Granular Columns for Geotechnical Applications

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ABSTRACT: Soft clay deposits are globally widespread and often coincide with strategic transport links and growing urban developments. These soft deposits are often waterlogged and are composed of clay with varying degrees of silt, sand and organic matter. These soils have low undrained shear strength and high compressibility, contributing to construction problems in relation to stability and settlement. Granular columns, also referred to as flexible piles, are one of the techniques widely considered in the industry for improving soft deposits for low-moderate structural loading. The purpose of this article is to highlight some of the key investigations carried out in the topic of granular columns at Queen's University Belfast, the UK.

The investigations focused on several aspects: (a) the interaction between columns and surrounding clay (b) containment of columns in geo-grid for enhanced strength performance (c) settlement performance under single or multiple column configuration (d) stress distribution under the footing and along the column (e) assessment of consolidation and creep settlement under constant loading and (f) granular columns for anchoring purposes and therefore stabilization of slopes. Overall observations are: settlement improvement factors were moderate under isolated loading, but granular columns are very effective in providing pull-out capacity in the form of anchors.

KEYWORDS: Organic soils, Creep, Settlement, Consolidation, Anchors

1. INTRODUCTION

Geotechnical engineering is a science, but its practice is an art (Madhav, 2004). Certainly the granular column application in geotechnical engineering is an art, but understanding the complex column-soil interactions leading to enhanced performance requires scientific knowledge. On this note, laboratory based model studies and full-scale investigations have been carried out to understand the load carrying capacity and settlement performance of foundations supported on soft clay with granular columns for a number of decades. An executive summary of some selected investigations is reported by Serridge, 2016. A classical knowledge contribution to this specific topic is based on laboratory investigations, carried out by Hughes and Withers, 1974, which was then further developed by many leading researchers in the field. Some of the recent work in this topic include: Raju, 2009; Madhav *et.al*. 2010; McCabe *et.al*. 2009; McKelvey *et al.* 2004; Sivakumar *et al.* 2002; Black and Sivakumar,2011; Sivakumar *et.al*. 2012; Shahu and Reddy, 2011; Cimentada *et al*. 2014; Jeludin *et al*. 2015.

Unlike rigid piles (Figure 1(a)), which generally mobilize only a fraction of the original shear strength of the surrounding clay, granular columns can actually enhance the strength of the clay due to accelerated consolidation. When load is applied, granular columns develop end bearing and side friction stresses in much the same way as piles. The columns also expand laterally but the expansion is restricted by the surrounding soil (Figure 1(b)). Increases in lateral stress lead to consolidation of the surrounding clay and therefore lead to further bulging of the column. This process continues until equilibrium is reached. The net result is higher strengths in both the clay and the column and improved bearing capacity and stiffness.

The majority of previous investigations have been undertaken with respect to bearing capacity as opposed to settlement control. Consequently, the extent by which granular columns reduce the settlement of foundations supported by soft clay treated with them remains unclear (McCabe *et al.*, 2009). Although the granular columns are generally used to treat soft deposits for widespread loading, their use has been recently extended to isolated loading such as supporting pad or strip footings for low rise buildings. The load carrying mechanism and the settlement performance of the foundations under widespread and isolated loading are quite different. This is largely due to the fact that the interaction between the column and the surrounding clay is different under pad/strip and raft footing.

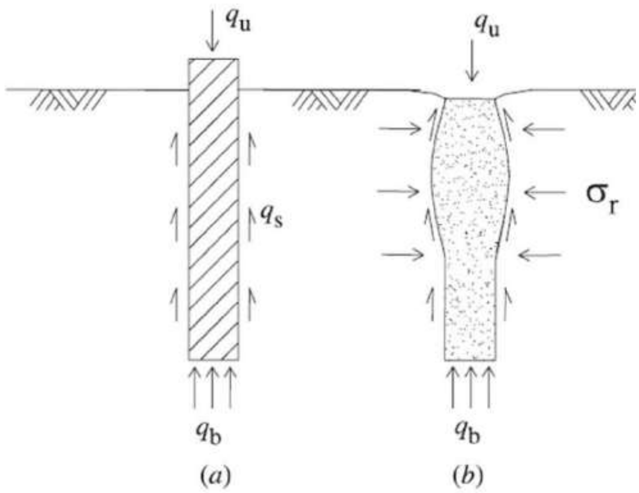


Figure 1 Solid and granular pile

The effectiveness and performance of granular columns is influenced by several factors such as column length to diameter ratio (L/d), area replacement ratio (A_s), column spacing (s), stiffness of the column (E_c) and of the surrounding soil (E_s), stress ratio (σ_{vc} / σ_{vs}), number of columns beneath the footing and the method of installation (Figure. 2). However information is limited, particularly in relation to settlement of pad/strip footings. This article reports some of the key research carried out at Queen's University Belfast in recent years in relation to the above aspects. Granular columns have also other applications, for example stabilization of slopes where anchor forces are needed to provide stability to geo-structures. This article also reports some recent research in this topic.

2. LOAD CARRYING CAPACITY

Granular columns are suitable for supporting lightweight structures such as low-rise housing developments and industrial warehouses. The technique is not usually recommended where the undrained shear strength of the insitu soil is less than 15 kPa because of the low lateral support provided to the columns. However, if there could be some other means of providing lateral support to the columns, then they could be applied to a wider range of soft deposits, including peat. This lateral support can be provided by reinforcing the columns with geo-grids. This method has been used on a trial basis for a high-speed railway embankment in the Netherlands (Nods, 2002). This aspect was carefully examined in laboratory based research. Sand granular columns were installed in the 200 mm high specimens of kaolin. Single columns were installed at lengths of 80, 120, 160, and 200 mm. This corresponds to values of H_c/H_s = 0.4, 0.6, 0.8, and 1.0, where H_c is the length of the column, and H_s is the length of the specimen. Woven cloth was to reinforce the column. Tests were also carried out on specimens without sand columns (H_c/H_s = 0). After the column had been installed, the composite clay–granular column specimen was consolidated to 100 kPa of effective confining pressure, with a "back" pore-water pressure of 300 kPa. The top cap on the sample consisted of a metal ring with an insert of diameter of 40mm representing a model footing (Figure 3). Upon consolidation of the sample, further loading was applied on this model footing under undrained conditions.

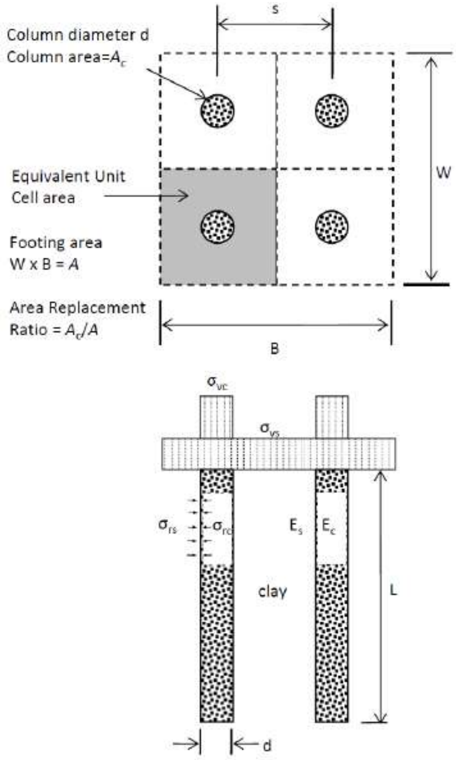


Figure 2 Key factors affecting granular column performance

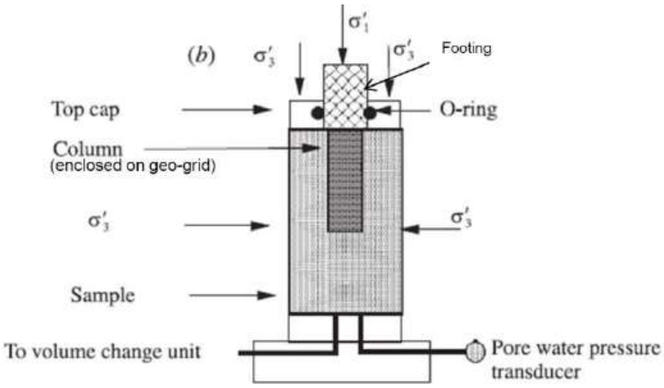


Figure 3 Testing arrangement where model footing is supported on granular column contained in geo-grid

Figure 4(a) shows the load–deformation characteristics for composite specimens installed with reinforced columns and subjected to foundation loading on a reduced area at the top of the specimen. Load–deformation curves for unreinforced columns are again included for comparison. Figure 4b shows the ultimate load carrying capacity of the foundation supported on clay or clay with granular columns included. The load carried by the foundation supported on the reinforced fully penetrating column (H_c/H_s = 1) was approximately 800 N (and was still increasing when the test ended). This represents an increase of approximately 60% over an equivalent unreinforced column and an increase of 185% over the same clay without a granular column. The large increase in capacity is due to the lateral support provided by the geogrid reinforcement. Shorter columns again showed no increase in capacity. It appears that lateral confinement provided by the geogrid leads to a substantial increase in the load-bearing capacity when the column is end bearing on a stronger stratum. In practice, columns are usually taken to stronger strata. When this is done, including geogrids around the columns may enhance the overall performance. Further details can be found in Sivakumar *et al*. 2004 and Black *et al.* 2008.

