Small Scale Physical Model Tests Evaluating the Settlement of Vibrated Stone Columns

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ABSTRACT

This paper reports on a series of laboratory experiments carried out to investigate the settlement performance of a rigid foundation supported on soft soil reinforced with Vibrated Stone Columns. A large triaxial system capable of testing samples of 300 mm diameter was developed. The parameters investigated included area replacement ratio (A_s) and column length to diameter ratio (L/d) beneath the footing. The significant findings of the work include (i) optimisation of A_s for settlement performance and (ii) the importance of foundation shielding to provide adequate column confinement.

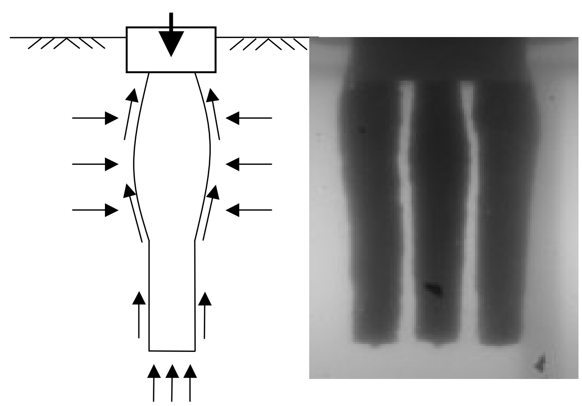
RÉSUMÉ
Cet article rapporte une série d'essais de laboratoire effectués afin d'étudier la performance en tassement d'une fondation rigide supportée sur un sol meuble renforcé par Colonnes de Pierre par Vibro-remplacement. Un system triaxial de grandes dimensions pouvant mettre à l'essai des échantillons de 300 mm a été conçu. Les paramètres étudiés incluent le rapport d'aire de remplacement (A_s) et le rapport de la longueur au diamètre de la colonne (L/d) sous la semelle. Les principales conclusions de ces travaux incluent (i) l'optimisation de As pour la performance de tassement et (ii) l'importance de la protection de la fondation afin d'assurer un confinement suffisant de la colonne.

1 INTRODUCTION

Sites that contain unfavourable ground conditions often require treatment to enhance the mechanical properties of the deposit in order to make it fit for purpose. In this respect a wide range of ground remediation techniques have been developed to deal with problematic ground conditions (Raison 2004, Moseley and Kirsch 2004, Moseley and Priebe 1993, Bergado et al. 1994, Bell 1993, and Kamon and Bergado 1991). A method that has witnessed significant application is that of Vibrated Stone Columns as they are considered highly versatile in treating soft cohesive soils and mixed fills of variable geotechnical properties (Raju 2004, Kirsch and Sondermann 2003, Slocombe and Moseley 1991, Rodgers 1979 and Hughes and Withers 1974).

Vibrated Stone Columns are primarily used to improve the bearing capacity of soft deposits so that more economic shallow foundation schemes may be employed. Additional benefits associated with the technique include (i) increased stiffness of the composite soil-column matrix thereby limiting foundation settlement and (ii) accelerated dissipation of excess pore water pressures. The effectiveness of the technique is largely influenced by various parameters such as area replacement ratio (A_s), column length to diameter ratio (L/d), stiffness of the column, the strength of the surrounding material and the number of columns beneath the footing.

Stone columns are non-rigid structural elements inserted into the ground which increase the stiffness of the native deposit therefore providing increased bearing capacity and reduced total/differential settlements. During loading, columns develop end bearing and side frictional resistance in a similar fashion as piles. Furthermore, columns also expand transversely and therefore acquire additional shear resistance from the surrounding soil (Figure 1). Increased lateral stresses in the surrounding soil due to column bulging enhance consolidation of the soft clay and the overall performance of the column is controlled by the lateral support provided by the surrounding clay, which typically increases with depth.



Many of the current design approaches used by industry have originated from empirical investigations. Several key laboratory investigations in relation to stone columns involved the assessment of bearing capacity. Early work by Hughes and Withers (1974) evaluated the ultimate load capacity based on the cavity expansion theory proposed by Gibson and Anderson (1961). This approach was validated by a full scale investigation (Hughes et al. 1975); however, the analysis was shown to be sensitive to correct estimation of installed column diameter which is difficult to evaluate in practice.

Assessment of large group behaviour and failure mechanisms were extensively reported by Hu (1995) while the performance of small group behaviour beneath pad and strip footings was investigated by McKelvey (2002). The latter study was conducted using transparent 'clay-like' material which allowed an opportunity to view the column-clay interaction and potential failure mechanism during foundation loading (Figure 1). Image analysis has shown that for undrained conditions short columns (L/d <6) failed in end bearing and longer columns (L/d >6) failed in bulging. These observations agreed with previous postulations by Wood et al. (2000) and Hughes and Withers (1974).

