The Drained Strength of Soft Clays with Partially Penetrating Sand Columns at Different Area Replacement Ratios

La résistance drainée des argiles molles avec des colonnes de sable pénétrant partiellement à différents taux de remplacement

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ABSTRACT: Granular columnar inclusions are generally used to improve the mechanical properties of soft clays. The objective of this paper is to investigate the long term behavior of clay/sand column composites as represented by the fully drained loading condition, for cases where the soft clay is reinforced by floating or partially penetrating sand columns. For this purpose, consolidated drained triaxial tests (CD) were performed on back-pressure saturated normally consolidated Kaolin specimens that were consolidated and tested at confining pressures of 100 kPa, 150 kPa, and 200 kPa. The sand columns were penetrated to 75% of the depth of the clay sample to represent a partially penetrating condition, while the main parameter that was varied in the study was the area replacement ratio which was varied from 7.9% to 17.8% to 31.2%. Results indicated that the positive effects of sand columns on strength are minimal for small area replacement ratios and increase gradually as the area replacement ratio increases. The average percent improvements observed for area ratios of 17.8% and 31.2% were 20% and 32%, respectively. These results indicate that partially penetrating columns may provide effective strengthening for soft clays, provided that a suitable area replacement ratio is adopted in design.

RÉSUMÉ: L'inclusion de colonnes granulaires est généralement adoptée pour améliorer les propriétés mécaniques des argiles molles. L'objectif de cet article est d'étudier le comportement à long terme des composites d'argile / sable, représentée par la condition de chargement complètement drainée, pour les cas où l'argile molle est renforcée par des colonnes de sable partiellement pénétrantes. A cet effet, des essais triaxiaux drainés consolidés (CD) ont été réalisées sur des spécimens de Kaolin saturés et normalement consolidés. Des échantillons consolidés à des pressions de confinement de 100 kPa, 150 kPa et 200 kPa ont été testés. Les colonnes de sable ont été établis jusqu'à à 75% de la profondeur de l'échantillon d'argile pour représenter un état partiellement pénétrant, tandis que le principal paramètre qui a été modifié dans l'étude était le taux de remplacement qui a varié de 7,9% à 17,8% à 31,2% de la surface de section du spécimen. Les résultats indiquent que les effets positifs de colonnes de sable sur la résistance sont minimes pour les petits taux de remplacement et augmentent progressivement avec l'augmentation du taux de remplacement. Les améliorations observées en moyenne pour des rapports de surface de 17,8% et 31,2% étaient de 20% et 32%, respectivement. Ces résultats indiquent que les colonnes partiellement pénétrantes pourraient être utilisées pour renforcer les argiles molles, à condition que le rapport de remplaçant approprié soit choisi dans le design.

KEYWORDS: soft clay, sand columns, stone columns, consoldiated drained triaxial tests, soil improvement, floating columns

1 INTRODUCTION

Granular columnar inclusions in the form of sand drains/columns or vibrated stone columns are commonly used to improve the mechanical properties of soft clays. Historically, experimental research studies have been designed to investigate the behavior of sand/stone column-reinforced clay systems in the laboratory using 1-g tests that are conducted in one dimensional loading chambers (Hughes and Withers 1974, Muir Wood et al. 2000, Malarvizhi & Ilamparuthi 2004, McKelvey et al. 2004, Ayadat and Hanna 2005, Ambily & Gandhi 2007, Gniel & Bouazza 2009, Murugeson & Rajagopal 2010, and Fattah et al. 2011).

The limitations of 1-g model tests were recognized by many researchers who resorted to testing soft clay specimens that were reinforced with sand/stone columns under triaxial conditions where the stress state, the drainage conditions, and the loading rate could be controlled. Examples of such studies include the work reported in Juran and Guermazi (1988), Sivakumar et al. (2004), Black et al. (2006), Black et al. (2007), Andreou et al. (2008), Najjar et al. (2010), Black et al. (2011), and Sivakumar et al. (2011).

For cases involving sites with deep deposits of soft clay, the use of sand/stone columns that fully penetrate the soft clay layer is prohibitive and may not be practically achievable. As a result, the use of partially penetrating columns is common as a practical soil improvement scheme.

Current design methods for stone columns do not reflect the effect of the degree of column penetration in the soft clay on the response of clay/stone column system. As a result, there is a need for investigating the behavior of clays with partially penetrating columns using an experimental framework in which the stress state and the drainage conditions could be controlled.