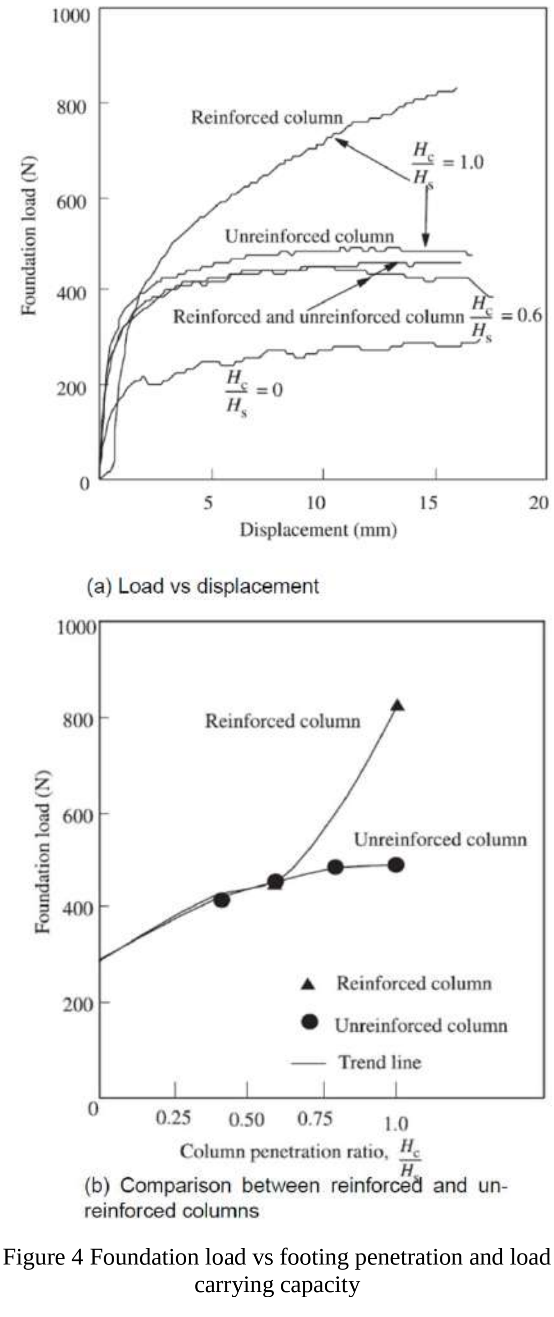
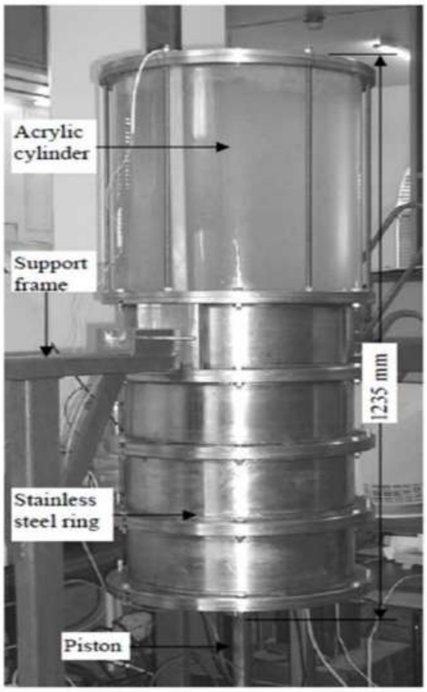


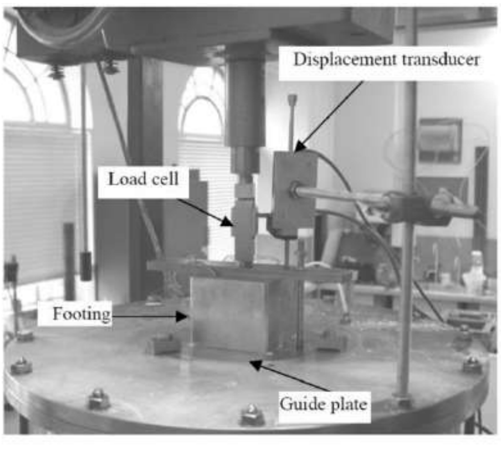
Figure 4 Foundation load vs footing penetration and load carrying capacity

3. CLAY-COLUMN INTERATION UNDER PAD/STRIP FOOTING

The interaction between the column and the surrounding clay under isolated loading (for a pad/strip) is complex. This aspect was examined using a laboratory model study in which the clay bed was formed using transparent material. A large loading chamber was manufactured for the purpose of producing one-dimensionally consolidated soft clay layer. The material was prepared by mixing fumed silica in an oil blend of mineral spirits and crystal light liquid paraffin. The slurry was 7% fumed silica by weight. The oil blend was 70% liquid paraffin and 30% mineral spirits. Figure 5(a) shows a photograph of the loading chamber. 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. A small triaxial compression machine mounted in an inverted position on top of the frame supporting the consolidation chamber allowed the application of loads to the model foundation at a constant rate of settlement (Figure 5(b)).



(a) Consolidation chamber



(b) Model footing supported on transparent clay

Figure 5 Consolidation chamber and testing arrangement for model footing supported on transparent clay

After mixing, the slurry of transparent clay was initially consolidated under a vertical pressure of approximately 70 kPa by elevating the air pressure in the lower part of the consolidation chamber. Upon consolidation, the pressure was removed, the lid on the middle of the top plate was removed and the columns were installed at the selected configuration. Once the columns were installed the lid was relocated, but this time the lid consisted of a strip or circular footing (Figure 5(b)). Three sand columns, 25 mm in diameter, were installed in a triangular arrangement beneath the circular footing (100 mm in diameter) and in a row beneath the strip footing (100 x 50 mm) to depths of 150 mm and 250 mm. This corresponds to L/d ratios of 6 and 10, where L is the column length and d is the column diameter. Displacement-controlled loading was applied to the model footing at a rate of 0.0064 mm/min until the footing penetrated approximately 35 mm into the clay. This rate was considered sufficiently slow to ensure a fully drained conditions. A digital camera was mounted outside the acrylic section of the loading chamber and photographs were taken of the deforming columns every six hours (i.e. the footing penetrations into the soil bed of 2.3 mm every 6 hours)