Solutions developed for bearing capacity are reasonably well researched understood; however, information in relation to settlement performance is scarce. This is principally due to limitations in the testing/modelling techniques currently available which has hindered potential progress and understanding. Current test configurations consider samples that are simply consolidated and restrained one dimensionally during loading. The boundary conditions associated with this test set-up are problematic for the evaluation of settlement as it necessitates an effective stress analysis.

This paper describes the development of an innovative large triaxial test set-up to overcome testing limitations experienced in previous small scale model tests and presents settlement results from a series of model tests on isolated columns.

2 EQUIPMENT DEVELOPMENT

2.1 Proposed methodology and system design

Previous testing difficulties experienced relate to the frictional resistance between the clay and the consolidation chamber. Furthermore, column loading was typically carried out on samples that were simply consolidated and restrained one-dimensionally during foundation loading (McKelvey 2002 and Hu 1995). The major difficulties of this approach are that (i) frictional resistance and the rigid boundary condition between the clay and the consolidation chamber leads to none vertical pressure distribution which compromises the homogeneity of the sample in relation to its stiffness and strength (Anderson et al. 1991, Navaneethan 2003, McKelvey 2002 and Black 2007) and (ii) lack of pore water pressure control for effective stress analysis for settlement evaluation. The present study considered samples that were initially consolidated one-dimensionally before they were transferred to the large triaxial cell for further testing. The new system has several advantages such as (i) flexible lateral boundary conditions, (ii) confining pressure/pore water pressure control and (iii) a dual loading system whereby independent application of vertical surcharge and foundation loading occur simultaneously. A brief

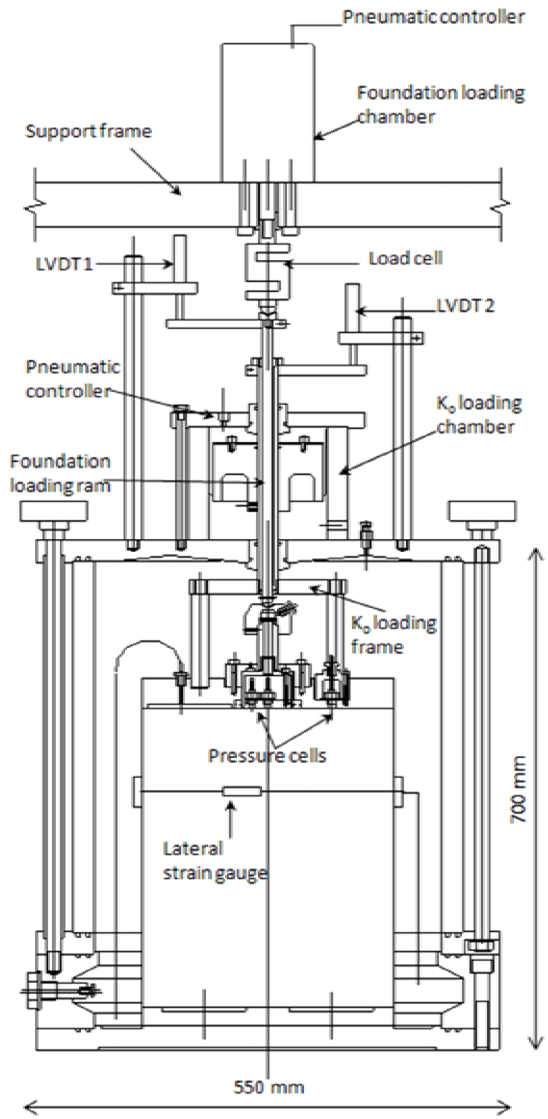


Figure 2. Design of experimental triaxial system.

description of this equipment is given below however a detailed review of this system is reported in Black (2007).

A large triaxial cell capable of testing 300 mm diameter by 400 mm high samples was designed and manufactured (Figure 2). A unique feature of this system is that it incorporates a dual axial loading system allowing samples to be consolidated under K_0 condition while applying an independent load on a small foundation 60mm in diameter. Pressure cells beneath the footing measure the pressure in the column (PT1) and soil (PT2). A third pressure cell (PT3) located on top plate measured the vertical pressure generated in the surrounding soil (Figure 3).

In addition the lateral strains were monitored using a strain gauge located 120 mm from the top of the sample. The confining pressure, foundation loading and pore-water pressures were controlled using pneumatic pressure controllers. An image of the final system is presented in Figure 4.

