composites in practical field applications.

2. Description of database of triaxial tests

A database that comprises 114 triaxial tests on control clay specimens and specimens that were reinforced with sand/gravel columns was compiled from data reported in 11 publications (Andreou *et al.*, 2008; Black *et al.*, 2006, 2007, 2011; Bou Lattouf, 2013; Juran and Guermazi, 1988; Kim *et al.*, 2007; Maalouf, 2012; Najjar *et al.*, 2010; Sivakumar *et al.*, 2004, 2011). Among the 114 tests, 35 tests are undrained, 56 are drained, three are partially drained, and 20 are control tests (composed of nine undrained and 11 drained tests having no sand/stone column).

The database is presented in Table 1 and includes specific information about the drainage conditions (drained, D; undrained, UD; or partially drained, PD), effective confining pressure (σ_3'), loading type (area, A, or footing, F) as well as the clay properties (Atterberg limits and undrained shear strength, C_u), specimen data (diameter, D_s, and height, H_s), and detailed properties of the sand/stone column(s). The table also includes a summary of the test results which include the measured deviatoric stress/bearing capacity and the excess pore-water pressure (PWP) at failure for the undrained tests. It should be noted that all of the studies (except Juran and Guermazi, 1988) that were included in the database used kaolin clay as the main soft soil. In addition, the tests that were included are restricted to tests where the sand/stone columns were installed in a non-encased state (no geosynthetic encasement was used to reinforce the sand column) and generally using a replacement method of construction. A brief description of the tests conducted in each study is provided in the following paragraphs.

Juran and Guermazi (1988) used a modified triaxial cell to investigate the effect of partial drainage of a silty soil sample ($D_s = 10$ cm) that was reinforced by compacted river-sand columns ($D_R = 80\%$, $\phi' = 38°$) at area ratios of 4 and 16% ($D_c = 20$ and 40 mm). They conducted tests at a deformation rate of 0·05 mm/min while allowing drainage of the columns (partially drained tests). They also ran tests where both the sand column and the surrounding soil were not allowed to drain (undrained tests).

Sivakumar *et al.* (2004) performed consolidated undrained triaxial tests on sand columns with a diameter of 3·2 cm and ratios of column height to height of specimen of 0·4, 0·6, 0·8 and 1 in soft

Authors Test No.	Test properties: Drainage D/UD	Test properties: σ'_3: kPa	Test properties: Loading type A or F: cm	Clay properties: LL/PI: %	Clay properties: C_u: kPa	Specimen data: D_s: cm	Specimen data: H_s: cm	Columns: Material properties: Type	Columns: Material properties: Dry density: kN/m^3	Columns: Material properties: Relative density: %	Columns: Material properties: Friction angle: degrees	Columns: Alignment: No. of columns	Columns: Alignment: Grid (triangle/square)	Columns: Dimensions: D_c: cm	Columns: Dimensions: H_c: cm	Columns: H_c/D_c	Columns: A_s: %	Deviatoric/bearing stress at failure: kPa	Excess PWP at failure: kPa
Najjar *et al.* (2010)																			
1	UD (1%/h)	100	A	56/22	32·3	7·1	14·2	–	–	–	–	0	–	0	0	0·0	0	64·6	61·3
2		100		56/22	32·3			Ottawa sand	16·2	44	33	1	–	2	7·1	3·6	7·9	65	59·9
3		100		56/22	32·3				16·2	44	33	1	–	2	10·65	5·3	7·9	70·4	57·3
4		100		56/22	32·3				16·2	44	33	1	–	2	14·2	7·1	7·9	73·2	51·2
5		100		56/22	32·3				16·2	44	33	1	–	3	10·65	3·6	17·8	77·8	48·9
6		100		56/22	32·3				16·2	44	33	1	–	3	14·2	4·7	17·8	113·4	42·7
7	UD (1%/h)	150	A	56/22	42·1			–	–	–	–	0	–	0	0	0·0	0	84·2	95·1
8		150		56/22	42·1			Ottawa sand	16·2	44	33	1	–	2	10·65	5·3	7·9	96·4	88·9
9		150		56/22	42·1				16·2	44	33	1	–	2	14·2	7·1	7·9	100·6	87·8
10		150		56/22	42·1				16·2	44	33	1	–	3	7·1	2·4	17·8	92	87·2
11		150		56/22	42·1				16·2	44	33	1	–	3	10·65	3·6	17·8	113·6	78·1
12		150		56/22	42·1				16·2	44	33	1	–	3	14·2	4·7	17·8	147·8	65·2
13	UD (1%/h)	200	A	56/22	55·1			–	–	–	–	0	–	0	0	0·0	0	110·2	130·9
14		200		56/22	55·1			Ottawa sand	16·2	44	33	1	–	2	10·65	5·3	7·9	120·2	120·3
15		200		56/22	55·1				16·2	44	33	1	–	2	14·2	7·1	7·9	131·6	112·1
16		200		56/22	55·1				16·2	44	33	1	–	3	10·65	3·6	17·8	142·4	107·8
17		200		56/22	55·1				16·2	44	33	1	–	3	14·2	4·7	17·8	184·6	89·4
Andreou *et al.* (2008)																			
18	UD (0·3 mm/min)	50	A	55/25	15	10	20	–	–	–	–	0	–	–	–	–	–	27	24
19		100		55/25	27			–	–	–	–	0	–	–	–	–	–	55	58·2
20		200		55/25	54			–	–	–	–	0	–	–	–	–	–	112	120·8
21	D (0·003 mm/min)	50	A	55/25	15			–	–	–	–	0	–	–	–	–	–	58	–
22	UD (0·3 mm/min)	50	A	55/25	15			Hostun gravel	15·6	–	–	1	–	2	20	10·0	4	40	24
23		100		55/25	27				15·6	–	–	1	–	2	20	10·0	4	72	58·2
24		200		55/25	54				15·6	–	–	1	–	2	20	10·0	4	112	116·7
25	UD (0·3 mm/min)	50	A	55/25	15			Hostun HF sand	–	–	–	1	–	2	20	10·0	4	38	20
26		100		55/25	27				–	–	–	1	–	2	20	10·0	4	68	51·4
27		200		55/25	54				–	–	–	1	–	2	20	10·0	4	130	100

Table 1. Database of triaxial tests for soft clay specimens that are reinforced with granular columns (continued on next page)