The objective of this paper is to investigate the load response of soft clay that is reinforced with partially penetrating sand columns in a triaxial framework. The parameters that were varied in the experimental program are the area replacement ratio which was varied from 7.9% to 17.8% to 31.2% and the effective confining pressure which was varied from 100 to 150 to 200 kPa. All tests were conducted using columns that penetrated the soft clay to a depth that is equal to 75% of the height of the clay sample. Since the sand columns are expected to act as drains that will facilitate radial drainage, fully drained tests were conducted to represent the long term behavior of the clay/stone column system and to provide an upper bound of the response for practical loading conditions in the field where the clay surrounding the columns is expected to be partially drained.

2 EXPERIMENTAL PROGRAM

In total, 12 isotropically consolidated drained (CD) triaxial tests were performed on 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 2cm, 3cm, and 4cm with a column penetration ratio H_c/H_s of 0.75. The 2-cm, 3-cm, and 4cm diameter columns represent area replacement ratios A_c/A_s of 7.9%, 17.8%, and 31.5% respectively. All sand columns were placed in pre-drilled holes in the center of the clay specimens. All specimens were saturated using a back pressure of 310 kPa and isotropically consolidated under effective confining pressures of 100, 150, or 200 kPa. In all tested specimens, the measured "B" value was greater than 0.96 indicating an adequate degree of saturation. Samples were then sheared in drained conditions at a strain rate of 0.25% per hour (~0.06mm/min). All tests were terminated at a maximum axial strain of about 12%.

2.1. Material Properties

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. Consolidation and strength properties for the clay are presented in Najjar et al. (2010). Ottawa sand which classifies as poorly graded sand (SP) according to the Unified Soil Classification System was used to construct the sand columns. For sand specimens prepared at a dry density of 16.2 kN/m^3 (relative density of 44%), Najjar et al. (2010) reported an effective peak friction angle of 33^o based on consolidated undrained triaxial tests with pore pressure measurement. In this study, isotropically consolidated drained triaxial tests were conducted on sand specimens with a height of 14.2 cm and a diameter of 7.1 cm at confining pressures of 100, 150, and 200 kPa to determine the friction angle of the sand. The resulting effective friction angle was found to be equal to 35^o. The difference between the measured effective friction angles from the CU+U and CD tests could be attributed to the respective mean effective stresses at failure which were an order of magnitude greater for the undrained tests.

2.2. Sample 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 one-dimensional consolidation. Dead weights were used to consolidate the specimens from slurry to a vertical effective stress of 100 kPa. The water content at the end of the consolidation stage was relatively uniform (~53%) throughout the depth of the sample. The average bulk density for all the clay specimens prepared was about 16 kN/m^3. A detailed description of the sample preparation and testing procedure is presented in Najjar et al. (2010).

The sand columns were formed from Ottawa sand at a dry density of about 16.2 kN/m^3. These sand columns were prepared by pouring 3 layers of dry Ottawa sand in cylindrical pre-cut and stitched geosynthetic fabrics. The fabrics were initially inserted in a glass tube having the same inner diameter as the sand column, and the sand layers were densified by vibration. Water was then added to the sand column to reach a water content of about 20%. The saturated sand column was then frozen for 24 hours (Fig. 1a). The geosynthetic fabric was cut and detached from the sand column. The frozen sand column was then inserted into a hole drilled at the center of the clay specimen (Fig. 1b) and allowed to thaw. The reinforced clay specimen (Fig. 1c) was then transferred to the triaxial cell and saturated using a back pressure of 310 kPa.

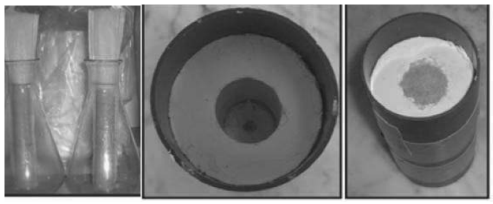


Figure 1. Installation process of sand columns.

3 TEST RESULTS AND ANALYSIS

The automated triaxial test setup "TruePath" by Geotac was used to conduct CD tests on control and reinforced clay specimens saturated at a back pressure of 310 kPa. The samples were then isotropically consolidated under confining pressures of 100, 150, or 200 kPa and sheared drained at a strain rate of 0.25% per hour, while measuring volume change through drain lines connected to the porous stones at the top and bottom of the sample. The measured volume change reflects a global change in the composite sample and do not provide information on local changes in the water content in the sand column and the surrounding clay. Throughout the tests, the total confining pressure was kept constant as the vertical stress was increased in compression.