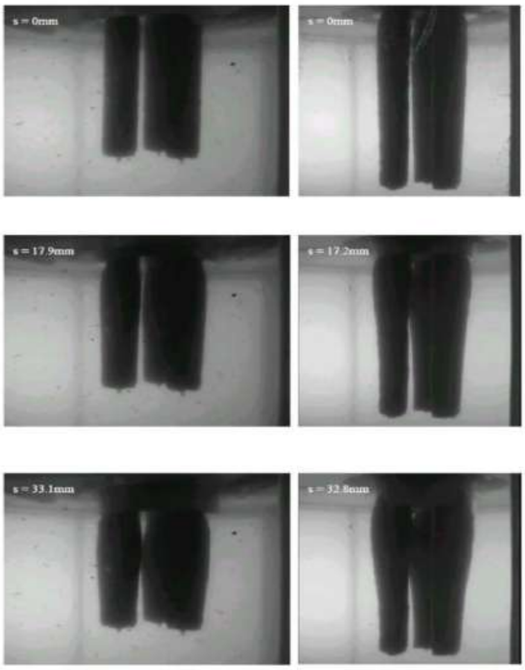
Figures 6(a) and 6(b) show the images of the deforming columns at the beginning, middle and end of the loading process for the circular and strip footing respectively. It should be noted that the scale of the photographs in each of the tests is different. In the circular footing tests (Figure 6(a)), the camera was not situated immediately in front of the columns. Instead it was positioned at a slight angle in order to photograph the entire deforming shape of at least one of the columns. In the strip footing tests however (Figure 6(b)), 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 took place over the entire length of the columns. The longer columns (L/d = 10) deformed significantly in the upper region while the bottom region did not appear to have undergone significant deformation. It may be assumed therefore that there was little or no load transferred to the lower parts of the longer columns. The observations and the subsequent image analysis showed strong evidence to suggest that the short columns penetrated (or punched) into the underlying soft clay. The amount of penetration was approximately 5 mm based on scaling the image. It is also evident that the bulging was not symmetrical about the axis of the columns, particularly in the case of the circular footing. The main reason for this is the interaction within the group and the restraint provided by neighboring columns. Further details can be found in McKelvey *et al.* 2004.

4. SETTLEMENT CONTROL

The extent by which granular columns reduce the settlement of foundations supported by soft clay treated with them remains unclear (McCabe *et al.*, 2009). This is particularly the case under isolated loading. Therefore a series of investigations was carried out to examine this aspect using a model study in laboratory conditions. The particular significance of the equipment developed for this purpose was the ability to test large samples (400 mm in height and 300 mm in diameter) under K_o conditions while applying the foundation load independently (Figure 7). The relevant arrangement of the top cap is shown in Figure 7(b), where the 60 mm diameter footing is housed and fastened to the top plate.

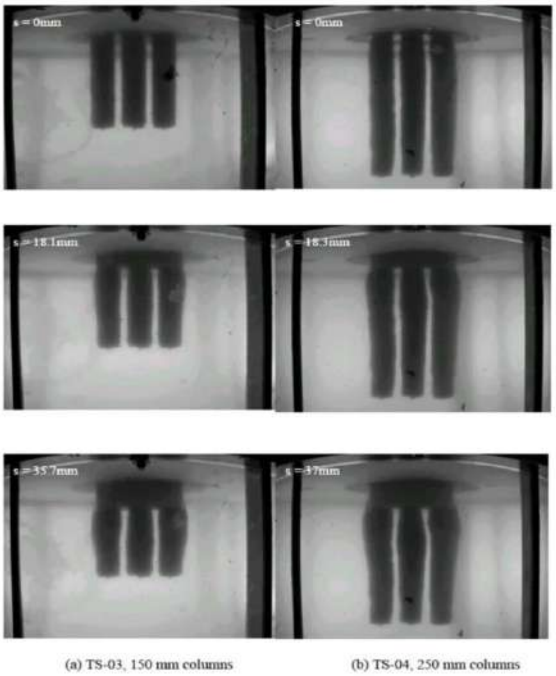
Samples were prepared by consolidating 35 kg of kaolin powder mixed at a water content of 105% in a one dimensional consolidation chamber to a vertical pressure of 200 kPa. Figure 8 shows the consolidation chamber and the extrusion process after consolidation. The sample was extruded and directly located on the base of the testing chamber shown in Figure 7. Granular columns were installed into pre-formed holes using compaction of uniformly graded basalt (90% of the material having 1-1.5 mm particle sizes).

The samples were consolidated initially to 75 kPa of isotropic consolidation pressure and then followed by Ko loading whereby the vertical and the horizontal pressures were increased slowly to 125 kPa and 100 kPa respectively, representing a Ko value of 0.8 (Ko is the coefficient of earth pressure and a slightly higher value implies that the kaolin was overconsolidated). This was then followed by a foundation loading typically lasting 2-3 weeks in which the loading on the footing was gradually increased at a rate of about 2 kPa/h (rate estimated using the coefficient of consolidation to achieve a fully drained conditions during foundation loading).



(a) TS-01, 150 mm columns (b) TS-02, 250 mm columns

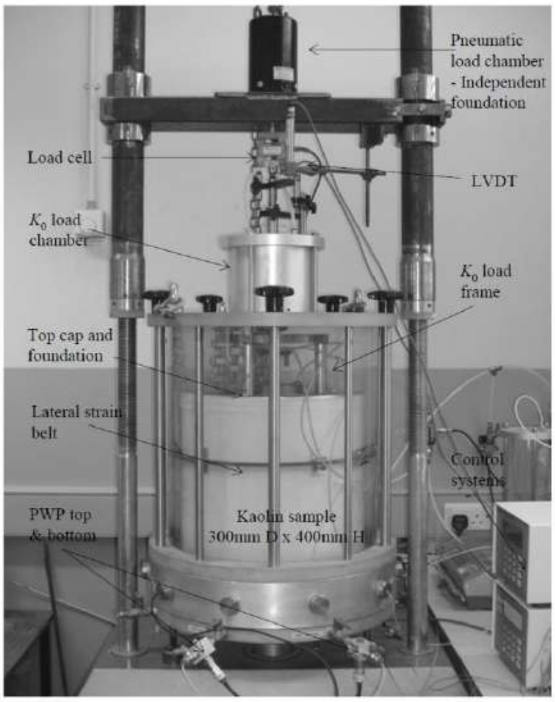
(a) Circular footing



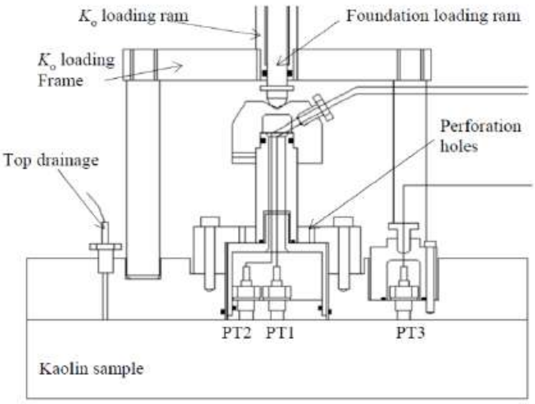
(a) TS-03, 150 mm columns (b) TS-04, 250 mm columns

(b) Strip footing

Figure 6 Column deformations under foundation loading



(a) Testing chamber



(b) Footing arrangement

Figure 7 Testing chamber for assessing settlement of footing supported on circular footing

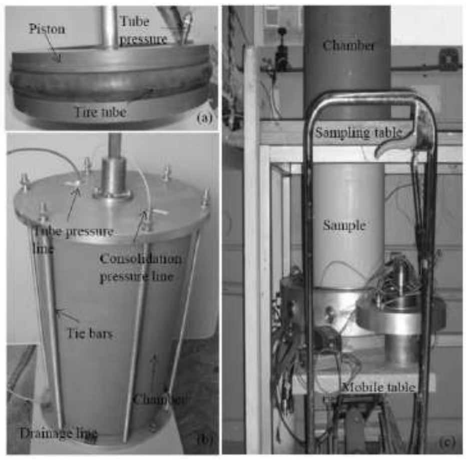


Figure 8 Sample preparation in one-dimensional consolidation chamber

The column diameters adopted in the research represent area replacement ratios of 17 %, 28 % and 40 % beneath the footing diameter of 60 mm. Figure 9(a) shows the bearing pressure - settlement relationship for A_s = 17 % at H_c/H_s ratios of 0.31, 0.62 and 1.0, for tests TS-02, TS-03 and TS-04 respectively. Similar figures for A_s = 28 % and A_s = 40 % are shown in Figure 9(b) and 9(c) respectively. While noting the fact that increasing the column length resulted in an enhanced load carrying capacity, the settlement at a bearing pressure of 160 kPa (design bearing pressure) was the main focus of this investigation. The settlement improvement factor is plotted with respect to the L/d ratio for all values of A_s in Figure. 10(a). It is evident that n increases with respect to the L/d ratio for each area replacement ratio although it appears that increasing the column geometry beyond L/d = 8 - 10 offers little significant improvement particularly at lower A_s values of 17 % and 28 %. This agrees favorably with findings previously published by McKelvey *et al.* 2004 who postulated a critical L/d ratio of 6 in relation to bearing capacity performance for physical model tests.