Authors Test No.	Test properties: Drainage D/UD	Test properties: σ'_3: kPa	Test properties: Loading type A or F: cm	Clay properties: LL/PI: %	Clay properties: C_u: kPa	Specimen data: D_s: cm	Specimen data: H_s: cm	Columns: Material properties: Type	Columns: Material properties: Dry density: kN/m^3	Columns: Material properties: Relative density: %	Columns: Material properties: Friction angle: degrees	Columns: Alignment: No. of columns	Columns: Alignment: Grid (triangle/square)	Columns: Dimensions: D_c: cm	Columns: Dimensions: H_c: cm	Columns: H_c/D_c	Columns: A_s: %	Deviatoric/bearing stress at failure: kPa	Excess PWP at failure: kPa
28	D (0·003 mm/min)	50	A	55/25	15			Hostun HF sand	–	–	–	1	–	2	20	10·0	4	77	–
29		100		55/25	27				–	–	–	1	–	2	20	10·0	4	155	–
30		200		55/25	54				–	–	–	1	–	2	20	10·0	4	250	–
31	D (0·3 mm/min)	50	A	55/25	15			Hostun HF sand	–	–	–	1	–	2	20	10·0	4	51	–
32		100		55/25	27				–	–	–	1	–	2	20	10·0	4	92	–
33		200		55/25	54				–	–	–	1	–	2	20	10·0	4	220	–
Black *et al.* (2007)																			
34	UD (4%/d)	100	A	70/34	28	10	20	–	–	–	–	0	–	0	0	0·0	0	57	62
35		100		70/34	28			Fine sand	16·0	–	35	1	–	3·2	12	3·8	10·24	58·4	56
36		100		70/34	28				16·0	–	35	1	–	3·2	20	6·3	10·24	75	50
37		100		70/34	28				16·0	–	35	3	Triangle	2	12	6·0	12	70	62
38		100		70/34	28				16·0	–	35	3	Triangle	2	20	10·0	12	87	40
39	D (1·25 kPa/h)	100	A	70/34	28			–	16·0	–	–	0	–	0	0	0·0	0	92	0
40		100		70/34	28			Fine sand	16·0	–	35	1	–	3·2	20	6·3	10·24	112	0
41		100		70/34	28				16·0	–	35	3	Triangle	2	20	10·0	12	104	0
Black *et al.* (2011)																			
42	D (1 kPa/h)	75	F (6 cm)	70/34	35	30	40	–	–		–	0	–	0	0	0·0	0	427·4	0
43		75		70/34	35			Crushed basalt	16·5	–	43	1	–	2·5	12·5	5·0	17	604·8	0
44		75		70/34	35				16·5	–	43	1	–	2·5	25	10·0	17	733·9	0
45		75		70/34	35				16·5	–	43	1	–	2·5	40	16·0	17	766·1	0
46		75		70/34	35				16·5	–	43	1	–	3·2	12·5	3·9	28	645·2	0
47		75		70/34	35				16·5	–	43	1	–	3·2	25	7·8	28	693·6	0
48		75		70/34	35				16·5	–	43	1	–	3·2	40	12·5	28	854·8	0
49		75		70/34	35				16·5	–	43	1	–	3·8	12·5	3·3	40	645·2	0
50		75		70/34	35				16·5	–	43	1	–	3·8	25	6·6	40	790·3	0
51		75		70/34	35				16·5	–	43	1	–	3·8	40	10·5	40	822·6	0
52	D (1 kPa/h)	75	F (6 cm)	70/34	35			Crushed basalt	16·5	–	43	3	Square	1·8	25	13·9	28	460	0
53		75		70/34	35				16·5	–	43	3	Square	1·8	40	22·2	28	792	0

Table 1. (continued)

Authors Test No.	Test properties: Drainage D/UD	Test properties: σ'_3: kPa	Test properties: Loading type A or F: cm	Clay properties: LL/PI: %	Clay properties: C_u: kPa	Specimen data: D_s: cm	Specimen data: H_s: cm	Columns: Material properties: Type	Columns: Material properties: Dry density: kN/m^3	Columns: Material properties: Relative density: %	Columns: Material properties: Friction angle: degrees	Columns: Alignment: No. of columns	Columns: Alignment: Grid (triangle/square)	Columns: Dimensions: D_c: cm	Columns: Dimensions: H_c: cm	Columns: H_c/D_c	Columns: A_s: %	Deviatoric/bearing stress at failure: kPa	Excess PWP at failure: kPa
54		75		70/34	35				16·5	–	43	3	Square	2·2	25	11·4	40	840	0
55		75		70/34	35				16·5	–	43	3	Square	2·2	40	18·2	40	880	0
Sivakumar *et al.* (2004)																			
56	UD (4%/d)	100	A	70/34	29	10	20	–	–	–	–	0	–	0	0	0·0	0	58	59·6
57		100		70/34	29			Fine sand	16·3	–	35	1	–	3·2	8	2·5	10·24	49	63·5
58		100		70/34	29				16·3	–	35	1	–	3·2	12	3·8	10·24	59	58·3
59		100		70/34	29				16·3	–	35	1	–	3·2	16	5·0	10·24	67	55·6
60		100		70/34	29				16·3	–	35	1	–	3·2	20	6·3	10·24	73	52·2
61	UD (9 mm/d for 36–48 h)	100	F (4 cm)	70/34	29			–	–	–	–	0	–	0	0	0·0	0	220·0	NA
62		100		70/34	29			Fine sand	16·3	–	35	1	–	3·2	8	2·5	64	330·2	NA
63		100		70/34	29				16·3	–	35	1	–	3·2	12	3·8	64	366·1	NA
64		100		70/34	29				16·3	–	35	1	–	3·2	16	5·0	64	393·9	NA
65		100		70/34	29				16·3	–	35	1	–	3·2	20	6·3	64	393·9	NA
Sivakumar *et al.* (2011)																			
66	D (1kPa/h)	50	F (6 cm)	70/34	28·5	30	40	Stone: crushed basalt	–	–	35	1	–	6	40	6·7	100	720·0	0
67		50		70/34	28·5				–	–	35	1	–	5	40	8·0	69·44	563·0	0
68		50		70/34	28·5				–	–	35	1	–	4	40	10·0	44·44	417·0	0
69		50		70/34	28·5				–	–	35	1	–	6	40	6·7	100	615·0	0
70	D (1 kPa/h)	50	F(6 cm)	70/34	28·5			–	–	–	–	0	–	0	0	0·0	0	333·0	0
Kim *et al.* (2007)																			
71	D (0·05 mm/min)	98	A	39/18	–	5·2	10·4	–	–	–	–	0	–	0	0	0	0	150	0
72		196		39/18	–			–	–	–	–	0	–	0	0	0	0	250	0
73		294		39/18	–			–	–	–	–	0	–	0	0	0	0	366·7	0
74	D (0·05 mm/min)	98	A	39/18	–			Joomunjin standard sand	–	70	38	1	–	1·61	30	18·6	9·6	193·5	0
75		98		39/18	–				–	70	38	1	–	2·53	30	11·9	23·6	258	0
76		98		39/18	–				–	70	38	1	–	3·24	30	9·3	38·7	281·3	0
77	D (0·2 mm/min)	196	A	39/18	–			Joomunjin standard sand	–	70	38	1	–	1·61	30	18·6	9·6	290·3	0
78		196		39/18	–				–	70	38	1	–	2·53	30	11·9	23·6	451·6	0

Table 1. (continued)