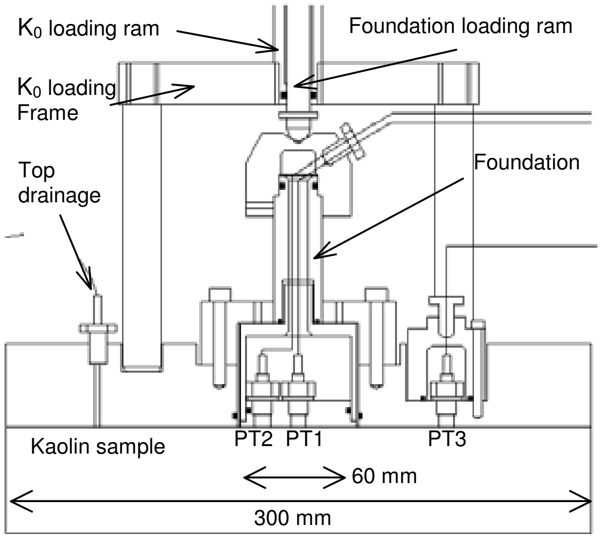


Figure 3. Dual loading systems and sample top cap instrumentation.

3 EXPERIMENTAL INVESTIGATION

3.1 Sample Preparation and Test Procedure

The standard approach to make large quality samples used by many researchers is that similar to a Rowe Cell configuration (Rowe and Barden 1966) where a rolling convoluted rubber jack is attached to a piston and compressed air is used to apply vertical pressure. This particular technique works satisfactorily in short consolidation chambers; however, difficulties such as over stretching of the bellow and loss of consolidation have been reported when using this method in longer chambers (Emmett 2007, Navaneethan 2003, McKelvey 2002 and Anderson et al. 1991). To mitigate this problem an innovative arrangement was adopted using an inflatable “O” ring (Figure 5a) located in the piston plate.

The consolidation chamber (Figure 5b) was fabricated from a polyethylene mains water pipe machined to leave a bore of 300 mm and height of 900 mm. The top and bottom plates of the chamber were manufactured from aluminium and were fitted with porous filter discs and drainage facilities. The piston plate was manufactured from PVC and was 298 mm in diameter by 60 mm thick (Figure 5a). An air line was connected to the inflatable “O” ring so that the pressure could be adjusted during the consolidation process. Both the tube and consolidation pressures were controlled using individual pressure regulators coupled with pressure transducers and digital displays.

Samples were produced by consolidating 70 kg of kaolin slurry in a one-dimensional mould (Figure 5b). A consolidation pressure of 150 kPa was used to produce samples with undrained shear strength (c_u) of 32 kPa. A 75 kPa pressure difference between the rubber tube (225 kPa) and consolidation pressure (150 kPa) provided successful sealing with the chamber side wall and

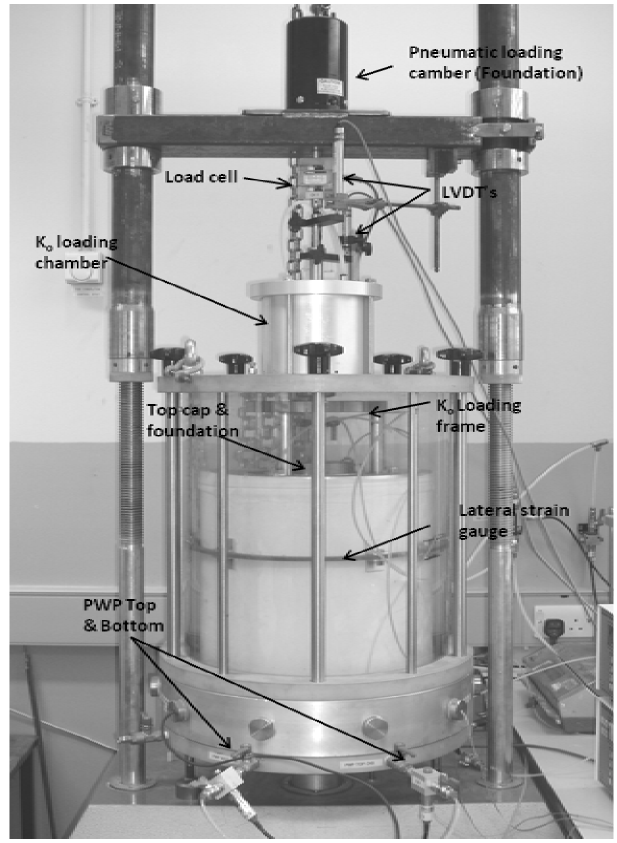


Figure 4. Triaxial cell apparatus.

prevented consolidation pressure being lost. The system was left to equalise for 24 hours prior to opening the drainage line located at the base of the chamber. 95 % consolidation was achieved in approximately 14 days after which the consolidation and tube pressures were reduced. The sample was extracted with the aid of a specially fabricated sampling table. The chamber was inverted allowing the sample was to slide slowly under its own weight into position on the triaxial pedestal resting on the vertical mobile table beneath (Figure 5c).