The settlement improvement factors n (ratio of the settlement of the footing supported on unreinforced clay and clay bed included with granular column) are also plotted with respect to the area replacement ratio in Figure 10(b). It is evident that the settlement improvement factor increases with area replacement ratio in a significant manner; however, there appears to be a threshold A_s level for improvement of between 30 % - 40 % particularly when the column is non-end bearing. When compared with predicted values of settlement reduction factor n from Priebe (1995), it is evident that the observed experimental results are much higher than expected. A possible explanation for this could be due to the confinement provided as a consequence of the rigid nature of the surcharge boundary condition provided by the top plate, i.e. away from the footing. For moderate area replacement ratios the clay annulus beneath the foundation surrounding the column is also subjected to increased vertical stress for the foundation load, hence is able to provide enhanced lateral restraint against bulging. This beneficial effect is dependent on the thickness of the annulus and when this is significantly reduced (as in the case of larger A_s values) the overall column performance is compromised as bulging failure occurs more readily. Further details can be found in Black *et al.* 2012.

5. EVALUATION OF SETTLEMENT UNDER FLEXIBLE BOUNDARY CONDITIONS

The work reported by Black *et al.* (2012) examined the settlement of footings supported on soft clay with granular columns and concluded that the settlement reduction factors are higher than reported in literature or using numerical predictions. The enhanced settlement reduction factors observed in the research were attributed to testing conditions in which the boundary condition away from the footing was maintained rigid. In real life this boundary condition is flexible. Jeludin *et al.* 2016 examined this aspect using a model study carried out on samples of kaolin, 400 mm in height and 300 mm in diameter (Figure 11). The settlements of the footing and that away from it were measured using internal displacement gauges. The boundary condition away from the footing (70 mm diameter) was maintained flexible using the following approach.

A flexible boundary condition was achieved for the horizontal clay bed surface away from the footing using a specially manufactured rubber membrane (Figure 12). The holder for the 70 mm diameter footing was located in its middle opening and sealed using 'O' rings. The outer perimeter of this membrane was sealed against a 300 mm diameter aluminum ring, with wall thickness of 12mm, again using 'O' rings (Figure 12(b),12(c)). The location and verticality of the footing holder was maintained by attaching the holder to the outer ring using an aluminium cover plate. This plate was perforated so that the cavity inside the unit could be filled with water, purging any air from inside, during the apparatus assembly (Figure 12(c)). The footing was also fitted with pressure cells to measure the contact pressure between the footing and the column or clay.

Two different column configurations were investigated: a single 40 mm diameter column with A_s = 33% and a group of 5 columns, each 18 mm in diameter but with a similar area replacement ratio. Columns were installed to depths of 200, 300 or 400 mm in the 400 mm long clay beds, which represent L/D (where L and D are the column length and diameter of the column) ratios for single column setups of 5.0, 7.5 and 10.0, respectively, and for group of five column groups of 11.1, 16.6 and 22.2 respectively. The granular columns were constructed in ~2 cm deep layers using uniformly-graded fine gravel.

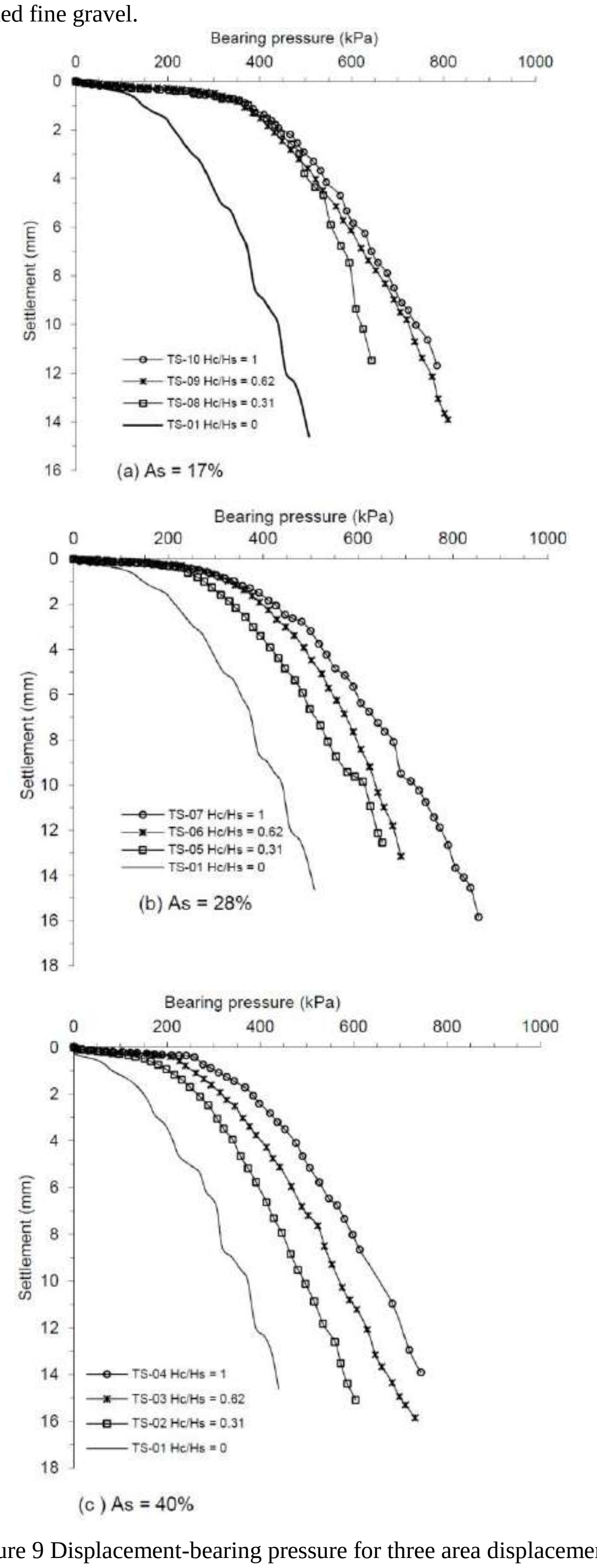
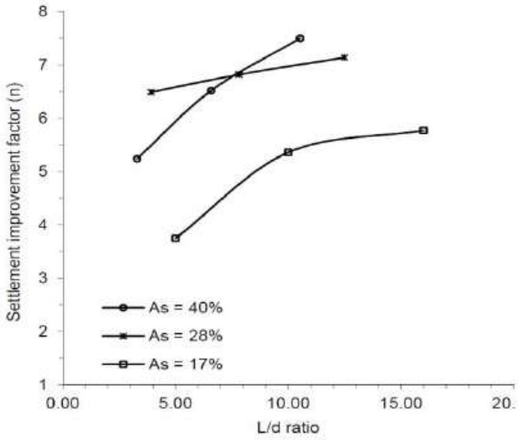
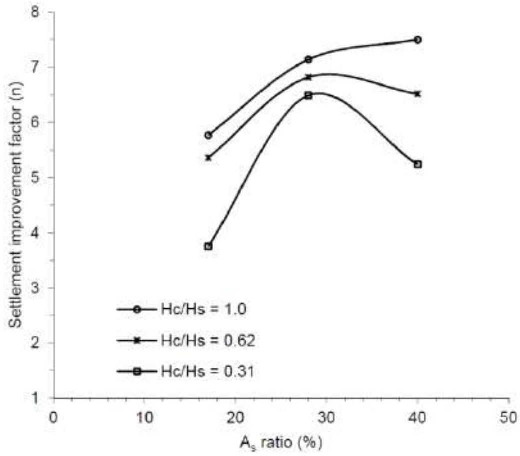


Figure 9 Displacement-bearing pressure for three area displacement ratio



(a) Settlement improvement factor n vs As



(b) Settlement improvement factor n vs L/d ratio

Figure 10 Settlement improvement ratio in relation to area replacement ratio A_s and L/d ratio