Authors Test No.	Test properties			Clay properties		Specimen data		Columns										Deviatoric/ bearing stress at failure: kPa	Excess PWP at failure: kPa
	Drainage D/UD	σ'_3: kPa	Loading type A or F: cm	LL/PI: %	C_u: kPa	D_s: cm	H_s: cm	Material properties				Alignment		Dimensions		H_c/D_c	A_s: %		
								Type	Dry density: kN/m^3	Relative density: %	Friction angle: degrees	No. of columns	Grid (triangle/ square)	D_c: cm	H_c: cm				
79		196		39/18	–				–	70	38	1	–	3·24	30	9·3	38·7	500	0
80	D (1·0 mm/min)	294	A	39/18	–			Joomunjin standard sand	–	70	38	1	–	1·61	30	18·6	9·6	405	0
81		294		39/18	–				–	70	38	1	–	2·53	30	11·9	23·6	580·6	0
82		294		39/18	–				–	70	38	1	–	3·24	30	9·3	38·7	688	0
Maalouf (2012)																			
83	D (0·25%/h)	100	A	56/22	32	7·1	14·2	–	–	–	–	0	–	0	0	0·0	0	124·8	0
84		100		56/22	32			Ottawa sand	16·2	44	35·1	1	–	2	10·65	5·3	7·9	124·8	0
85		100		56/22	32				16·2	44	35·1	1	–	2	14·2	7·1	7·9	134	0
86		100		56/22	32				16·2	44	35·1	1	–	3	10·65	3·6	17·8	138·2	0
87		100		56/22	32				16·2	44	35·1	1	–	3	14·2	4·7	17·8	169·7	0
88	D (0·25%/h)	150	A	56/22	42			–	–	–	–	0	–	0	0	0·0	0	179·4	0
89		150		56/22	42			Ottawa sand	16·2	44	35·1	1	–	2	10·65	5·3	7·9	175·1	0
90		150		56/22	42				16·2	44	35·1	1	–	2	14·2	7·1	7·9	183·5	0
91		150		56/22	42				16·2	44	35·1	1	–	3	10·65	3·6	17·8	193·1	0
92		150		56/22	42				16·2	44	35·1	1	–	3	14·2	4·7	17·8	238·5	0
93	D (0·25%/h)	200	A	56/22	55			–	–	–	–	0	–	0	0	0·0	0	232·4	0
94		200		56/22	55			Ottawa sand	16·2	44	35·1	1	–	2	10·65	5·3	7·9	217·9	0
95		200		56/22	55				16·2	44	35·1	1	–	2	14·2	7·1	7·9	233·4	0
96		200		56/22	55				16·2	44	35·1	1	–	3	10·65	3·6	17·8	260·6	0
97		200		56/22	55				16·2	44	35·1	1	–	3	14·2	4·7	17·8	311·9	0
Bou Lattouf (2013)																			
98	D (0·25%/h)	100	A	56/22	32	7·1	14·2	Ottawa sand	16·2	44	35·1	1	–	4	10·65	2·7	31·7	155·0	0
99		100		56/22	32				16·2	44	35·1	1	–	4	14·2	3·6	31·7	210·7	0
100		150		56/22	42				16·2	44	35·1	1	–	4	10·65	2·7	31·7	219·0	0
101		150		56/22	42				16·2	44	35·1	1	–	4	14·2	3·6	31·7	292·4	0
102		200		56/22	55				16·2	44	35·1	1	–	4	10·65	2·7	31·7	291·0	0
103		200		56/22	55				16·2	44	35·1	1	–	4	14·2	3·6	31·7	369·7	0
104	UD (1%/h)	100	A	56/22	32			Ottawa sand	16·2	44	33	1	–	4	10·65	2·7	31·7	83·0	42·8
105		100		56/22	32				16·2	44	33	1	–	4	14·2	3·6	31·7	173·5	2·9
106		150		56/22	42				16·2	44	33	1	–	4	10·65	2·7	31·7	136·2	64·1
107		150		56/22	42				16·2	44	33	1	–	4	14·2	3·6	31·7	256·0	4·9

Table 1. (continued)

Authors Test No.	Test properties			Clay properties		Specimen data		Columns										Deviatoric/bearing stress at failure: kPa	Excess PWP at failure: kPa
	Drainage D/UD	σ'_3: kPa	Loading type A or F: cm	LL/PI: %	C_u: kPa	D_s: cm	H_s: cm	Material properties				Alignment		Dimensions		H_c/D_c	A_s: %		
								Type	Dry density: kN/m^3	Relative density: %	Friction angle: degrees	No. of columns	Grid (triangle/square)	D_c: cm	H_c: cm				
108		200		56/22	55				16·2	44	33	1	–	4	10·65	2·7	31·7	148·2	99·9
109		200		56/22	55				16·2	44	33	1	–	4	14·2	3·6	31·7	300·3	24·1
Juran and Guermazi (1988)																			
110	PD (0·05 mm/min)	200	A/F (10 cm)	35/14	–	10·0	NA	Compacted river sand	15·5	80	38·0	1	–	NA	Full penet.	–	4·0	294·0	–
111	UD (0·05 mm/min)	200		35/14	–				15·5	80	38·0	1	–	NA	Full penet.	–	4·0	159·0	–
112		200		35/14	70			–	–	–	–	0	–	0	0	–	0·0	140·0	–
113	PD (0·05 mm/min)	K_0 test (no lateral displace-ment)		35/14	–			Compacted river sand	15·5	80	38·0	1	–	NA	Full penet.	–	4·0	746·0	–
114				35/14	–				15·5	80	38·0	1	–	NA	Full penet.	–	16·0	1404·0	–

A, F, area loading type or footing loading type; LL/PI, liquid limit/plasticity index.

Table 1. (continued)

kaolin specimens with diameter of 10 cm and length of 20 cm. The kaolin specimens were subjected to entire area loading and/or foundation loading with a footing having a diameter of 4 cm. Kaolin specimens were initially consolidated under a vertical air pressure of 200 kPa. Sand columns with lengths of 8, 12, 16 and 20 cm were then installed in the clay using wet compaction and freezing methods. Prior to shearing the specimen, the specimens were isotropically consolidated at an effective confining pressure of 100 kPa, and then a back pressure of 300 kPa was applied to guarantee complete saturation.

Black *et al.* (2006) tested K_0-consolidated kaolin clay samples with a diameter of 30 cm and a height of 40 cm. Kaolin specimens were prepared from a slurry and consolidated under a stress of 75 kPa. Crushed basalt columns with a diameter of 2·5 cm were then installed using the wet compaction method and the specimen was subjected to an isotropic effective confining pressure of 75 kPa. This was then followed by K_0 loading with a vertical stress of 125 kPa and a horizontal stress of 100 kPa. The loads were applied by way of a circular plate with a diameter of 6 cm at a rate of 0·8 kPa/h to achieve fully drained loading conditions. The test took 2–3 weeks to reach a settlement of 15–20 mm.

Black *et al.* (2007) prepared kaolin specimens with a length of 20 cm and a diameter of 10 cm from slurry that was consolidated under one-dimensional conditions. Specimens were reinforced either with a single frozen sand column with a diameter of 3·2 cm or with three frozen sand columns of diameter 2·0 cm. The lengths of the columns were varied between 12 cm and 20 cm. The kaolin specimens were consolidated under a pressure of 100 kPa prior to undrained or drained shearing.

Kim *et al.* (2007) conducted a series of consolidated drained (CD) triaxial tests on soft kaolin clay (52 mm in diameter and 104 mm high) reinforced with fully penetrating sand columns (diameters of 16·1, 25·25, 32·35 and 52 mm). Kaolin specimens were prepared from slurry and then consolidated under a preconsolidation pressure of 49 kPa. Sand columns were prepared by freezing Joomunjin standard sand in prefabricated moulds and installed using casing pipes into the kaolin samples. The system was subjected to isotropic consolidation under confining stresses of 98, 196 and 294 kPa and sheared at a strain rate of 0·05, 0·2 and 1·0 mm/min, respectively.

Andreou *et al.* (2008) conducted triaxial compression tests on kaolin clay reinforced with single columns consisting of Hostun (HF) sand and gravel. Drained, undrained and partially drained tests were conducted on 10 cm diameter clay samples reinforced with 2 cm diameter columns with a height of 20 cm. The samples were consolidated under confining pressures of 50–200 kPa.

Najjar *et al.* (2010) conducted 31 consolidated undrained triaxial tests on normally consolidated kaolin specimens (height = 14·2 cm and diameter = 7·1 cm) that were prepared from slurry and reinforced with ordinary and encased quartz sand columns, and tested at confining pressures of 100, 150 and 200 kPa. The diameter of the sand columns was varied from 2 to 3 cm, while the height of the columns was varied from 7·1 to 10·6 to 14·2 cm.

Black *et al.* (2011) tested clay samples with diameters of 30 cm and depths of 40 cm that were consolidated under K_0 conditions. Kaolin slurry was initially consolidated to a vertical pressure of 150 kPa and gravel columns with diameters of 2·5, 3·2 and 3·8 cm and lengths of 12·5, 25 and 40 cm were installed using the replacement method. For group loading, three columns of 1·8 cm and 2·2 cm diameters were adopted to produce area ratios of 28% and 40%, respectively. Following column installation, the sample was consolidated under an effective cell pressure of 75 kPa followed by K_0 consolidation with a K_0 of 0·71. The final step included applying foundation loading under drained conditions at a rate of 1 kPa/h.

