Proceedings of the 14th European Conference on Soil Mechanics and Geotechnical Engineering, Madrid, Spain, 24-27 September 2007

Comptes Rendus du XIVème Congrès Européen de Mécanique des Sols et de la Géotechnique Madrid, Espagne, 24-27 Septembre 2007

Geotechnical Engineering in Urban Environments

Les Problèmes Géotechniques en Milieu Urbain

Volume 3

Edited by
V. Cuéllar, E. Dapena, E. Alonso, J.M. Echave, A. Gens, J.L. de Justo,
C. Oteo, J.M. Rodríguez-Ortiz, C. Sagaseta, P. Sola, A. Soriano

Millpress

Millpress Science Publishers, Rotterdam, The Netherlands

A laboratory model study of the performance of vibrated stone columns in soft clay

Etude en modèle de laboratoire des performances de « vibrated stone columns » dans de l'argile mou

V. Sivakumar
Queen's University Belfast, United Kingdom
D Glynn
Byrne Looby Partners (Formerly CPD), Northern Ireland, United Kingdom
J Black
University of Plymouth, United Kingdom
J McNeill
Keller Ground Engineering, United Kingdom

ABSTRACT

The vibrated stone column technique is an economical and environmentally friendly process that treats weak ground and enables it to withstand low to moderate loading conditions. The performance of the treated ground depends on various parameters such as the strength of the *in situ* natural deposits and/or backfill materials, together with the spacing, length and diameter of the columns. In practice, vibrated stone columns are frequently used for settlement control. Studies have shown that columns can fail by bulging, bending, punching or shearing. These failure mechanisms are examined in this paper.

The study involved a series of laboratory model tests on a consolidated clay bed, using transparent material with 'clay like' properties. The tests, probably for the first time, have permitted visual examination of the behaviour of granular columns during the loading process. They show that significant deformation in the form of bulging occurs in long columns, while punching was prevalent in shorter columns. The presence of the columns also greatly improved the load carrying capacity of the soft clay bed. However, columns longer than about six times their diameter did not lead to any improvement or increase in the load carrying capacity. This suggests that there is an optimum column length for a given arrangement of stone columns beneath a rigid footing.

RÉSUMÉ

La technique dite "vibrated stone column » est un processus économique et respectueuse de l'environnement qui traite le sol lâche et lui permet de supporter des charges basses ou modérées. La performance du sol traité dépend de plusieurs paramètres tels que la résistance des dépots naturels in situ et/ou des remblais, ainsi que de l'espacement, de la longueur et du diamètre des colonnes. En pratique, les "vibrated stone columns » sont fréquemment utilisées pour le contrôle de stabilité. Différentes études ont montré que ces colonnes peuvent échouer pour cause de bulging, fléchissement ou cisaillement. Les mécanismes provoquant l'échec sont examinés dans cet article.

L'étude a inclus une série de tests en modèle de laboratoire sur un socle d'argile consolidé, comportant du matériel transparent aux propriétés similaires à l'argile. Les tests, probablement pour la première fois, ont permis l'examination visuelle du comportement des granuler columns au cours du processus de charge. Ils montrent qu'une déformation significative sous forme de bulging apparaît dans les longues colonnes, alors que le puncheur prévaut dans les colonnes plus courtes. La présence des colonnes a grandement amélioré la capacité de charge des socles d'argile. Toutefois, des colonnes plus longues qu'environ six fois leur diamètre n'ont pas amélioré la capacité de charge. Ceci suggère qu'il existe une longueur de colonne optimale pour un arrangement donné de stone columns sous un rigid footing.

Keywords: Ground Improvement, Granular columns, Settlement, Bearing Capacity

1 INTRODUCTION

Vibrated stone columns have been used extensively over the last three decades in Europe, Asia and the United States in a wide range of ground improvement projects. They are mainly used in weak soils to increase load carrying capacity and reduce total and differential settlements of light structural foundations. They are suitable for supporting foundations beneath low-rise housing, retail developments, industrial warehouses, waste treatment plants and car parks. This technique works satisfactorily in cohesive soils when the strength of the *in situ* material is greater than about 15 kPa (Bachus and Barksdale, 1983). The vibro stone column process usually involves replacing 10% - 35% of the *in situ* soil with crushed rock. Various methods have been adopted for installing granular columns in soft deposits to various depths. A detailed description of these methods is given by Greenwood and Kirsch (1984) and McKelvey (2002). Although there are no upper limits regarding the depth of treatment, typical column lengths range from 3m to 15m, depending on soil type, etc.