Stone columns were installed into preformed hole using wet compaction of uniformly graded basalt (1-1.5 mm particle size). The holes were formed by a helical auger that was mounted onto a specially manufactured drilling rig. Previous research by Hu (1995) and McKelvey (2002) showed this installation method to be highly repeatable, though a limitation is that it does not produce dynamic installation effects typically observed in practice. The average column density was 1648 kg/m^3 with a variation of ±43 kg/m^3. One-dimensionally consolidated samples were subjected to three further stages of loading within the triaxial cell of isotropic compression, K_0 loading and foundation loading as indicted in Table 1. In stage 2 and 3 the loadings were applied in ramped fashion using pneumatic pressure units.

In total 10 tests were performed which considered various parameters such as A_s and L/d ratio; these are highlighted in Table 2. A detailed review of the test setup is reported in Black (2007).

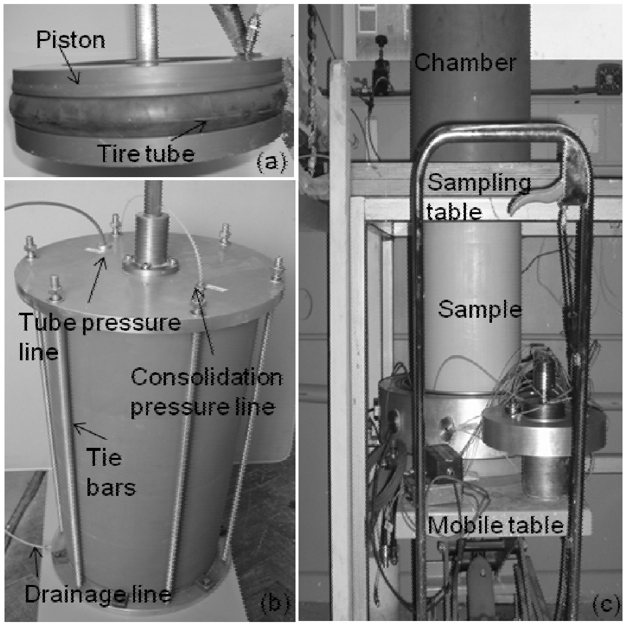


Figure 5 (a) Innovative piston sealing arrangement (b) consolidation chamber and (c) sample extrusion.

Table 1. Triaxial loading stages.

Stage	Description	Stress conditions (kPa)
1	Isotropic compression	Cell 275; Back 200
2	K_0 loading	σ'_v 141; σ'_h 100 (K_0 =0.7)
3	Foundation loading	0.8 kPa/h

Table 2. Test Schedule and Column Parameters.

Test No.	d: mm	L: mm	A_s (%)	L/d
TS01		Un-reinforce Sample		
TS02	25	125	17	5.0
TS03	25	250	17	10.0
TS04	25	400	17	16.0
TS05	32	125	28	3.9
TS06	32	250	28	7.8
TS07	32	400	28	12.5
TS08	38	125	40	3.3
TS09	38	250	40	6.6
TS10	38	400	40	10.5

4 RESULTS AND DISCUSSION

4.1 Sampling and Quality Control

Trimmings from the top and bottom of the consolidated sample in the one-dimensional consolidation chamber were used to access sample consistency and stress-strain homogeneity. The average void ratio at the top and bottom of the samples was 1.38 and 1.55 respectively. This variation in voids profile is attributed to stress reduction caused by frictional resistance between the wall of consolidation chamber and the consolidating clay. This is further substantiated by a direct pressure measurement taken at the base of the sample using a pressure cell mounted on the base plate. Pressure measurements indicated that the vertical consolidation pressure of 150 kPa gradually reduced to approximately 100 kPa as the consolidation progressed; thus confirming frictional loses and the need for an advanced test protocol for settlement evaluation.