Sivakumar *et al.* (2011) used a large triaxial cell to test clay samples with diameters of 30 cm and depths of 40 cm, reinforced with columns of compacted crushed basalt. Samples were consolidated under a confining pressure of 50 kPa followed by foundation loading at a rate of 1 kPa/h. The column diameters were 4, 5 and 6 cm and the loading plate had a diameter of 6 cm.

Maalouf (2012) conducted 15 consolidated drained triaxial tests on kaolin specimens (height = 14·2 cm and diameter = 7·1 cm) that were prepared from slurry and reinforced with ordinary quartz sand columns (D_R = 44%), and tested at confining pressures of 100, 150 and 200 kPa. The diameter of the sand columns was varied from 2 to 3 cm while the height of the columns varied from 10·6 to 14·2 cm. Using the same test set-up, Bou Lattouf (2013) conducted consolidated drained and undrained triaxial tests on kaolin samples that were reinforced with 4 cm sand columns.

It should be noted that the study by Sivakumar *et al.* (2011) was not considered, owing to the incorporation of pressure cells within the sand columns, which according to the authors affected the measured capacity and led to unreliable results.

The database of triaxial tests as reflected in Table 1 covers the typical ranges of soil properties and design conditions that exist in practical field applications involving the use of granular columns in clays. With regard to soil properties, the cases under consideration encompass clays with undrained shear strength values ranging from 15 to 55 kPa, with granular columns consisting of either sand or gravel with friction angles ranging from 33° to 43°. The variation in the geometries of the columns (diameter and length) relative to the geometries of the clay specimens and the size of the loading plates allowed for relatively wide ranges of area replacement ratios (between 4 and 64%), and ratios of column height to column diameter (between 2·4 and 22·2).

3. Analysis of test results

The main data that originated from the collected test results consisted of axial stress–strain curves, excess pore pressure plotted against axial strain curves, and volumetric strain plotted against axial strain curves. Since the presentation of such data for all the database is not practical, given the limitations of the length of this paper, a decision was made to limit the analysis of the test results to improvements in the deviatoric stresses at failure and their relationship to the different parameters that affect the load response of the composite specimens. These parameters include: the effective confining pressure, the area replacement ratio, the ratio of the column height to column diameter, and the type and density of the columns as indicated by the friction angle of the column material. For all the above parameters, the analysis was conducted separately for the drained tests and the undrained tests in an attempt to isolate the effect of the drainage conditions and the loading rate on the response of the composite.

3.1 Effect of confining pressure on improvement in load-carrying capacity

Unlike laboratory 1***g*** tests, which are generally conducted at very low effective confining pressures due to the relatively small scale utilised, triaxial tests allow for subjecting the composite specimen to effective confining pressures that could be comparable to the stress levels that are expected in practical field applications involving sand/stone columns in soft clays. In this section, the results from the triaxial tests are analysed to isolate any possible effect that confining pressure could have on the response of the composite specimens. To achieve this objective, a percentage improvement in strength is defined as the ratio of the difference between the deviatoric stresses of the composite specimen and the control specimen (measured at failure) to the deviatoric stress of the control specimen. This percentage improvement in strength is a normalised parameter that could be used to reflect the role of the granular column in defining the load-carrying capacity of the composite compared to an equivalent unreinforced clay specimen.

The percentage improvement in strength was calculated for all the tests in the database. For the tests presented in Juran and Guermazi (1988), the percentage improvement could only be calculated for the undrained test with an area replacement of 4%, since the other tests did not have an equivalent test done on a control specimen. To test the effect of confining pressure on the calculated percentage improvement, the variation of the percentage improvement in strength with effective confining pressure is plotted in Figures 1(a) and 1(b) for tests conducted under undrained and drained conditions, respectively. To allow for a consistent analysis, only cases with fully penetrating columns are included in the analysis.

For the undrained tests that are shown in Figure 1(a), results indicate that the percentage improvement in the deviatoric stress at failure (and thus the undrained shear strength) is generally not sensitive to the effective confining pressure. The only exceptions are the test cases presented in Andreou *et al.* (2008) which showed that the percentage improvement decreases systematically from 48·1% to 0% as the effective confining pressure increases from 50 kPa to 200 kPa, with the major drop being from an effective confining pressure of 50 kPa to a confining pressure of 100 kPa. The authors attributed this behaviour to a phenomenon of grain-size redistribution of the gravel columns causing a drop in the columns' bearing capacity at high confining pressures.

It is worth noting that in all the other studies in which no systematic effect of confining pressure was observed, the range of confining pressure was from 100 kPa to 200 kPa, indicating that the effect of confining pressure could be more evident at relatively low confining pressures (less than 100 kPa). The improved performance of the granular column at lower confining pressure could be the result of the higher secant friction angles that are expected at small confining pressures due to the non-linearity of the Mohr–Coulomb envelope in the low-pressure range. The effect of curvature in the Mohr–Coulomb failure envelope is described with respect to bearing capacity of stone columns in clays in Stuedlein and Holtz (2013).

The effect of confining pressure on the percentage improvement in deviatoric stress at failure for the drained triaxial tests is reflected in Figure 1(b). Results shown in Figure 1(b) are in line with the results observed for the undrained tests, where no significant effect of confining pressure was observed in the typical range 100–300 kPa, despite a clear general trend for the improvement to be reduced slightly as the confining pressure increased. However, an investigation of the results in Figure 1(b) in the relatively small range of confining pressures (generally less than 75 kPa) indicates that the measured percentage improvements for tests conducted in this range were generally higher than those observed at the higher confining pressures (greater than 100 kPa). For example, the test conducted by Andreou *et al.* (2008) at a confining pressure of 50 kPa using a small area replacement ratio of only 4% resulted in relatively higher percentage improvements than the tests conducted by Black *et al.* (2007), Kim *et al.* (2007) and Maalouf (2012) at larger confining pressures (between 100 kPa and 200 kPa) despite the fact that the latter tests were conducted at higher area replacement ratios (ranging from 7·9 to 17·8%). Similar observations could be made for the tests conducted by Black *et al.* (2007) at a confining pressure of 75 kPa, which showed relatively larger improvements in strength compared to tests conducted by Kim *et al.* (2007) and Bou Lattouf (2013) for relatively similar area replacement ratios but at higher confining pressures (larger than 100 kPa).

Care should be exercised in generalising the above findings given that other parameters (such as the type and density of the columns) could have also played a role in defining the percentage improvement in deviatoric stress. As with the undrained tests, it is hypothesised that the seemingly better performance of granular columns in the low-pressure range could be due to the higher