In an attempt to understand and predict the behaviour of vibrated stone columns, many studies - based on physical modelling, mathematical analysis and full-scale testing - have been carried out. These studies have highlighted the various parameters that influence overall performance of the technique. They include - column diameter, column length, column spacing, area replacement ratio, size and flexibility of the footing, strength of the *in situ* soil, strength of the column material and method of installation. However, it should be noted that experimental evidence is limited, particularly for small groups of columns supporting footings.

Laboratory research by Hughes and Withers (1974), Charles and Watts (1983), Bachus and Barksdale (1984) and Hu (1995) has helped considerably in understanding the behaviour of stone columns. Research has also been extended to full-scale studies in which observed behaviours are in general agreement with laboratory findings (Watts and Serridge, 2000, Greenwood, 1991, Bell *et al*, 1986, Munfakh, 1984 and Hughes *et al* 1975). These laboratory-based, full-scale field studies have concentrated mainly on the behaviour of isolated columns or very large groups of small columns beneath raft footings. They show that columns can fail by bulging, bending, punching and shearing. Punching failure is predominant in short columns, though this mode of failure may cease to exist when the column length to diameter ratio exceeds a value of around 6. It is unlikely that any further increase in column length may contribute towards enhancing bearing capacity, though it may have some impact on settlement performance. Long columns, therefore, fail in bulging and columns located eccentrically under footings can also fail in bending. There have been many postulations on the mode of failure by many researchers, yet there appears to be no direct evidence to support any of the claims. This paper addresses these issues and examines the deformation characteristics of columns using a novel testing method.

Previous researchers have used various methods of examining ground deformation under applied surface loads. Within the context of vibrated stone columns, X-ray techniques have been used successfully to monitor the deformation of an isolated granular column and surrounding clay (Hughes and Withers, 1974). Although this method was successful, the technique is expensive and raises various health and safety issues when used in a laboratory environment. In a recent study, Hu (1995) investigated the failure mechanisms of a large group of columns (beneath a large raft footing) by exhuming the column material and forming plaster-casts of the vacated holes or voids at the end of the test. This allowed the failure patterns of the columns to be examined when the loading cycle had terminated. The work reported in this paper reveals, probably for the first time, the process of deformation that takes place in a stone column-reinforced clay bed during loading. An artificial transparent clay-like material was used in the test model to represent the *in situ* soil surrounding the granular columns.

2 EXPERIMENTAL PROGRAMME

Two large loading chambers were designed and manufactured for the purpose of producing test beds of soft clay for one-dimensional consolidation. A transparent material - described later in the paper - was used to model the soft clay bed. Figure 1 shows a photograph of the loading chamber. Due to the high compressibility of the transparent material, the chambers had to be approximately 1.2m in height to provide a sufficiently thick clay bed and thereby accommodate the large order of settlements expected for each consolidation test -about 500mm . The top section of the loading chamber consisted of an acrylic cylinder while the bottom section was made up of a series of stainless steel rings. The internal diameter of the chamber was 413mm. The consolidation chamber was supported on a metal frame. A small triaxial compression machine, mounted in an inverted position on top of the frame, allowed for the application of a constant rate of load to the model foundation. Full details on this aspect of the model can be seen in the research carried out by McKelvey et al 2003.

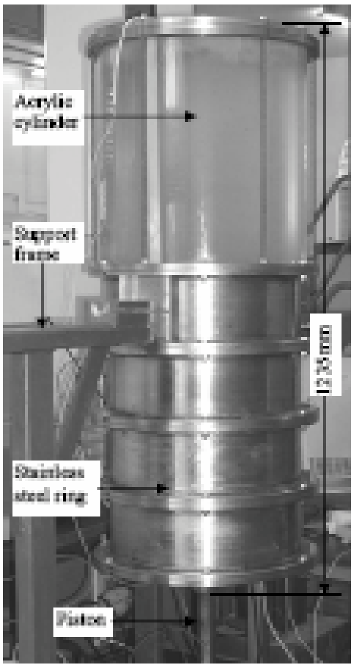


Figure 1 Consolidation chamber

In the present study, a transparent material with properties similar to those of soft clay was used to model a soft clay bed. The material was developed in Trinity College Dublin to mimic the geotechnical properties of natural clay in an attempt to understand ground deformation during cone penetration tests. Basic characteristics of the material were investigated by Gill (1999). Further tests were carried out as part of the present study in order to gain a better understanding of its mechanical properties and to further explore its use as a key constituent in geotechnical research.