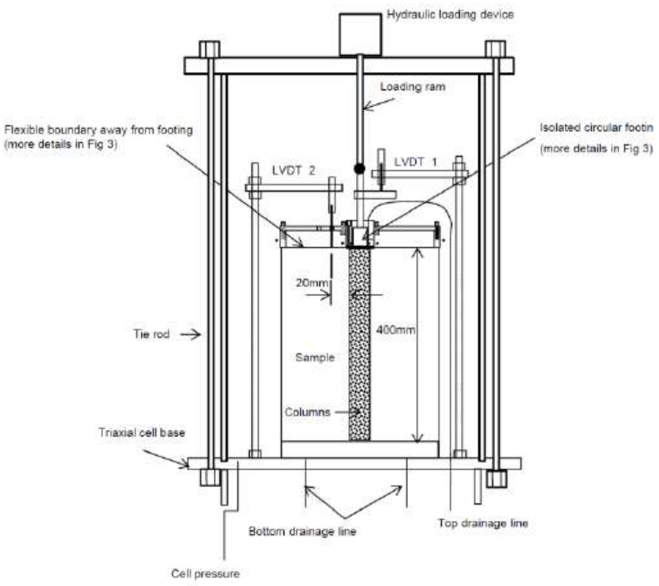
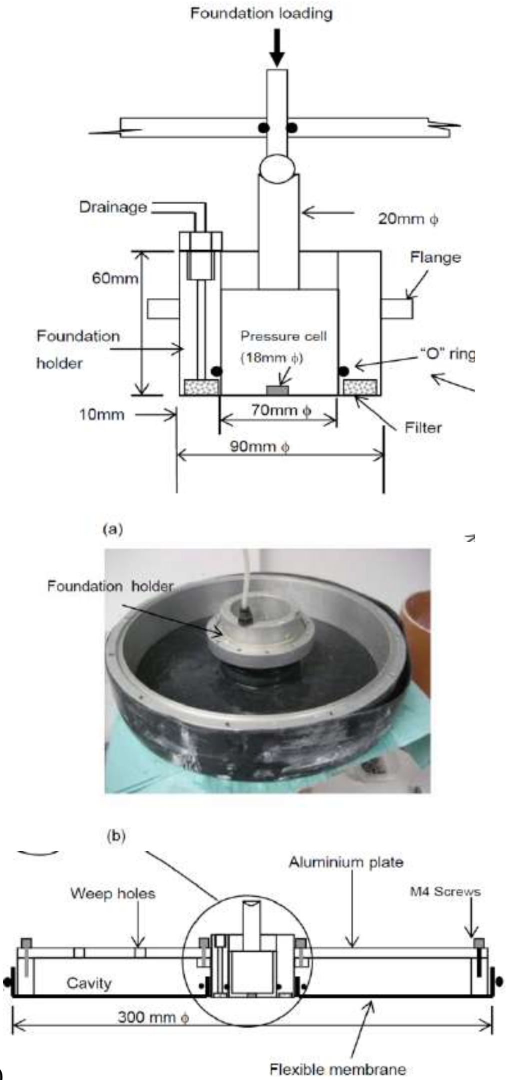


Figure 11 Testing chamber for assessing settlement of footing supported on circular footing under realistic boundary conditions

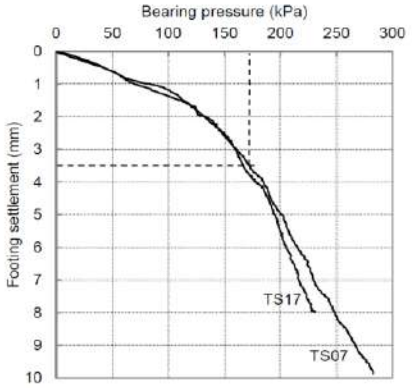
For brevity the observations referring to fully penetrating single columns is presented in Figure 13 (also included is the observation referring to a clay bed without granular columns). For the footing supported on a clay bed incorporating a single end-bearing column, the bearing pressure of the footing mobilized at 10 mm of footing displacement increased to 315 kPa (Figure 13(b)).



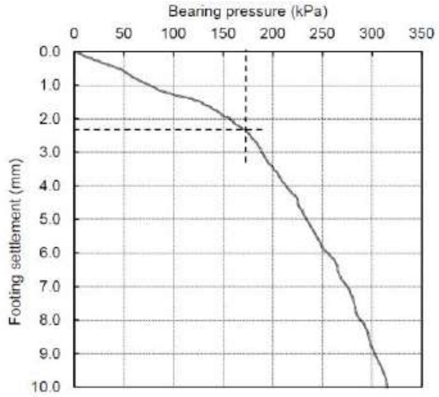
(c)

Figure 12 Footing and testing conditions away from footing

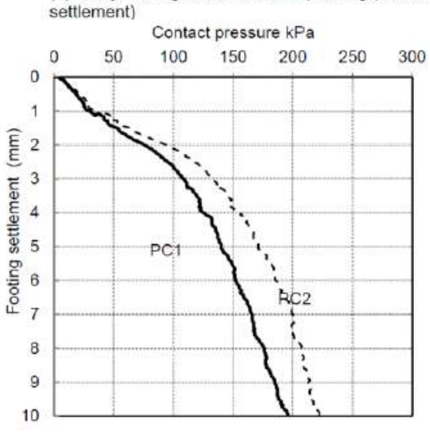
At a design pressure of 175 kPa, the footing settled by about 2.3mm when compared with untreated clay bed which settled by 3.5mm. This corresponds to a settlement reduction factor n of 1.5. Although not shown here, the shorter column with a L/D ratio of 5 yielded no significant settlement reduction factor but in contrast the column with a L/D ratio of 7.5 yielded a settlement reduction factor of 2.0 suggesting that the end-bearing column (L/D ratio of 10) performed less well than the floating column with a L/D ratio of 7.5. The reason for this lies in the contact pressures between the column and the clay. The observations as per contact pressures are shown in Figures 13d. The change in contact pressure between the footing and the column had steadily increased to 685 kPa by the termination of testing (Figure 13d). In contrast, for the clay beneath the footing, an increase in contact pressure of ~130 kPa occurred for a footing displacement of 2 mm, with further loading producing a slight reduction in contact pressure to ~100kPa. However, not shown here, the contact pressure between the clay and the footing was significantly higher in the case of a column with a L/D ratio of 7.5, suggesting that the clay may have gained strength during the foundation loading and therefore provided significant confinement to the column against bulging.



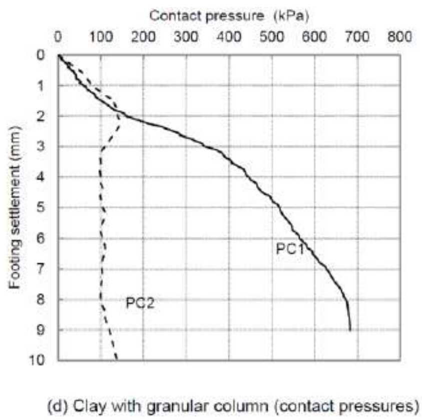
(a) Untreated clay (bearing pressure vs settlement)



(b) Clay with granular column (bearing pressure vs settlement)



(c) Untreated clay (contact pressures)



(d) Clay with granular column (contact pressures)

Figure 13 Bearing pressure vs settlement (Fully penetrating columns)

In practice, granular columns are usually installed in groups; e.g. 3 to 5 granular columns are installed under strip or pad foundations while arrays of granular columns are installed under raft foundations. The group effect for granular columns has not been well researched, but the study reported by Black *et al.* 2011 suggested that the effectiveness of granular columns in containing settlement is reduced for column group formations, especially for floating columns. This aspect was examined further in the present investigation using the improved testing chamber (i.e. with the flexible clay bed surface boundary condition).

Figure 14 shows the bearing pressure–settlement relationships for a footing supported on a clay-bed reinforced by a group of 5 granular columns with lengths of 200, 300 and 400 mm. Also included are the relationships for a footing supported on an unreinforced clay bed and a clay bed reinforced by a single 40 mm diameter end-bearing column with the same area replacement ratio as the column groups. The long floating and end-bearing column groups (TS13 and TS14) had similar initial stiffness responses, marginally greater than those for the unreinforced clay bed, single end-bearing column and the short floating column group (TS07, TS10 and TS15). Overall, the post yield stiffness responses of the short, long and end-bearing column groups were similar, slightly lower than that of the unreinforced clay bed, and below that of the single end-bearing column. Based on the criterion adopted for evaluating settlement (i.e. design bearing pressure for the footing of 175 kPa), compared with the footing displacement of 3.5 mm for the unreinforced clay bed, some improvement was achieved for the end-bearing single and multiple columns which performed similarly (footing penetrations of 2.4 and 2.3 mm respectively). No improvement was achieved for the floating column groups. Further details can be found in Jeludine *et.al.* 2016.