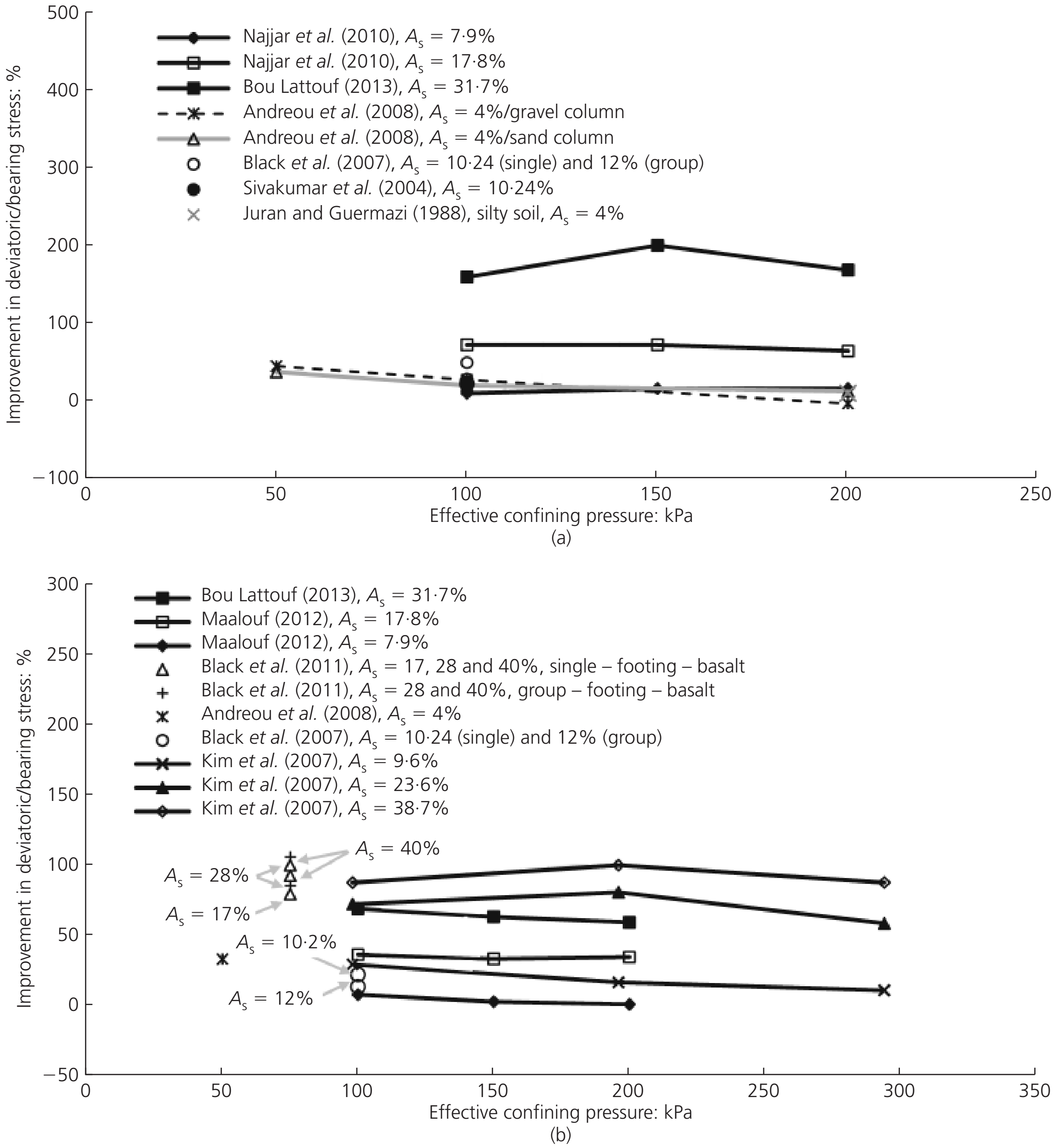


Figure 1. Variation of the improvement in deviatoric/bearing stress as function of the confining pressure in fully penetrating columns for (a) undrained and (b) drained loading conditions

secant friction angle which is expected to improve the column performance.

3.2 Effect of area replacement ratio on improvement in capacity

Results in Figure 1 illustrate clearly that the area replacement ratio plays a significant role in defining the ability of a composite specimen to carry load. To illustrate the effect of the area replacement ratio on the degree of improvement in strength, the variation of the percentage improvement in deviatoric stress at failure with area replacement ratio for samples reinforced with fully penetrating columns is plotted in Figures 2(a) and 2(b), respectively for undrained and drained conditions.

For the undrained tests, the available data points cover area replacement ratios reaching a maximum value of 31% (Figure 2(a)). In this range, results in Figure 2(a) indicate that the percentage improvement in strength increases systematically with increases in the area replacement ratio, reaching maximum values of about 200% for the maximum area replacement ratio encoun-

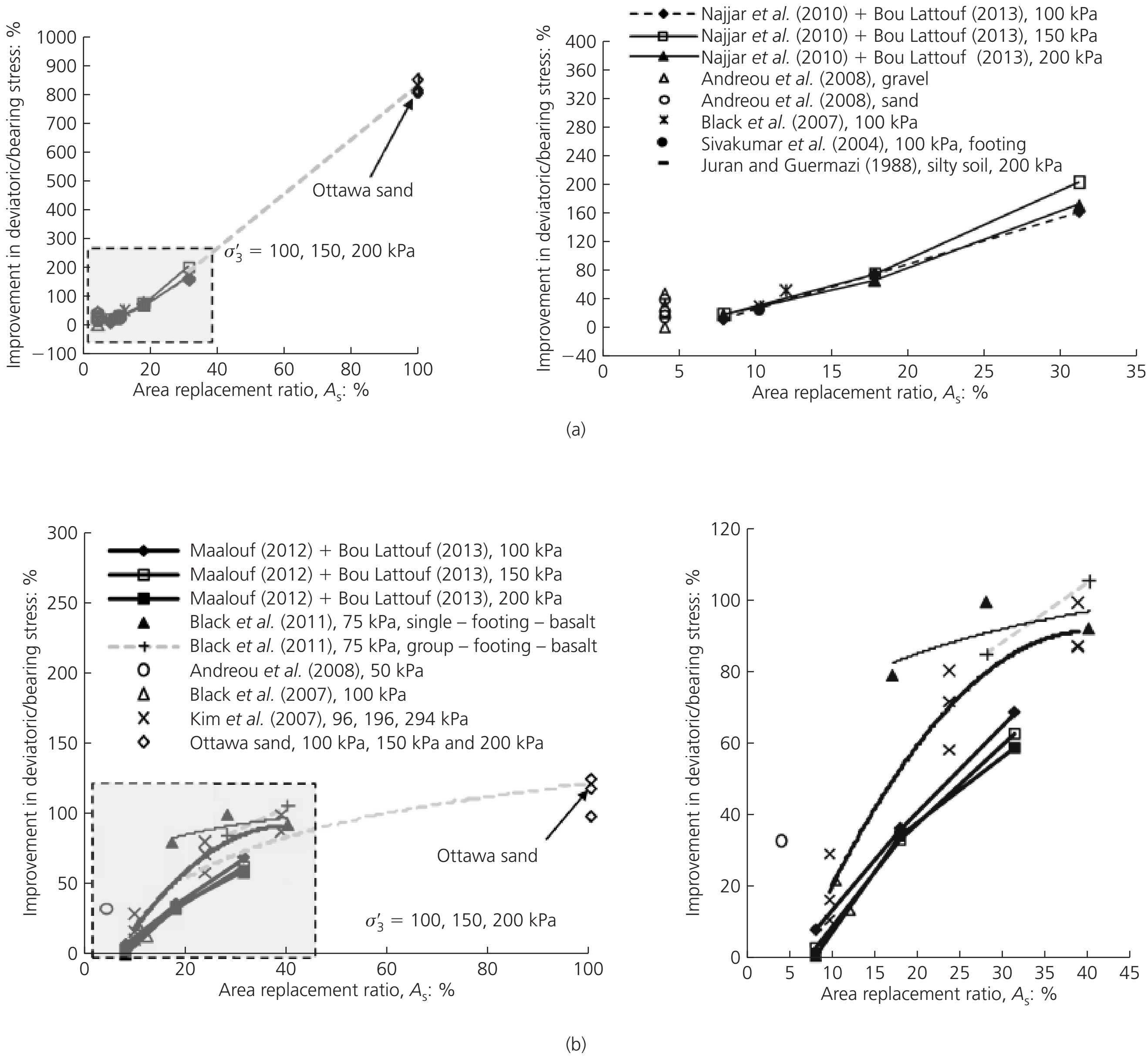


Figure 2. Effect of the area replacement ratio on the degree of improvement in deviatoric/bearing stress in fully penetrating columns for (a) undrained and (b) drained loading conditions

tered (about 31%). This relatively large increase in deviatoric stress at failure for undrained loading conditions is related to the generation of negative pore-water pressures in the granular column during shearing. The generation of negative pore pressures due to the dilative tendency of medium dense to dense granular columns will increase the effective stresses in the column, leading to a better performance.