The material was prepared by mixing fumed silica in an oil blend of mineral spirits and crystal light liquid paraffin. The properties of these materials are presented in Table 1. The slurry was 7% fumed silica by weight. The oil blend was 70% liquid paraffin and 30% mineral spirits. There are three main features that contribute to the excellent optical transparency of the material: (a) the silica and the oil blend have the same refractive indices; (b) the fumed silica aggregates are made up of ultra fine particles with individual diameters in the order of 0.014μm. Since these particles are smaller than the wavelength of light, they do not cause the light to scatter, and (c) the porous silica particles have high surface energy levels in oil and adsorb large quantities of pore fluid. Air, therefore, is completely displaced from the micro pores.

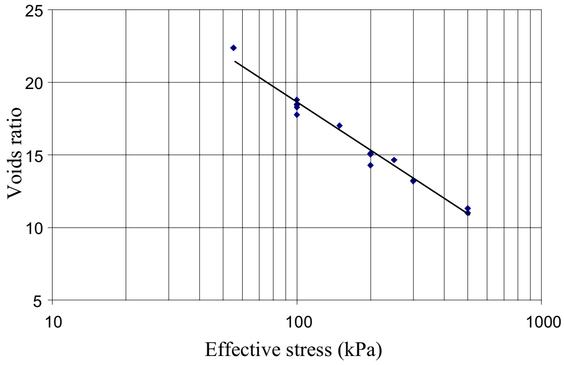


Figure 2 Pressure-volume characteristics

A series of one-dimensional loading tests was carried out in a Rowe cell in order to characterize the material. The Coefficients of Consolidation, c_v, were in the range of 0.8 - 1.2 m^2/year and are comparable with values typical of normally consolidated alluvial clays (Head, 1994). Figure 2 shows the pressure-volume characteristics. The Coefficients of Compressibility, m_v, ranged from 1.3 to 3.4m^2/MN and, again, these values are similar to those of normally consolidated and organic alluvial clays.

After mixing, the slurry of the transparent clay was initially consolidated under a vertical pressure of approximately 70kPa (Figure 1) by elevating air pressure in the lower part of the consolidation chamber. Once the consolidation was complete the applied pressure was reduced to zero and the centre section in the top plate of the chamber was removed to allow the columns to be installed. A special drilling rig, consisting of a 25mm diameter helical auger, was used to drill into the consolidated transparent clay. Dry sand was poured through wire mesh and allowed to fall from a fixed distance of 115mm above the surface of the consolidation chamber to form the columns. Once the columns had been installed the centre section in the top plate of the chamber was relocated in position, but this time with a required footing (circular or strip) inserted in the middle (Figure 3). The footing was secured in position using a cross bar fed through two tie rods located on the top plate of the loading chamber. The pressure in the lower chamber was increased to 85kPa in TS-01, 130kPa in TS-02 and 100kPa in TS-03 and TS-04 respectively. The clay was then allowed to consolidate under the new consolidation pressure.

Three sand columns, measuring 25mm diameter, were installed in a triangular arrangement beneath the 100mm diameter circular footing and introduced as a single row beneath the rectangular strip footing (measuring 100mm x 50mm) to the respective depths of 150mm and 250mm for each model foundation. This corresponds to L/d ratios of 6 and 10 respectively, where L is the column length and d is the column diameter 9or width of footing). Displacement-controlled loading was applied to the model footing at a rate of 0.0064mm/min until the footing penetrated approximately 35mm into the clay.