4.2 Isotropic Compression and K_0 Loading

The samples were re-consolidated under isotropic confinement and K_0 loading as outlined in Table 1. It should be noted that where the column is end bearing, the K_0 configuration is representative of the unit cell concept (Mattes and Poulos 1969, Priebe 1993 and 1995). Figure 6 shows the settlement of the top plate with change in the vertical pressure for columns 400 mm long with increasing column diameter. Also included is the performance of the TS01 were no column is present. Under the unit cell consideration the column diameters of 25 mm, 32 mm and 38 mm relate to A_s ratios of 0.7%, 1.1% and 1.6% respectively. From Figure 6 it is evident that the inclusion of a column has a significant effect on reducing foundation settlement for small values of A_s under this unit cell configuration.

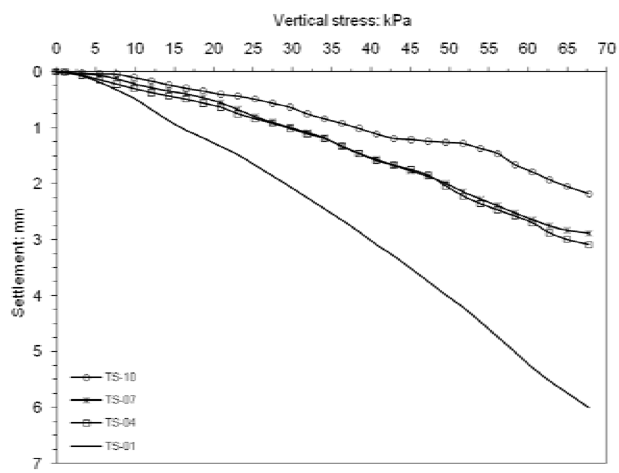


Figure 6. Pressure settlement response during K_0 loading for samples reinforced.

Figure 7 presents the lateral strain measurements during the K_0 loading of samples reinforced with fully penetrating columns together with the response of the sample where no column was present. Note, due to technical difficulties no results were obtained for TS04. It is evident that for the unreinforced sample K_0 conditions were reasonably maintained as displacements are negligible (Figure 7 – TS01). However, in the reinforced samples a true K_0 condition was not achieved. This is largely due to the non-homogeneity resulting from different stiffness properties of the composite samples. When a column was presence, the composite sample laterally contracted suggesting that the column supported a significantly greater amount of the applied vertical load than the soil. This is substantiated by the pressure measurements taken on the top of the column (PT1) and the peripheral soil (PT3) (Figure 8).

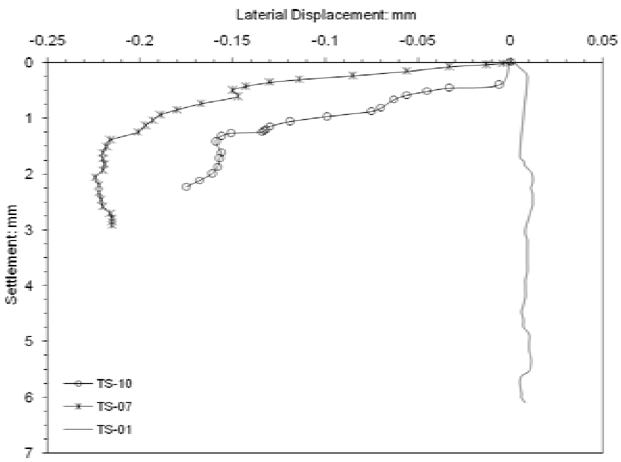


Figure 7. Lateral strain during K_0 loading for samples reinforced with fully penetrating columns.

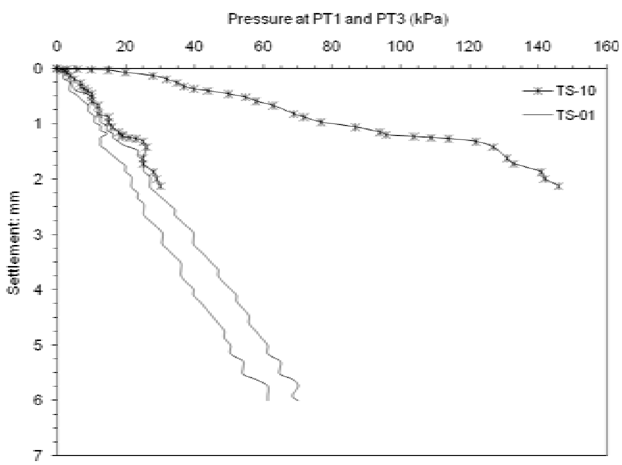


Figure 8. Pressure monitored at PT1 and PT3 for an un-reinforced and reinforced.