It is worth noting that as the area replacement ratio increases, the contribution of the column to the undrained bearing capacity of the composite seems to increase in a non-linear manner. This is exhibited in Figure 2(a) by observing the seemingly increasing slope (concave upward relationship) that relates the area replacement ratio to the percentage improvement in undrained strength. To check whether the concave upward relationship holds for higher area replacement ratios, the percentage improvement in undrained shear strength was calculated for the case with an area replacement ratio of 100% (samples including only sand). Such results (undrained tests on sand specimens) were published in Najjar *et al.* (2010) for Ottawa sand specimens that were tested in a medium dense state under confining pressures of 100, 150 and 200 kPa. The 'ultimate percentage improvements' calculated from the results of these tests indicate improvements in strength reaching about 800% when compared to the control clay specimen tested in the same study. When plotted in Figure 2(a), these results further reinforce the concave upward relationship between the area replacement ratio and the percentage improvement for undrained loading conditions, indicating an increased efficiency of granular columns as the area replacement ratio increases. This observation, which is based on a relatively small number of undrained triaxial tests, should be confirmed in the future whenever additional tests become available in the published literature.

For drained loading conditions, the data available covered a wider range of area replacement ratios (up to 40%) as shown in Figure 2(b). As with the undrained tests, the percentage improvement in capacity increased as the area replacement ratio increased. However, the data reflect two major differences between the drained and undrained results. The first difference is in the magnitude of the percentage improvement in deviatoric stress at failure, which was found to be smaller (maximum value of about 100%) in drained tests compared to undrained tests (maximum value of about 200%). The second difference is related to the variation of the percentage improvement with area replacement ratio which indicates a decreasing trend of improvement (a concave downward relationship) as the area replacement ratio increases. This observation is important because it indicates that the benefit of increasing the area replacement ratio for drained conditions becomes less significant at replacement ratios of the order of 40%. For practical applications, this may indicate that 40% could be a representative upper bound for the area replacement ratio given that increasing the ratio beyond that value might not result in benefits that are economically justifiable. These outcomes are valid in the case where the columns are fully penetrating in the soft material and for long-term analysis which corresponds to fully drained conditions.

To check whether the concave downward relationship that was observed for the drained tests holds for higher area replacement ratios, the percentage improvement in drained shear strength was calculated for the case with an area replacement ratio of 100% (samples including only sand). Results of drained tests on Ottawa sand specimens that were tested in a medium dense state under confining pressures of 100, 150 and 200 kPa (Maalouf, 2012) indicate that the average ‘ultimate percentage improvement’ in strength reached only 120%, a value that is slightly higher than the percentage improvement in strength obtained for samples reinforced at much smaller area replacement ratios of 30–40% (about 100% improvement).

3.3 Comparison between triaxial test data and limited field data

Stuedlein and Holtz (2013) published a database of field tests that were conducted on clays that were reinforced with stone columns. To test the applicability of the trends observed in Figure 2 to actual field tests, the percentage improvements in capacity for field tests that included information on the bearing capacity of footings for both the reinforced and unreinforced clay (Baumann and Bauer, 1974; Greenwood, 1975; Han and Ye, 1991; Stuedlein and Holtz, 2012) were calculated and plotted in Figures 3(a) and 3(b) for undrained and drained conditions, respectively. Also plotted in Figures 3(a) and 3(b) are the results of the triaxial tests and the general trend that was observed in the triaxial tests for comparison. Detailed information on the field tests that were used is presented in Stuedlein and Holtz (2013) and in the corresponding original references.

Results in Figure 3 indicate that the magnitudes of the percentage improvement in the bearing capacity for the field tests are generally in line with those witnessed in the triaxial tests, although they exhibit scatter around the relationship observed in the triaxial tests. This result is expected given that the field tests are conducted on natural clays (and not kaolinite clay) that were reinforced with relatively dense, full-scale stone columns (and

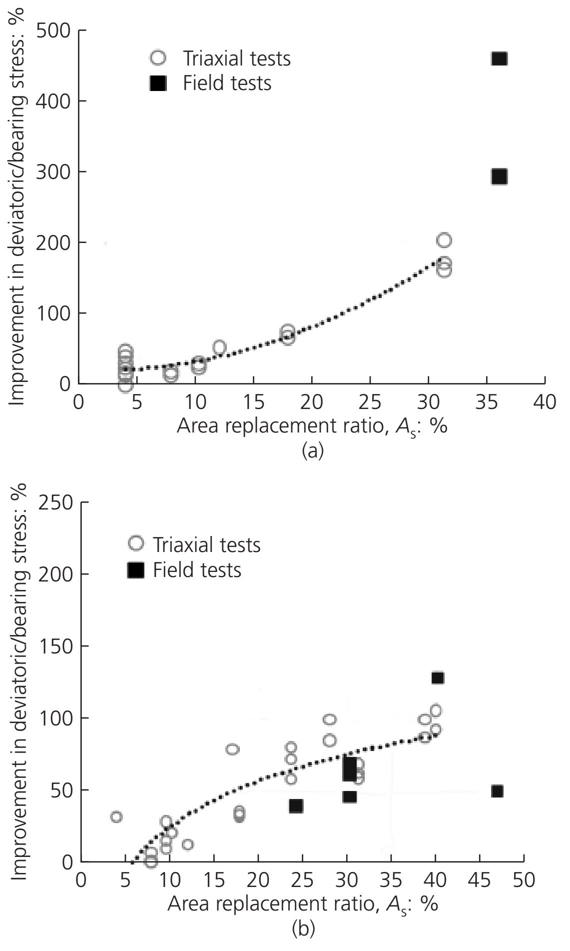


Figure 3. Comparison between percentage improvement in deviatoric/bearing capacity from triaxial and field tests: (a) undrained loading (note: field tests on improved ground are ‘partially drained’, but more likely to be closer to ‘undrained’); (b) drained loading (note: field tests on improved ground are ‘partially drained’, but more likely to be closer to ‘drained’)

not on small-size medium-to-dense sand/gravel columns). Interestingly, the results of the limited field tests provide supporting evidence to the observation of concavity in the relationship between area replacement ratio and the percentage improvement in strength/bearing capacity for both drained and undrained conditions. Additional field tests could be conducted in the future to confirm these relationships.

It should be noted that there is considerable uncertainty associated with the decision of designating a given field test in the 'undrained' or 'drained' category. The control tests in the field (the relatively quick load tests on the unimproved clay) are very likely to be associated with an 'undrained' response. On the other hand, test cases on improved ground with area replacement ratios and/or coefficients of consolidation that are relatively high are likely to be associated with a partially drained response that could be classified as 'drained'. This is because with the reduced drainage path, the clay surrounding the columns could drain radially into the granular column even if the test duration is relatively short. Accurate quantification of the degree of partial drainage in each field test is not practical given that most studies do not report accurate data about the radial coefficient of consolidation. The field test by Baumann and Bauer (1974) is an example of how a field test, which was conducted over 8 h on a clay that was reinforced with stone columns at a relatively large area replacement ratio of about 47%, exhibited more than 90% dissipation of pore pressures during each loading increment, resulting in a more-or-less drained response. The pore pressures were measured using piezometers that were placed in between the columns.

Unfortunately, readings of pore pressure were not available in the other published field tests to confirm the drainage conditions. As a result, judgement was exercised in determining whether a given field test result should be plotted with the undrained triaxial tests (Figure 3(a)) or drained triaxial tests (Figure 3(b)). Another complicating factor lies in the computation of the percentage improvement in strength for field tests that are designated as drained. In these cases, since the control test is expected to be 'undrained', the improvement in bearing capacity is calculated from a 'drained capacity' minus an 'undrained capacity', which further complicates the analysis and presentation of the field results. Based on the above limitations, it should be reiterated that the comparison presented on Figures 3(a) and 3(b) between triaxial and field results should be considered as indicative only.