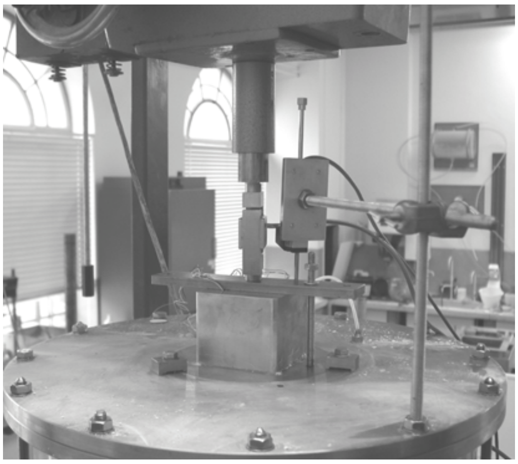


Figure 3 Loading arrangement

A digital camera was mounted outside the acrylic section of the loading chamber and photographs were taken of the deforming columns every six hours. This time period corresponds to footing penetrations of 2.3mm. During foundation loading, the consolidation pressure was maintained constant at the bottom of the chamber while the model footing was loaded at the top. A load cell, attached to the shaft of the triaxial loading apparatus, and a linear displacement transducer were used to measure the load and penetration of the footing respectively.

3 RESULTS AND DISCUSSION

Figures 4 and 5 show load-displacement relationships for the circular footing tests (TS-01 and TS-02) and the strip footing tests (TS-03 and TS-04) respectively. In each case, the pressure on the footing is normalised with respect to the pressure applied to consolidate the clay. Side friction between the clay and the loading chamber caused attenuation of the vertical pressure that was applied at the bottom of the consolidation chamber. As a consequence, the vertical consolidation pressure at the top of the layer was significantly lower than the pressure applied in the lower chamber. Figure 4 shows that the footing begun to penetrate into the clay once the load had overcome the thrust exerted by the clay. It suggests that the vertical pressure at the top of the clay layer was only about half of the pressure applied at the bottom.

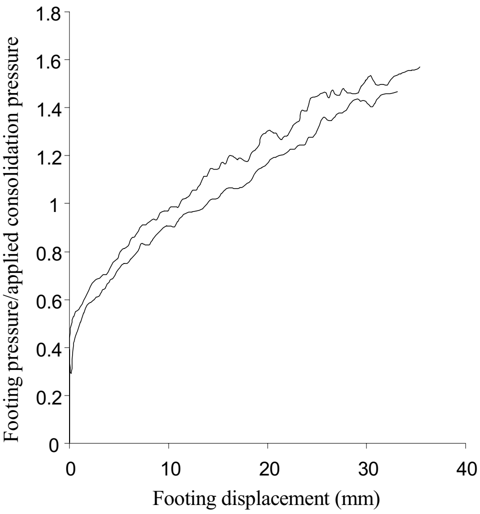


Figure 4 Normalized bearing pressure-displacement (circular footing)

Following the initial rapid increases in load to pre-consolidation pressures, the footing pressures in these transparent material tests increased at relatively steady rates, even after footing displacements of 35mm occurred. This was almost certainly due to the rigid confinement provided by the loading chamber, particularly the restriction imposed at the top of the clay. In the circular footing tests, an assessment of the load bearing capacity at a footing displacement of 20mm indicated that the performance of the composite sample increased by about 5% when the column length increased from 150mm to 250mm. This improvement persisted throughout the entire footing displacement. The loading curves in the strip footing tests (Figure 5) are similar to the circular footing tests, although there was a more significant increase in the load carrying capacity when the column length increased.

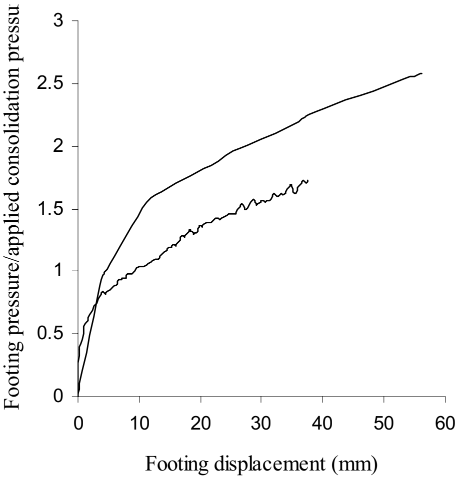
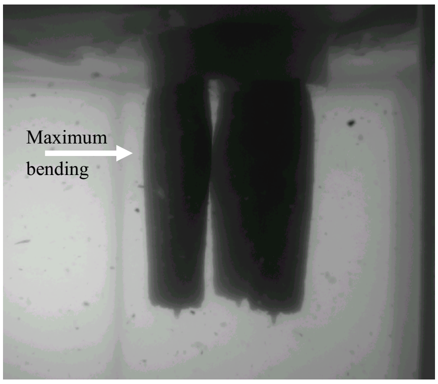
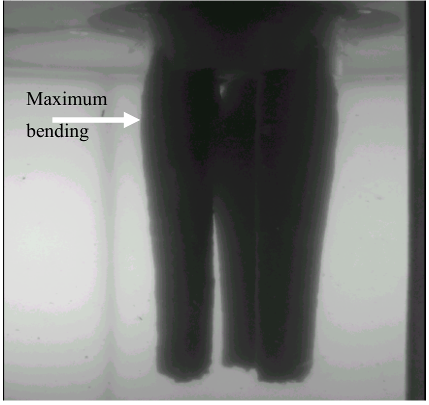