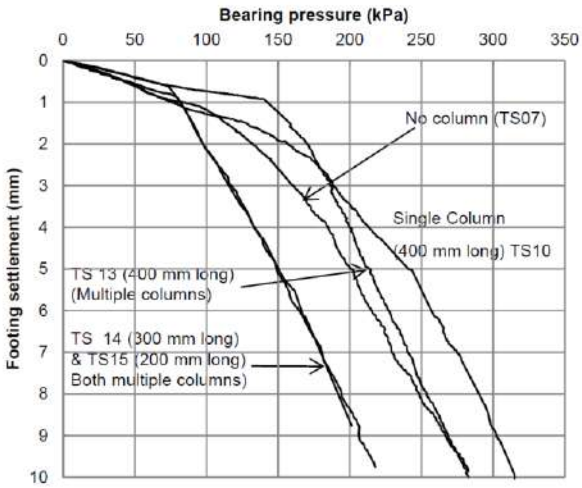


Figure 14 Bearing pressure–settlement relationships for an unreinforced clay bed and clay beds reinforced by a single end bearing column and column groups (floating and end bearing)

6. PRESSURE DISTRIBUTION ALONG THE GRANULAR COLUMN

A large triaxial cell capable of testing 300mm diameter by 400mm high samples was used in this investigation. Figure 15 illustrates the key aspects needed in the equipment, which include: (a) application of independent foundation loading; (b) flexible boundary conditions away from the footing; (c) measurements of pressure along the column using pressure cells contained in capsules and located at four locations along the columns. The electrical signals from the pressure cells were fed through the base of the sample using a novel experimental set-up.

The granular columns, on 400mm high 300 mm diameter clay samples were formed by compacting crushed basalt with particle sizes between 2.5-3mm into a pre-bored hole. Once the column length of 90mm was achieved the pressure cell (Figure 15(b)) was carefully located on the top of the column. Using this procedure, the column was built to the full height. The settlement of the footing and the surrounding clay were measured using LVDTs, as shown in Figure 15. In the first stage of testing the sample was allowed to consolidate under 50kPa of effective pressure, resulting from 300kPa of confining pressure and 250kPa of back pore water pressure. In the second stage, the foundation was loaded in a ram fashion at a rate of 1 kPa per h. Further information as to the instrumentation and testing procedure can be found in Sivakumar *et al.* (2012).

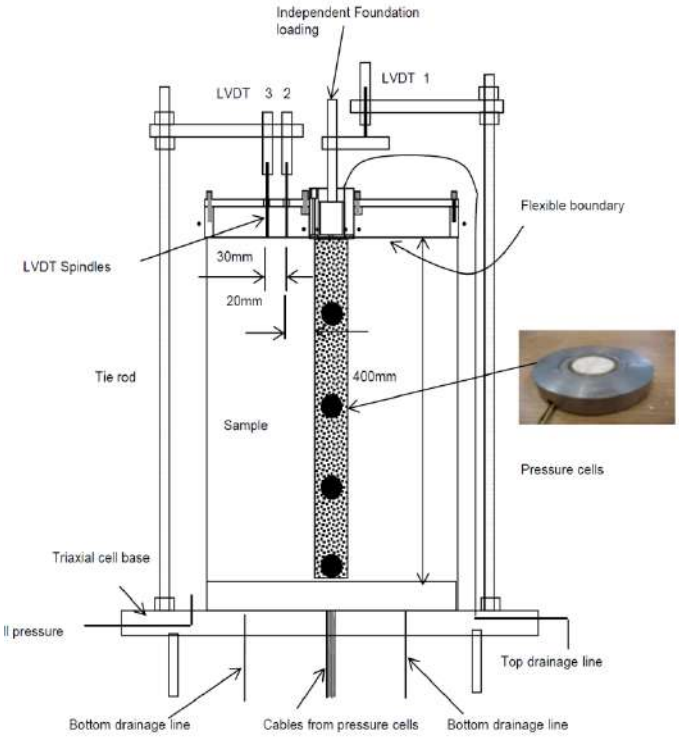


Figure 15 Assembled sample

Figure 16(a) shows how the vertical pressure changed with time as the excess pore water pressure dissipated over the consolidation period. Pressure cells PC1 to 4 were located along the column where PC0 was just under the footing and PC4 was at the base of the column. The pressure under the footing increased by around 18kPa but at the bottom of the column the pressure was 90kPa. Figure 16b shows the pressure distribution along the column at specified times intervals. Observations show moderate to large increases in vertical pressure at the bottom third of the sample, but a slightly lesser increase at the top third of the sample and generally a small increase in the middle section of the sample. Figure 15c shows the compression of the granular column and the surrounding clay (LVDT 1 and LVDT 2 respectively) during the consolidation process. The granular column underwent settlement during the entire phase of the consolidation (i.e. exhibiting time-dependent behaviour). This could be attributed to a gradual increase in vertical stress in the column caused by the consolidation of the surrounding clay. The evidence for this is shown in Figure 15c where the surrounding clay settled more than the column. The consolidation of the clay is a time-dependent process and, therefore, its effects on the stone column are also time dependent. The surrounding clay consolidates more than the stone column, resulting in the development of negative skin friction on the stone column.

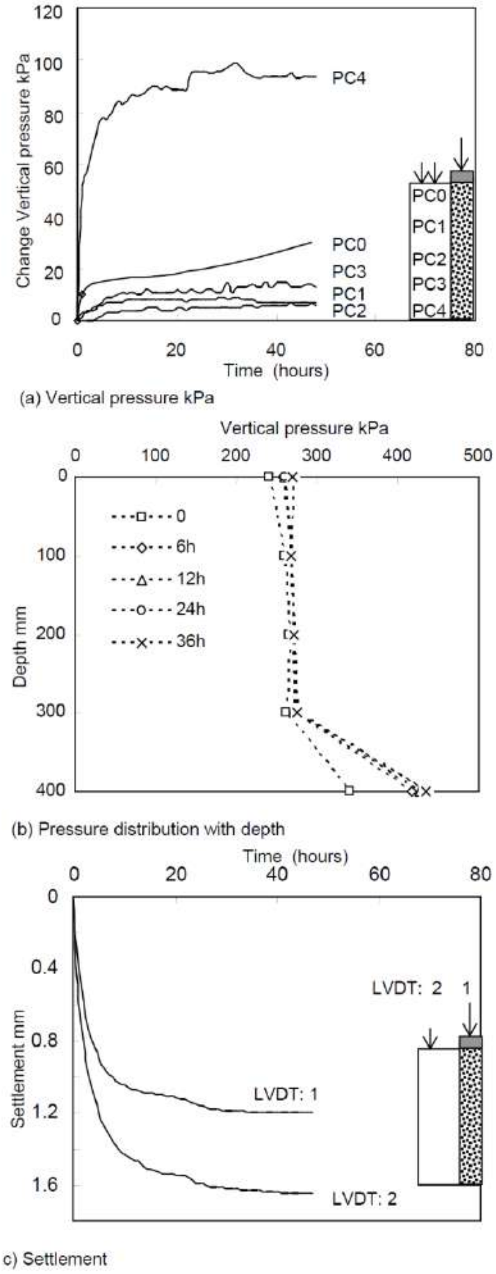
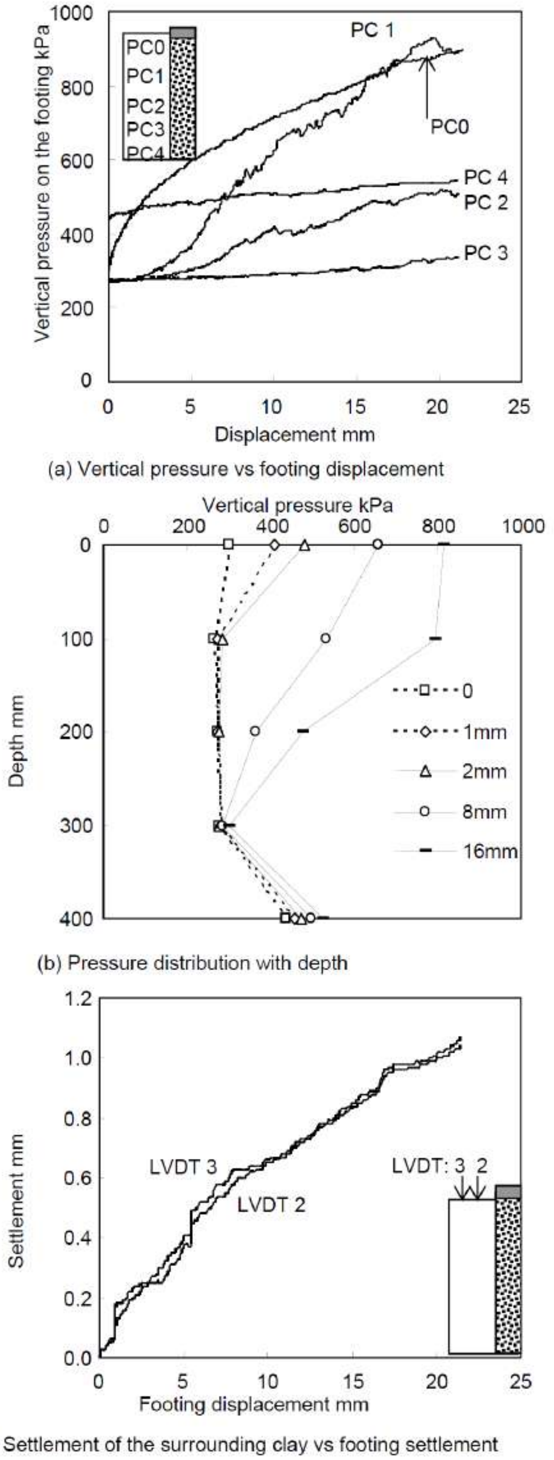