3.4 Effect of friction angle of column on improvement in capacity

For a given effective confining pressure, the density and grain size of granular columns are expected to play a significant role in defining the response of the column. Larger densities and grain sizes (gravel as opposed to sand) will result in larger friction angles for a given column material. For composite samples with area replacement ratios that are relatively similar, it is expected that the behaviour of the composite will be dependent on the friction angle of the column material. Differences in friction angles could explain part of the scatter that was observed in the relationship between the area replacement ratio and the percentage improvement in strength.

To quantify the effect of the friction angle of the column material on the observed response of the composite, test cases involving fully penetrating columns tested under fully drained conditions were subdivided into categories based on the area replacement ratio utilised. The four categories that were adopted are shown in Figure 4, where the percentage improvement in the drained capacity was plotted against the friction angle of the column material. An investigation of the data in Figure 4 leads to the following observations.

(*a*) As the friction angle of the column material increases from about 35° to 43°, the percentage improvement in the drained deviatoric stress exhibits a relatively linear and consistent increase, irrespective of the category of the area replacement ratio.
(*b*) The trend lines for the variation of the percentage improvement with area replacement are more-or-less parallel to each other, irrespective of the area replacement ratio. The average slope of the trend lines indicates that the percentage improvement in the deviatoric stress at failure is expected to increase by 5% for a 1° increase in the friction angle of the column material.

3.5 Effect of column slenderness ratio on improvement in capacity

For a given area replacement ratio, increasing the height-to-diameter ratio of the column (referred to as slenderness ratio) results in an increase in the ultimate capacity of the composite. Based on a compilation of results from published field, laboratory and finite-element tests, Najjar (2013) reports that the column length is significant up to a certain point beyond which increasing the column length will not lead to an increase in strength and that the optimum length of stone columns for effective load transfer lies between five and eight times the diameter of the column. Narasimha *et al.* (1992), Hu (1995) and Sivakumar *et al.* (2004)

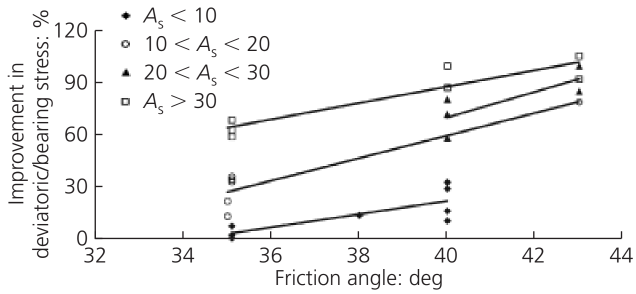
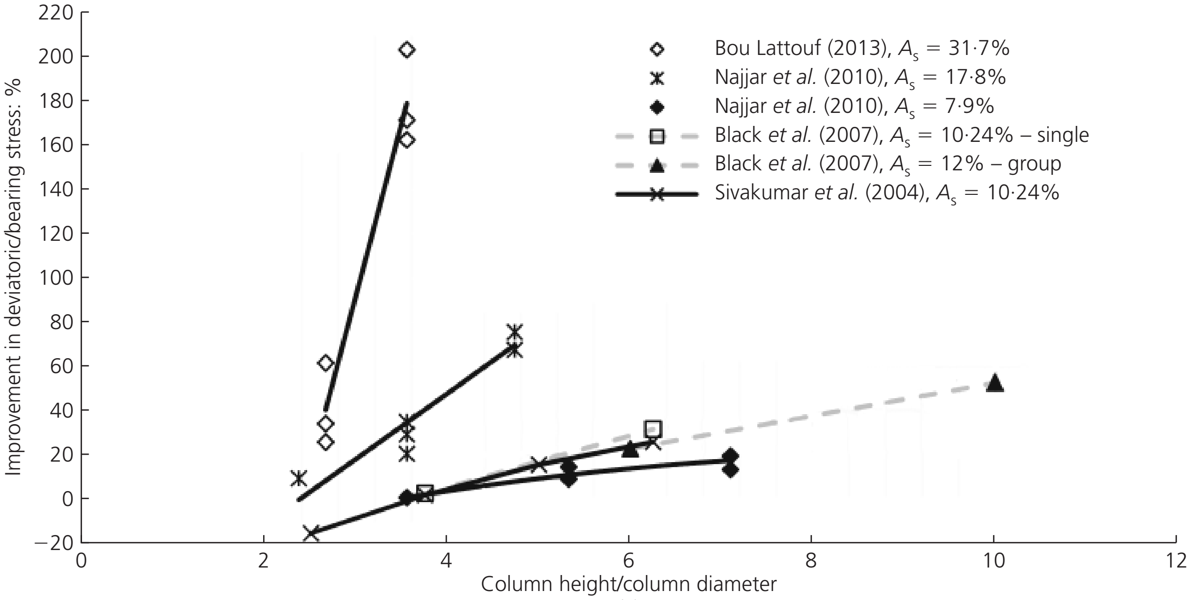


Figure 4. Effect of the friction angle of the column on the degree of improvement in deviatoric/bearing stress in fully penetrating columns under drained loading conditions

and claimed that the critical column length is five times the column diameter for undrained loading conditions, and Najjar *et al.* (2010) suggested that the critical column length is six times the column diameter. Other studies, such as Hughes and Withers (1974), Muir Wood *et al.* (2000), and McKelvey *et al.* (2004), report that a column height of four to eight times its diameter constitutes the depth beyond which the column will no longer cause major improvement to the composite soil. The critical length may increase as the area ratio increases since the failure mechanism could be pushed deeper below the footing.

The results of the triaxial tests that were assembled in this paper were used to investigate the impact of the slenderness ratio on the improvement in strength for partially penetrating/floating columns. The relatively large number of tests that were compiled allows for testing the hypothesis of the critical column length from the perspective of triaxial loading conditions and for different drainage conditions. To achieve this objective, the degree of improvement in strength was plotted against the slenderness ratio (H_c/D_c) for both undrained and drained tests as indicated in Figure 5(a) and Figure 5(b), respectively.



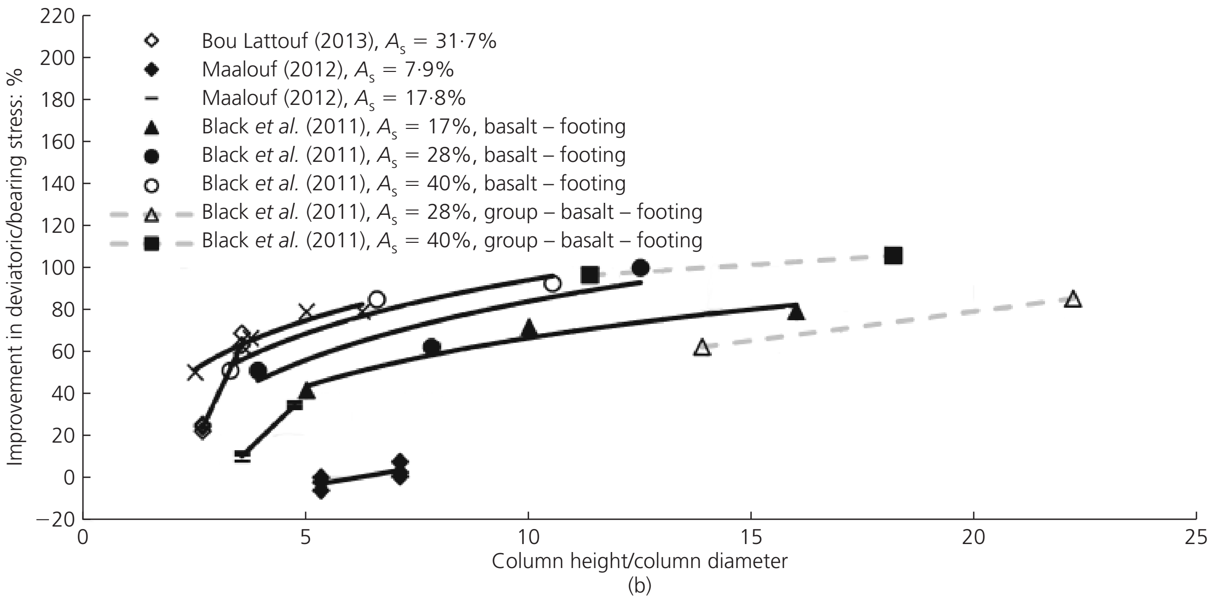


Figure 5. Effect of the slenderness ratio of the column on the degree of improvement in deviatoric/bearing stress for (a) undrained and (b) drained loading conditions