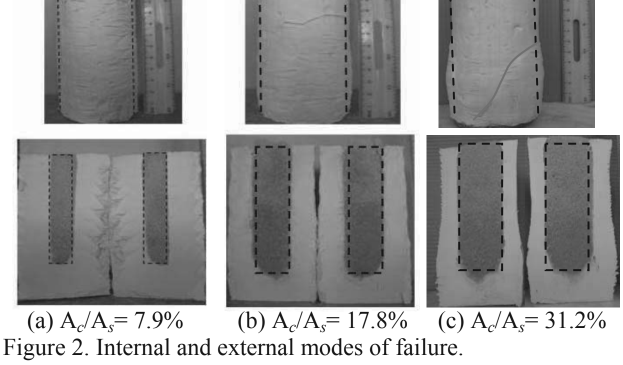
3.1. Mode of Failure

The mode of failure was characterized by bulging of the clay specimen. The bulging was slight and relatively uniform along the height in samples reinforced with the smallest area replacement ratio of 7.9% (see Fig. 2a). As the area replacement ratio increased, the bulging was significant and concentrated in the lower half of the clay specimen, indicating stress and strain concentration in the unreinforced portion of the specimen. For the largest area replacement ratio of 31.2%, clearly defined shear planes formed in the lower half of the sample as indicated in Fig. 2c.

To investigate the mode of failure of the sand columns, the same test specimens were split along their vertical axes to expose the columns and the surrounding clay (Figs. 2a-2c). The figures indicate that relatively uniform bulging of the sand columns occurred with depth, with the specimens at the higher area replacement ratios showing signs of punching of the sand columns into the unreinforced clay.

3.2. Stress-Strain Response

The variation of the deviatoric stress and volumetric strain with axial strain is presented in Figs. 3, 4, and 5 for tests with replacement ratios of 7.9, 17.8, and 31.2%, respectively. The stress-strain curves exhibited consistent increases in deviatoric stresses with strains as the samples were sheared towards critical state conditions. In this paper, failure is defined at an axial strain of 12%, which is the maximum strain measured.



(a) A_c/A_s= 7.9% (b) A_c/A_s= 17.8% (c) A_c/A_s= 31.2%

Figure 2. Internal and external modes of failure.

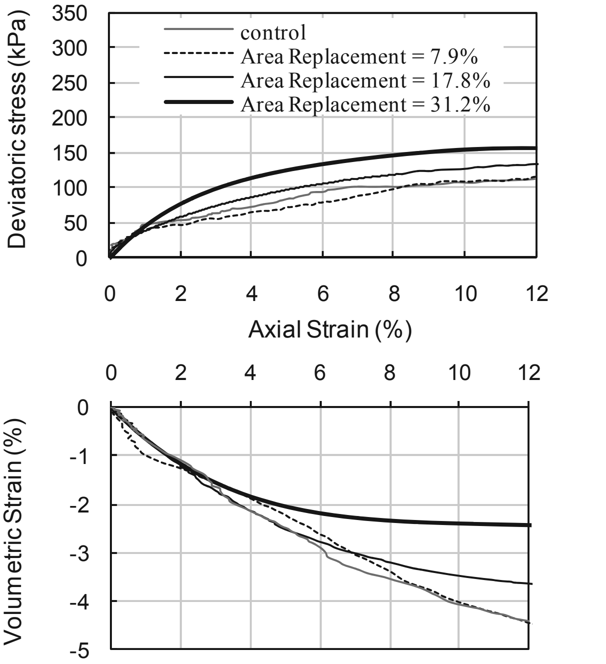


Figure 3. Variation of deviatoric stress and volumetric strain with axial strain (A_c/As=7.9%).

Results presented in Figs. 3 to 5 indicate that for the smallest area replacement ratio used in this study (A_c/As=7.9%), no improvement was evident in the stress-strain response compared to the control clay specimen. In fact, slight reductions in the load-carrying capacity were measured at all levels of strain. For the higher area replacement ratios, clear and consistent improvements in the stress-strain response were observed for all effective confining pressures. With regards to the volumetric strains, results indicate that the measured volumetric strains were contractive for all the specimens tested. However, the volumetric strain at failure was found to decrease for specimens that were reinforced with 3-cm and 4-cm sand columns (higher area replacement ratios), compared to control clay specimens and specimens reinforced with 2-cm columns. This reduction in volumetric strains at failure for the reinforced clay specimens is expected and is due to the dilative nature of the sand columns, particularly at higher area replacement ratios.