4.3 Foundation Loading

Figure 9 shows the pressure-settlement response of the independent 60 mm diameter footing supported on columns of varying length for similar values of A_s. In the current configuration the column diameters 25 mm, 32 mm and 38 mm relate to A_s ratios of 17%, 28% and 40% respectively. When relatively low values of A_s are employed (17% and 28%) (Figure 9 a and b) it is evident that there is no apparent threshold value for column

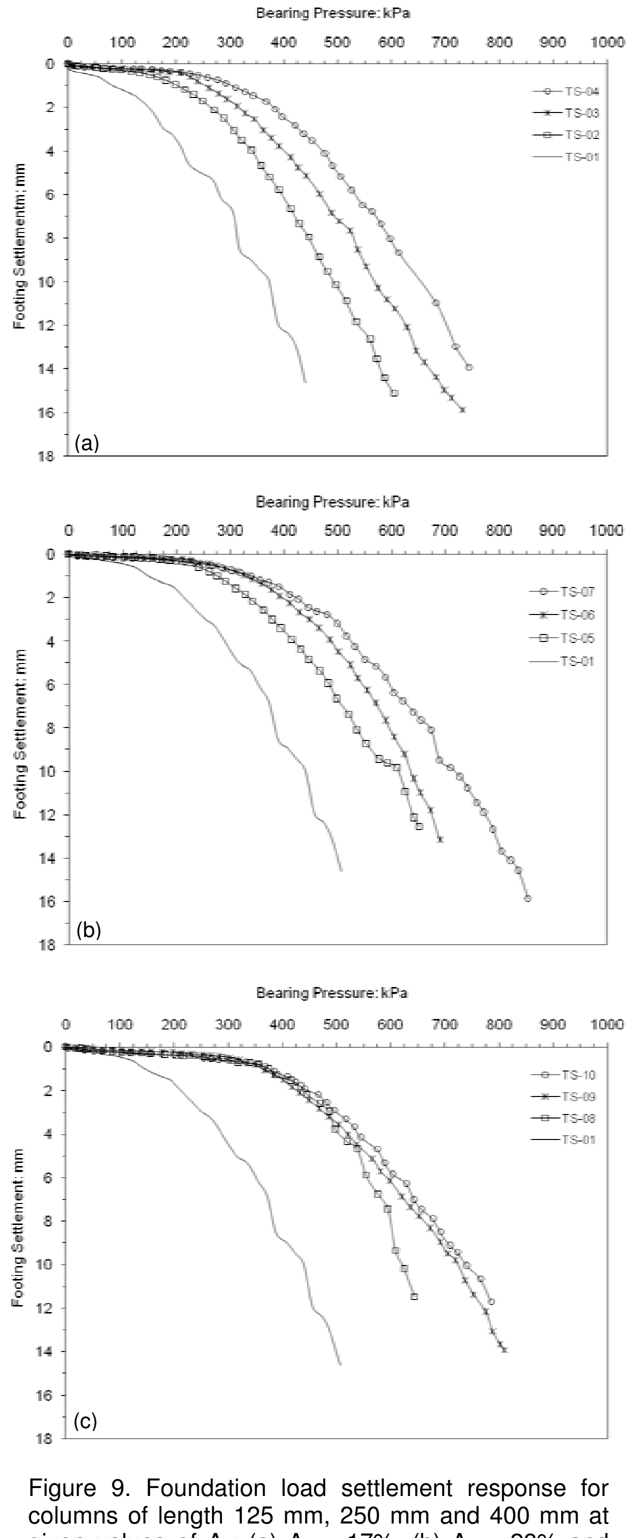


Figure 9. Foundation load settlement response for columns of length 125 mm, 250 mm and 400 mm at given values of A_s; (a) A_s = 17%, (b) A_s = 28% and (c) A_s = 40%.

length in controlling settlement. However, when A_s = 40% the performance of the partially penetrating columns (125 mm TS-08 and 250 mm TS09) are almost identical to that of the 400 mm column (TS10) up to a bearing pressure of 500 kPa. A comparative analysis of a long slender column (i.e. TS04) with that of a short larger diameter column (i.e. TS08) revealed that similar magnitudes of settlement are achieved at moderate loading conditions. This indicates a possible design flexibility relating to the L/d combination for the control of settlement. This finding could have significant implications for designers of VSC foundations as the design may be optimised for individual projects for specific ground conditions.

Further examination of the pressure-settlement performance relating A_s at similar column lengths provides a hypothesis of an optimum value for A_s when the column L/d > 6 for settlement control (Figure 10). It is evident that relatively no performance benefit is observed when A_s increases from 28% to 40%. A possible explanation for this behaviour may be determined using the lateral displacements monitored at the sample boundary.