A thorough analysis of the data presented on Figure 5(a) for the 23 undrained triaxial tests (composite specimens) that were analysed in this study leads to the following conclusions. For tests conducted at relatively large area replacement ratios (greater than 12%), the range of the slenderness ratios encountered was relatively narrow, with ratios in the range 2·0–5·0. The relatively small slenderness ratios are related to limitations in the triaxial testing cells, which limit the height of the sample. Since tests with relatively high area replacement ratios are generally associated with columns of relatively large diameters, the limitation in the height of the specimen (and thus the height of the columns) will result in a practical upper bound for the column slenderness ratio. In this relatively small range of slenderness ratios, results indicate a consistent increase in the percentage improvement in undrained shear strength as the slenderness ratio increases. More importantly, the slope at which the percentage improvement increases becomes steeper as the area replacement ratio increases. Since the range of the slenderness ratio was bounded by a ratio of 5·0, no conclusions could be drawn regarding the possibility of a critical column length for these tests (undrained tests with high area replacement ratios).

For undrained tests with area replacement ratios that are less than or equal to 12%, the range of the observed slenderness ratio was larger, with typical ratios in the range 2–10. In these tests, the trends in Figure 5(a) show clearly that as the slenderness ratio increases beyond values of about 5·0–6·0, the rate at which the percentage improvement in undrained strength increases becomes smaller, indicating a relatively smaller contribution for the additional length of column in improving the capacity of the composite. It should be noted that the number of tests that were conducted in some studies was too small to allow for a proper validation of the presence of a critical column length. Examples include the tests reported in Black *et al.* (2007) where only two column penetration ratios were tested for a given area replacement ratio. Two tests do not allow for a proper determination of the variation of the percentage improvement with the slenderness ratio, since the critical ratio could be located anywhere between the two available points.

For the 42 drained tests that were analysed in this paper, results in Figure 5(b) indicate increases in the percentage improvement in the drained capacity of the composites as the slenderness ratio increases from values as low as 2·5 to values as high as 22. As with the undrained tests, the rate at which the percentage improvement in strength increases becomes smaller as the slenderness ratio increases. However, the data do not allow for the proper determination of a critical slenderness ratio after which no increases in strength were obtained. Despite this fact, it is clear from the data that the slenderness ratio after which relatively small improvements in drained capacity were obtained could range from 7 to 10.

It should be stressed that the results presented in Figure 5 and the conclusions emerging from these results pertain to triaxial testing conditions and may not reflect the true field behaviour, for a number of reasons. The most important reason is that the stress field within a triaxial test (sample subjected to a uniform effective confining pressure along its length) is significantly different than in the field where the effective confining stress profiles increase with depth. The difference in the stress fields is particularly significant when analysing the effect of the column length and slenderness ratio on the observed response and failure mode of the reinforced clay.

Based on photographs of exhumed sand columns from 1***g*** model tests and numerical analyses conducted for footings resting on clays reinforced with column groups, Muir Wood *et al.* (2000) report four modes of failure for stone columns (bulging, formation of shear planes, bending and punching into clay). In field applications involving stone columns, as the length of the column increases, the sum of shaft and end bearing resistance increases and the effect of column length is expected to diminish rapidly as the bulging failure mechanism governs. Both the area replacement ratio and the position of the column with respect to the footing boundary conditions are expected to play a role in determining whether bulging or the development of distinct shear zones will become the governing failure mode.

The potential failure modes in field applications involving the use of stone columns to support clays may not be directly correlated/applicable to the failure modes witnessed in the triaxial tests reported in this study. For example, the bulging failure mode for the columns was rarely reported in the triaxial studies presented in this paper. This is directly attributed to the uniform effective confining pressure that exists along the length of the specimen in the triaxial tests, the generally symmetrical conditions that are dictated by the unit cell configuration used in most of the triaxial tests (with a single central column surrounded by a cylindrical clay), and the possible end effects that could exist in the triaxial samples due to the presence of the rigid porous stones and loading plates. All of these factors should be taken into consideration before extrapolating or adopting the conclusions related to the effect of the slenderness ratio on the improvement in capacity in practical field applications.

4. Discussion and conclusion

The analysis of the published results of the 114 triaxial tests that were compiled in this study resulted in several findings that could impact the design practice of granular columns in soft clay. What follows is a summary of the major findings and their relevance to practical design considerations.

(*a*) The magnitude of the percentage improvement in load-carrying capacity of soft clays that are reinforced with granular columns is affected by the drainage conditions. For a given reinforcement condition (area ratio, density of column, etc.), the percentage improvement in strength was found to be higher (almost double) for undrained conditions than for drained conditions. Since the conditions in the field are expected to be governed by partial drainage from the clay to the granular column, future work should target understanding the load response of the composite for realistic drainage conditions and loading rates. This could be done using advanced experimental testing or through numerical modelling, which should consider coupled flow and deformation as a basis for the analyses.

(*b*) Along the same lines, the variation in the magnitude of the percentage improvement in strength with area replacement ratio indicated a concave upward relationship for undrained conditions (larger rates of improvement at higher area replacement ratios) and a concave downward relationship for drained conditions (smaller rates of improvement at higher area replacement ratios). The practical significance of this finding is that for practical applications involving fully drained conditions (long-term stability or relatively permeable clays), there could be an upper bound value for the area replacement ratio (30 to 40% based on this study) beyond which the benefits of increasing the area ratio become economically unjustifiable.

(*c*) As expected, the friction angle of the granular column plays a role in defining the percentage improvement in capacity for the composite. Results of the triaxial tests that were analysed in this study for kaolinite clay indicate that the percentage improvement in the deviatoric stress at failure is expected to increase by 5% for a 1° increase in the friction angle of the column material, irrespective of the area replacement ratio used. This number might differ for other types of clays that could exhibit various levels of plasticity.

(*d*) The concept of the critical column length was tested for the cases involving columns with different slenderness ratios. For both drained and undrained tests, the triaxial test results showed clear evidence that the rate of the percentage improvement in strength decreases significantly as the ratio of the column height to diameter increases beyond values of 5·0 to 6·0 for undrained tests and 7·0 to 10·0 for drained tests. This new evidence from triaxial test results supplements the existing evidence from 1***g*** laboratory model tests and field tests on clays that are reinforced with granular columns.

As mentioned previously, differences in the stress fields and potential failure modes between triaxial and field conditions should be taken into consideration before extrapolating or adopting the conclusions related to the effect of the slenderness ratio on the improvement in capacity in practical field applications involving soft clays that are reinforced with dense granular columns.

Acknowledgement

The authors would like to acknowledge the University Research Board (URB) at the American University of Beirut for funding this research study.

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