Figure 16 Performance of the sample with 50mm f column during consolidation

The loading on the foundation was increased at a rate of 1kPa/h. Referring to Figure 17(a), the pressure under the footing was about 318kPa at the beginning of the foundation loading but increased to 800kPa at a footing displacement of 16mm. Figure 17b, where the relevant pressures are plotted against depth, shows how it varied at specified footing displacements. The changes in the vertical pressures reduced along the column length up to a depth of about 300mm before it began to increase along the bottom quarter of the column. Unlike rigid piles, any load that is applied to the stone column has to be supported by the surrounding clay, particularly at shallow depths. The bulging generates enhanced lateral pressures and shear stresses in the upper region, which in turn leads to settlement of the surrounding clay. Negative skin friction only develops when the settlement of the pile is less than that of the surrounding clay. It is possible that the overall compression of the granular column below the critical length is not significant compared with the compression of the surrounding clay below this depth, thereby leading to the development of negative skin friction. Further details can be found in Sivakumar *et.al.* 2010.



(a) Vertical pressure vs footing displacement

(b) Pressure distribution with depth

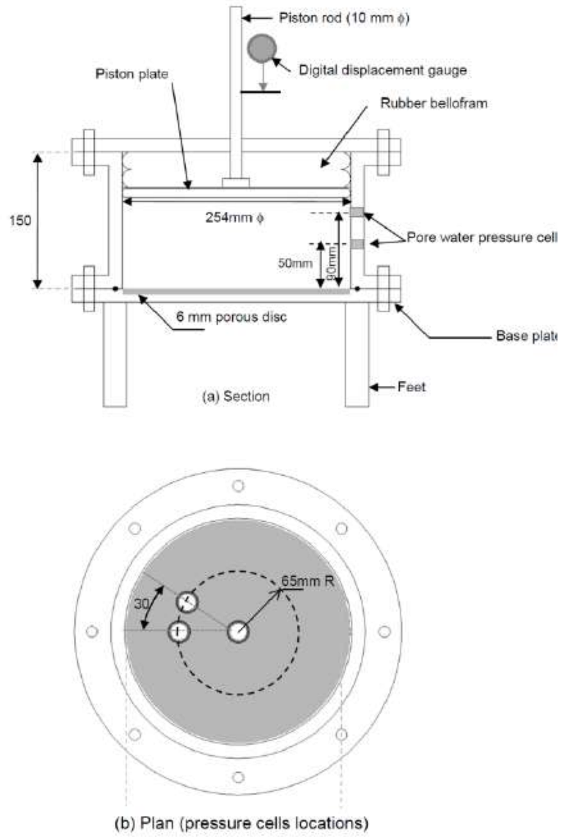
(c) Settlement of the surrounding clay vs footing settlement

Figure 17 Performance of the sample with 50mm □ column during foundation loading

7. CONSOLIDATION, CREEP SETTLEMENT UNDER RAFT LOADING

This study aimed to quantify the effectiveness of granular columns to reduce initial, primary consolidation and creep settlements through the analysis and interpretation of data collected from a laboratory based study. A one dimensional loading chamber was developed (Figure 18) which included: (a) pressure transducers to monitor the pore water pressures along the length of the sample; (b) three pressure cells located on the base plate to measure the load carried by the column and surrounding clay; (c) a displacement gauge to monitor the vertical settlement; (d) a pneumatic pressure controller to monitor and maintain a constant back pore water pressure in the sample; (e) a volume change unit to monitor the volume of water drained from the sample and (f) a pneumatic regulator to maintain a constant overburden pressure during each loading increment. The diameter of the consolidation chamber was 254 mm and the height was 150 mm; drainage was permitted only from the bottom.

Alluvial soil, known locally as 'sleech', was used to prepare samples. The material was remoulded at its natural water content of 38% and hand kneaded into the one dimensional consolidation chamber. Once filled the clay was levelled to the required height of 120 mm, a rigid piston plate combined with a rolling diaphragm was positioned and secured using standard procedures. In the case of the unreinforced clay bed, the samples were subjected to four loading stages corresponding to effective overburden pressures of 25 kPa, 50 kPa, 100 kPa and 200 kPa with a constant back pore water pressure of 200 kPa. In the case of the reinforced clay bed, the sample was consolidated to 25 kPa of effective vertical pressure and it was unloaded under undrained conditions. At this stage a 100 mm diameter granular column was formed by the replacement method. The pressures were re-applied and subsequently taken to 50 kPa, 100 kPa and 200 kPa of vertical pressures.



(a) Section

(b) Plan (pressure cells locations)

Figure 18 Schematic of one dimensional testing chamber

Figure 19 shows the settlement of the sleech clay bed plotted against log time (without and with the granular column) for the three loading increments. The required time for 100% consolidation, t_{100}, is 300 hours for the first loading increment of 25-50 kPa with slightly reduced durations for the other loading stages in the case of the clay bed without granular columns. This reduced to 70-90 hours for the clay bed with 100 mm diameter granular columns. The average coefficient of consolidation values c_v for untreated clay bed varied between 0.4-0.5m^2/year during three stages of loading and it increased to values between 2.7-4.5 m^2/year for clay bed with granular columns included. This is a significant increase in the coefficient of consolidation value and represents a reduction in consolidation time between 6-9 fold when granular columns are included for settlement control. The relevant creep rates c_α (defined as $c_\alpha = \Delta\varepsilon_\alpha/\Delta\log(t)$) are 0.96, 1.17 and 0.93 for loading stages 25-50 kPa, 50-100 kPa and 100-200 kPa respectively for untreated clay bed. The creep rates for the clay bed with granular columns included are 0.54, 0.78 and 0.6 respectively for the above loading ranges. Accordingly the creep reduction factor n_α are 1.8, 1.5 and 1.5. The primary consolidation settlements of the untreated clay bed under the three stages of loading are 5.2 mm, 4.9mm and 4.5mm. These settlements reduced to 1.4mm, 2.9mm and 3.7mm for the clay bed with granular columns under the three stages of loading. This reduction in settlement represents a settlement reduction factor n_c of 3.7, 1.7 and 1.2 respectively for the loading rates mentioned above. The improvement factors as per the settlement are significant under low loading ranges (25-50 kPa and 50-100 kPa) but not significant under higher loading ranges. The creep reduction factor n_α seems to be moderate at all three loading stages. Further details can be found in Sivakumar *et al.* (2017)