Figure 5 Normalized bearing pressure-displacement (strip footing)

Figures 6 and 7 show photographs of the deformed columns at the end of the loading process for the circular and strip footing tests respectively. Approximately 18 photographs were taken in total during each test but not all are shown here. It should be noted that the scale of the photographs in each of the tests is different. In the circular footing tests (Figure 6), the camera was not situated immediately in front of the columns. Instead, it was positioned at a slight angle in order to capture the entire deforming shape of at least one of the columns throughout the test. In the strip footing tests the camera was positioned perpendicular to the row of columns. The photographs show that both short and long columns bulged in the unrestrained directions (away from neighboring columns) as the foundation load increased. In the case of the short columns (L/d = 6), bulging was less significant, though detailed analysis of the image reveals that the columns punched into the clay, thereby implying that the failure mechanism was "punching". The longer columns (L/d = 10) deformed significantly in the upper region. The foundation penetrated about 35mm, though no significant penetration of the column into the bearing clay was observed. It may be assumed, therefore, that there was little or no load transferred to the lower parts of the longer columns.

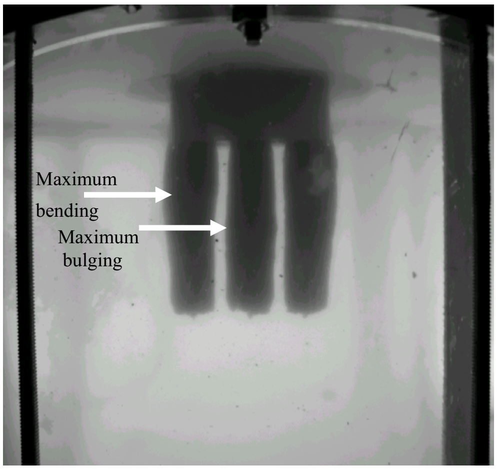


(a) Short column

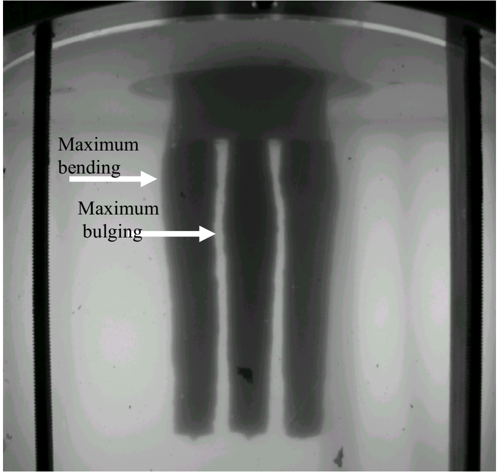


(b) Long column

Figure 6 Deformed columns under circular footing



(a) Short column



(b) Long column

Figure 7 Deformed columns under strip footing

Figures 6 and 7 clearly show that there are at least three different types of failure mechanism:

(a) punching in which the column penetrates the bearing clay;

(b) bulging failure where the column expands symmetrically into the surrounding clay. The degree of expansion largely depends on the strength of the insitu clay. The results show that maximum bulging takes place around 3 times the diameter of the column, regardless of the geometry of the foundation ie circular or strip.