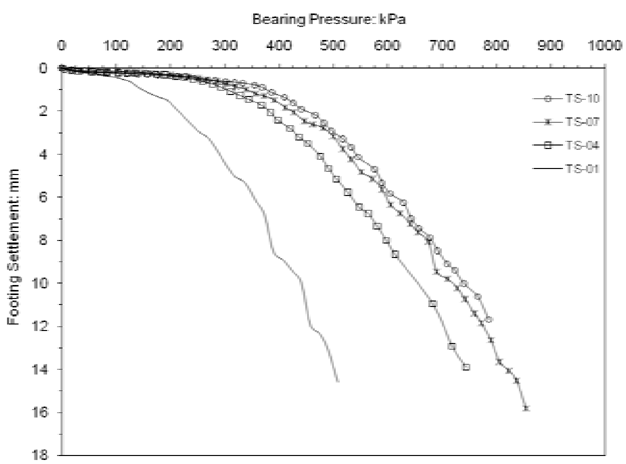


Figure 10. Foundation load settlement response for 400mm long columns with varying A_s (17% - 40%).

Figures 11a and 11b show the lateral displacement of the sample in the case of a 125 mm and 250 mm long columns with increasing values of A_s. Also included is the lateral deformation of the sample when the column was not present. Note that a positive displacement refers to expansion of the sample. The results indicate that samples reinforced with 125 mm and 250 mm columns exhibit conflicting behaviour. In the case of 250 mm long columns the lateral strain at the boundary of the sample indicated a lateral expansion of the sample at all values of A_s. However, in the case of 125 mm long columns the sample laterally contracted; the magnitude of which increased with A_s. This occurrence can be related to the eventual failure mechanics.

Shorter columns tend to fail in pile action where most of the loading is transferred to the base of the column; therefore transferring the stresses to greater depth beyond the lateral strain gauge. Longer columns fail in bulging (Figure 12) and the lateral expansion of the sample reported in Figure 11b is largely due to this effect. Bulge failure is attributed to a lack of radial confinement in the upper region of the column where stresses from the

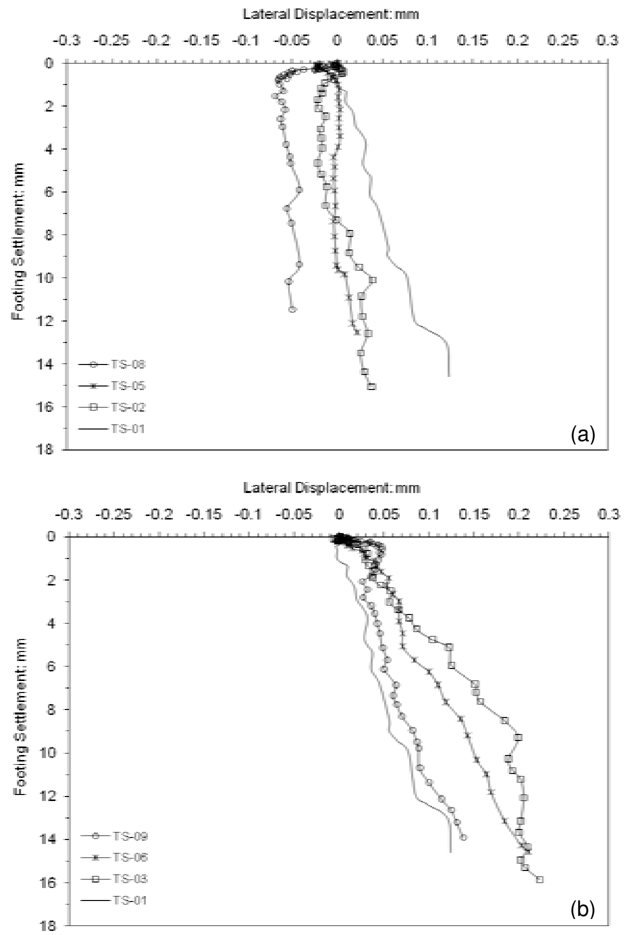


Figure 11. Lateral displacements for (a) 125 mm long and (b) 250 mm columns during foundation loading.