3.3. Effect of Sand Columns on Deviatoric Stress at Failure

The percent improvement in the deviatoric stress at failure for the series of tests involving area replacement ratios of 7.9%, 17.8%, and 31.2% was calculated and presented in Fig. 6 as a function of the effective confining pressure. Results in Fig. 6 indicate that the use of 2-cm diameter sand columns (area ratio of 7.9%) did not result in increases in the deviatoric stress at failure. For the higher area replacement ratios of 17.8% and 31.2%, improvements ranging from 17% to 25% and from 28% to 38% were observed in the deviatoric stress at failure, respectively.

The calculated percent improvement in the deviatoric stress at failure was also plotted in Fig. 7 as a function of the area replacement ratio. Interestingly, the results in Fig. 7 indicate that for the smallest effective confining pressure of 100 kPa, the percent improvement increased at the same rate as the area replacement ratio was increased from 7.9% to 17.9% to 31.2%. For the tests conducted at the higher confining pressures of 150 kPa and 200 kPa, the rate of improvement in the deviatoric stress at failure decreased as the area replacement ratio was increased from 17.9% to 31.2%. This decrease in the percent improvement could be attributed to the mode of failure observed for the samples reinforced at an area replacement ratio

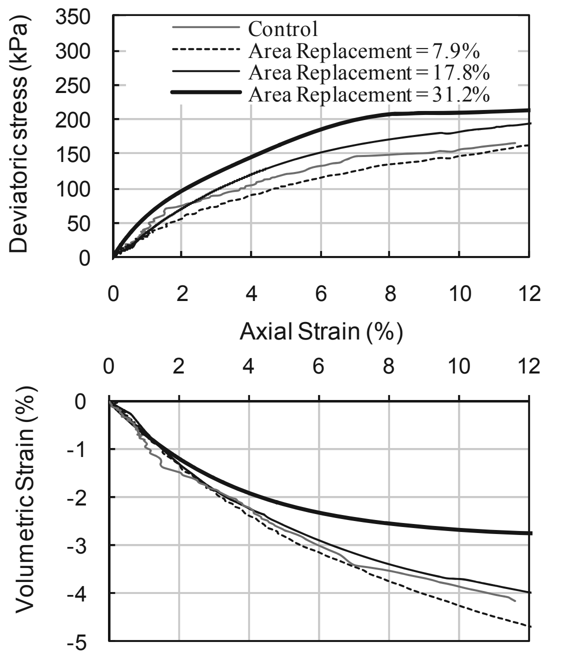


Figure 4. Variation of deviatoric stress and volumetric strain with axial strain (A_c/As=17.8%).

of 31.2% and tested at confining pressures of 150 kPa and 200 kPa. For these cases, clear shear planes formed at the bottom of the sample (see Fig. 2c) indicating a possible premature failure in the lower-half of the sample due to elevated stresses in the sand columns that are bearing on the unreinforced clay.

3.4. Effect of Sand Columns on Shear Strength Envelope

Figure 8 shows the effective Mohr-Coulomb envelopes corresponding to the different area replacement ratios used in this study. As expected the Mohr-Coulomb failure envelope for the specimens reinforced with the smallest area replacement ratio of 7.9% was almost identical to that of the control clay, with an effective apparent cohesion c' = 0 kPa and an effective friction angle of about 21^o.

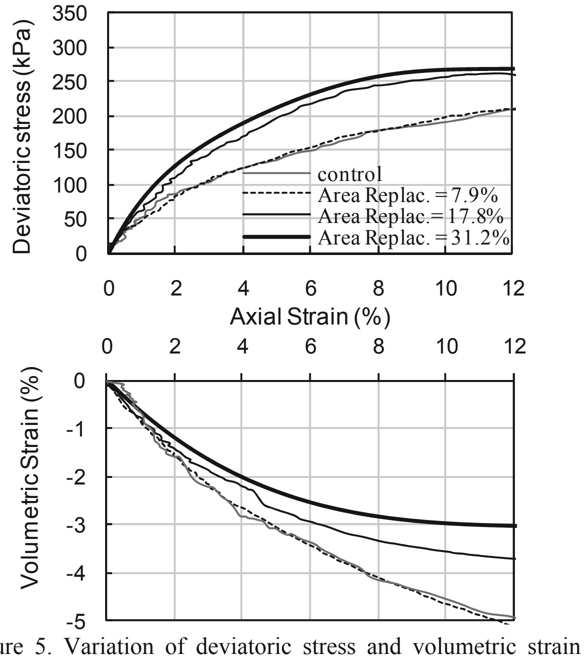


Figure 5. Variation of deviatoric stress and volumetric strain with axial strain (Ac/As=31.2%).