(c) bending of column. This particular mechanism is observed in both strip and circular footings. The tendency to 'bending' is caused by eccentric loading, either applied on the footing or the generated within the clay bed itself. Figure 6b shows the deformed shape of the columns under a circular footing. The three columns were located on a triangular configuration, leaving no column at the centre. Application of loading on the footing is not eccentric, though the column was subjected to eccentric loading due the away stress regime developed in the clay. The clay under the footing enclosed by three long columns is subjected to high degree of straining when the foundation penetrates the clay bed. This can result in a build-up of pressure in this region causing the columns to expand laterally outwards. Observations indicted in Figure 6b show that maximum bending takes place at a location about 1.5 times the diameter of the column.

Three columns were located under the strip footing and one of the columns was located in the middle. In this case, the middle column is subjected to axial loading and the column did not fail in bulging. There is no evidence of bending failure because the loading was generally symmetric. However, the adjacent columns have undergone bending failure as a direct result of eccentric loading arising from the bulging middle column bulging during loading. Once again, the bending was generally prevalent at 1.5 times the diameter of the column. It should be noted that the 'external' columns – outside the centre - also bulged, though the tendency to bending is more prominent

4 CONCLUSIONS

The observations made in the present study may have significant implications in practice. It shows that shorter columns (ie. possibly L/d > 6) give adequate bearing capacity, however, longer columns may be needed to control settlements. Accordingly, rational decisions can be taken to tailor design of stone column installations to achieve maximum performance at optimum cost.

Direct observations of columns subjected to loading from circular and strip footings reveal that such columns can fail in three different ways : bulging, punching and bending. Punching is more prevalent in short column whilst bending failure is predominant in 'perimeter' columns located beyond the centre of the footing. Bulging was more generally common in long columns.

ACKNOWLEDGEMENTS

The authors thank Dr Bell, Keller Ground Engineering UK for funding this project.

REFERENCES

Bachus, R.C. and Barksdale R.D. (1984) 'Vertical and lateral behaviour of model stone columns', *Proc. Int. Conf. on In Situ Soil and Rock Reinforcement,* Paris, 99-104.

Bell, A.L., Kirkland, D.A., and Sinclair,A. (1986). Vibrated-replaced ground improvement at General Terminus Quay. Glasgow, *Building on marginal and derelict land,* Thomas Telford Ltd, London pp 499-514.

Charles, J.A. and Watts, K.A. (1983) 'Compressibility of soft clay reinforced with granular columns', *Proc. 8th European Conf. on Soil Mechanics and Foundation Engineering,* Helsinki, 347-352.

Gill, D.R. (1999) 'Experimental and theoretical investigations of pile and penetrometer installation in clay', *PhD Thesis,* Trinity College Dublin.

Greenwood, D.A. (1991) 'Load tests on stone columns', *Deep foundation improvements: design, construction and testing,* ASTM STP 1089*,* Philadelphia, 148-171.

Greenwood, D.A. and Kirsch, K. (1984) 'Specialist ground treatment by vibratory and dynamic methods', *Proc. Int. Conf. on Piling and Ground Treatment,* Thomas Telford Ltd., London, 17-45.

Head, K.H. (1994) 'Manual of soil laboratory testing – Volume 2', 2nd edn. *John Wiley & Sons, Inc.*

Hu, W. (1995) 'Physical modelling of group behaviour of stone column foundations', *PhD Thesis,* University of Glasgow.

Hughes, J.M.O and Withers, N.J. (1974) 'Reinforcing of soft cohesive soils with stone columns', *Ground Engineering*, 42-49.

Hughes, J.M.O., Withers, N.J. and Greenwood, D.A. (1975) A field trial of the reinforced effect of a stone column in soil, *Ground Treatment by Deep Compaction*, Instn. Civil Engineers., London, 31-44.

McKelvey, D. (2002) 'The performance of vibro stone column reinforced foundations in deep soft ground', *PhD Thesis,* Queen's University Belfast.

Munfakh,G.A.(1984) Soil Reinforcement by stone columns – varied case applications, *Int. Conference on Insitu Soil and Rock Reinforcement*, Paris, 157-162.

Watts, K.S. and Serridge, C.J. (2000) A trial of vibro bottom-feed stone column treatment in a soft clay soil. *Proc. 4th Int. Conference on ground Improvement Geosystems – grouting, soil improvement and geosystems including reinforcement. Helsinki,* June 7-