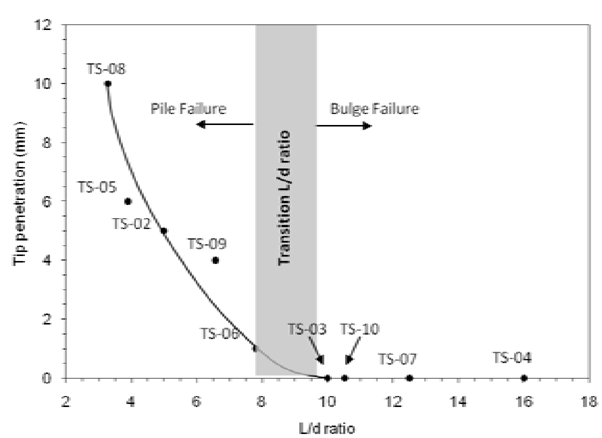


Figure 12. Column failure modes in relation to L/d.

applied loading are highest. When a small diameter column is placed beneath the foundation (i.e. low A_s - 17%) additional confinement is provided to the column by the enhanced stresses in the soil surrounding directly beneath the foundation. This process is referred to as *'foundation shielding'*. Conversely, if the column has a larger diameter (i.e. high A_s - 40%) then the lateral restraint to the column is diminished due to a reduction of the soil annulus beneath the foundation. In such cases a no improvement in settlement performance is observed for a large A_s due to the effects of column dilation due to reduced shielding effects. This can be observed in both the pressure-settlement response in Figure 10 by comparing the performance of TS10 and TS07.

5 CONCLUSIONS

The experimental program utilises a newly developed large triaxial system that is capable of testing samples of 300 mm diameter. This system has a dual axial loading system that allows samples to be consolidated under K_0 condition whilst applying an independent load on a small foundation area. The significant findings of the work include (i) optimisation of A_s for foundation performance (ii) design flexibility relating to the L/d ratio, and (iii) the importance of foundation shielding to provide adequate column confinement.

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REFERENCES

Anderson, W.F., Pyrah, I.C. and Fryer, S. 1991. A clay calibration chamber for testing field devices. *Geotechnical Testing Journal*, Vol. 14, No. 4, pp 440-450.

Black, J.A. 2007. The settlement performance of a footing supported on soft clay reinforced with stone columns. *PhD Thesis, Queen's University of Belfast*.

Bell, F.G. (1993) Engineering Treatment of soft soils. *Spon press*.

Bergado, D.T. Chai, J. C Alfaro, M. C. and Balasubramaniam, A. S. 1994. Improvement Techniques of Soft Ground in Subsiding and Lowland Environnent.

Emmett, K. 2007. Movement of soil and groundwater around piles in layered soil. *PhD Thesis, University of Sheffield*.

Gibson, R.E. and Andersons, W.F. 1961. In-situ measurements of soil properties with the pressuremeter. Civil. Engineering and Public Works Reverence, Vol. 56, No. 65.

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*, Vol. 2, pp 42-49.

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

Kamon, M. and Bergado, D.T. 1991. Ground improvement techniques. *Proc. 9th Asian Regional Conference on Soil Mechanics and Foundation Engineering*, Vol. 2, 521-548.

Kirsch, K. and Sondermann, W. 2003. Ground improvement, In: *Geotechnical Engineering Handbook*, Vol. 2, Ernst and Sohn.

Mattes, N.S. and Poulos, H.G. 1969. Settlement of single compressible pile. Journal of the Soil *Mechanics and Foundations Division, American Society Civil Engineers* (ASCE), Vol. 95, pp 189-207.

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

Moseley M.P. and Kirsch, K. 2004. Ground Improvement. *Spon Press*.

Moseley, M.P. and Priebe, H.J. 1993. Vibro techniques in ground improvement. *Blackie Academic and Professional*.

Navaneethan, T. 2003. Pre-yield characteristics and earth pressure coefficient of over consolidated clays. *PhD Thesis, Queen's University, Belfast.*

Priebe, H.J. 1993. Design criteria for ground improved by stone columns. *Proc. 4th National Conf. on Ground Improvement*, Pakistan.

Priebe, H.J. 1995. The design of vibro replacement. *Ground engineering*, Vol. 28, No. 10, pp 31-37.

Raju, V.R. 2004. Ground improvement using vibro replacement in Asia 1994 to 2004 – A 10 year review. *5th International Conference on Ground Improvement Techniques*, Kuala Lumpur, Malaysia. Keynote session.

Raison, C.A. 2004. Ground and soil improvement. *Thomas Telford, Institute of Civil Engineers*.

Rodgers, A.A. 1979. Vibrocompaction of cohesionless soils. *Cementation Research Limited*. Internal report Rowe, P.W. and Barden, L. 1966. A new consolidation cell. *Géotechnique*, Vol. 16, No. 2 162-170.

Slocombe, B.C. and Moseley, M.P. 1991. The testing and instrumentation of stone columns, *ASTM STP* 1089.

Wood, D.M., Hu, W. and Nash, D.F.T. 2000. Group effects in stone column foundations: model tests. *Géotechnique,* Vol. 50, No. 6. pp 689-698.