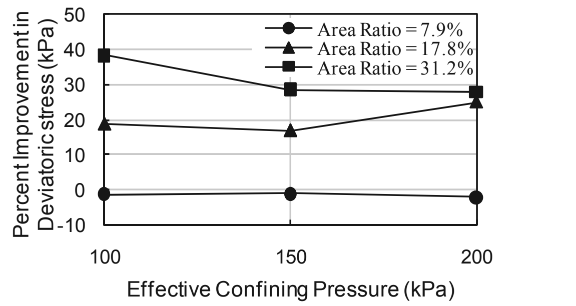


Figure 6. Dependency of the percent improvement in deviatoric stress at failure on the effective confining pressure

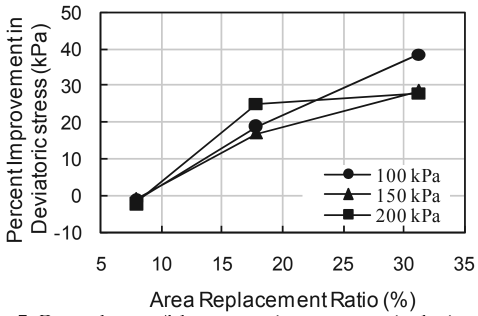


Figure 7. Dependency of the percent improvement in deviatoric stress at failure on the area replacement ratio.

For specimens reinforced with an area replacement ratio of 17.8%, the friction angle ϕ' was found to increase to 23^o (compared to 21^o for the control clay) with the effective cohesion intercept *c'* remaining at zero. On the other hand, samples with an area replacement ratio of 31.2% showed no improvements in the friction angle compared to the control specimens ($\phi'=21^o$), but were associated with a non-zero *c'* value of 18 kPa. The non-zero *c'* could be related to the relative reduction in the percent improvement in the deviatoric stress for samples tested at the higher confining pressure of 150 kPa and 200 kPa as indicated in Figs. 6 and 7.

4. CONCLUSIONS

Based on the results of 12 consolidated drained triaxial tests the following conclusions can be drawn on the effect of partially penetrating sand columns on the drained response of soft clay:

1. The mode of failure of the test specimens was governed by bulging that was relatively uniform for specimens reinforced at a small area replacement ratio of 7.9% and concentrated in the lower half of the specimens for the higher area replacmenent ratios of 17.8% and 31.2%. For an area ratio of 31.2%, specimens tested at confining pressures of 150 kPa and 200 kPa exhibited clear shear planes in the lower half of the specimens indicating elevated stress concentrations in the unreinforced clay.
2. The specimens tested with the lower area replacement ratio of 7.9% did not show any improvement in the load carrying capacity. For the higher area replacement ratios of 17.8% and 31.2%, average improvements of 20% and 32% were observed in the deviatoric stresses at failure, respectively. For the higher confining pressures of 150 kPa and 200 kPa, the rate of improvement in the deviatoric stress at failure was found to decrease as the area replacement ratio was increased from 17.9% to 31.2%. This could be due to the premature formation of shear planes in the lower half of the specimens
3. An analysis of the Mohr-Coulomb envelopes indicated that for an area replacement ratio of 17.8%, ϕ' increased from 21^o (control clay) to 23^o, while for an area replacement ratio of 31.2%, *c'* increased from 0 (control clay) to 18 kPa with ϕ' remaining constant at 21^o.

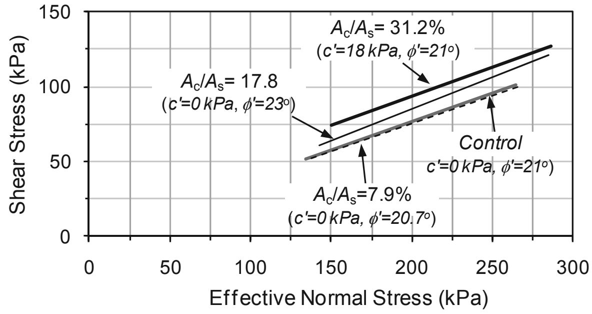


Figure 8. Failure envelops for control and reinforced clay specimens.

5. ACKNOWLEDGEMENTS

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

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