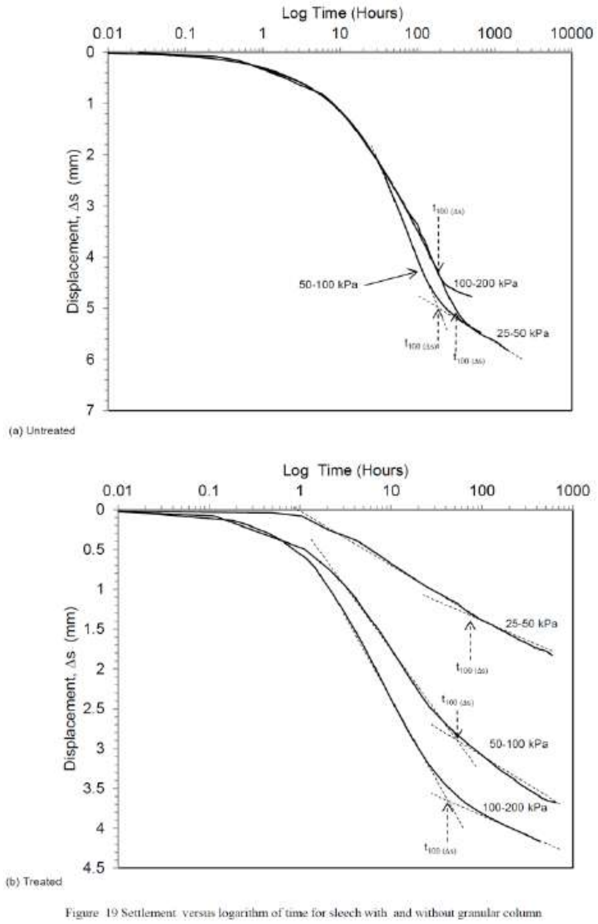


Figure 19 Settlement versus logarithm of time for sleech with and without granular column

8. GRANULAR COLUMNS IN SAND LAYER UNDER CYCLIC PORE WATER PRESSURE

These experiments were performed in a test chamber with dimensions of 1.4m × 0.72m in plan by 0.70m deep. The experimental sand-bed deposit was prepared to represent a loose sand layer. This bed was subdivided into six bays that included different column configurations (see Figure 20 single or multiple columns). A 5-cm deep gravel layer comprising 6 mm aggregate was first placed at the bottom of the testing chamber and overlain by a highly porous synthetic sheet. Slightly moist commercially-available washed sand with a natural moisture content of approximately 4% was spread into the chamber in stages, forming successive layers, each approximately 0.10 m in uncompacted layer thickness. These were lightly tapped with wooden block of 3.4 kg mass resulting in bulk densities 1.51 Mg/m^3.

The diameter for the single column configurations was 6 cm. A smaller column diameter of 2.8 cm was used in the case of the configuration with four columns located at the corners of the 9-cm square footing. The area replacement ratios for these arrangements were 35% and 30%, respectively, typical of values for isolated footings. The columns were installed using a displacement method. The granular columns extended over the full 0.6-m depth of the sand beds, end bearing at the top of the underlying gravel layer. An exception was one of the single columns (Bay 1, see Figure 20), which was terminated at a depth of 0.36 m in the sand bed. The performance of the experimental model footings constructed in the loose sand bed was studied under cyclic variation of the water table over a period of 28 days, with one filling/empting cycle occurring every 18 h (fluctuation of the water-table by 0.6m water table). Each of the rigid footings, 0.09×0.09 m in plan dimensions, was attached to an aluminum rod (see Figure 21) along which a 0.015 m diameter loading platform was secured in order to support dead-weights that would apply a maintained load on the horizontal footing. Displacement gauges mounted on guide bars (Figure 21) monitored the footing settlement response.

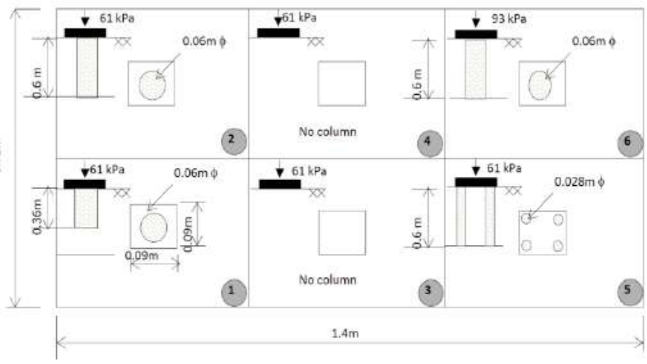


Figure 20 Footing Layouts

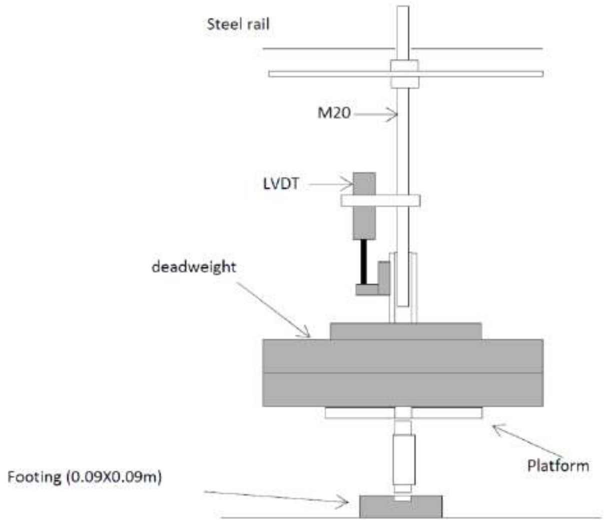


Figure 21 Foundation loading

Figure 22 shows that the sand surface remote from the footings settled by ~0.3 mm, although it appears not to have fully reached an equilibrium steady-state condition by the end of the pumping test. Again, greater footing settlement occurred for unreinforced sand (Bays 3 and 4). However only low-to-moderate footing settlement occurred for sand reinforced by single or multiple columns (Bays 1, 2 and 5), with significantly lower settlement occurring for reinforced sand subjected to higher bearing pressure (Bay 6). The settlement of the footings was also ongoing for all setups. At the termination of testing the settlement ratio (defined as δs/H, where δs and H are recorded settlement and bed thickness respectively) in increasing order for Bays 1–6 were 15.5×10^{-4}, 24.6×10^{-4}, 196.4×10^{-4}, 196.4×10^{-4}, 44.6×10^{-4} and 7.5×10^{-4}. This has highlighted a potential settlement problem, caused by pore water pressure variation in sand deposits, that can be contained by the introduction of granular columns. However the latter can attract additional problems, including migration of fines into the large voids of the granular column, which can also lead to settlement related issues. Hence the settlement, while greatly reduced compared with unreinforced sand, still occurred at a significant rate. This study also highlights that using longer and/or multiple columns may attract higher settlement than shorter and/or single columns. Further details can be found in Sivakumar *et.al.* 2013.

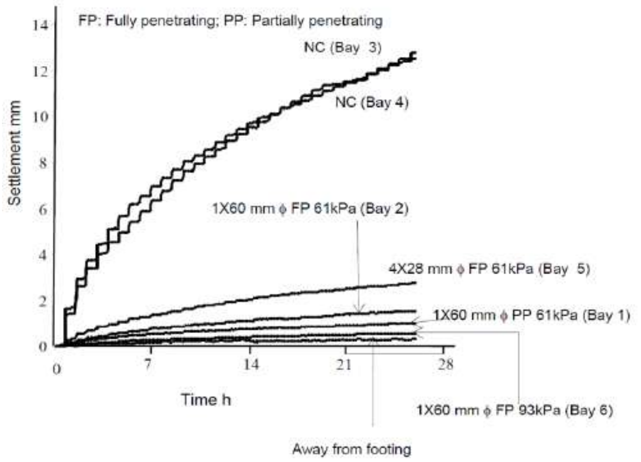


Figure 22 Settlement against time for loose sand bed

9. GRANULAR ANCHORS

Granular columns are traditionally used for improving weak deposits by means of increasing the bearing capacity reduced settlement. There has been some discussion in recent years as to whether granular columns could also be used to resist tension/pull-out forces (Phani Kumar and Ramachandra Rao 2000, Liu *et al*. 2006, Madhav *et al*. 2008). Such granular anchors consist of a horizontal base plate, a centrally-located tendon (stretched cable or metallic rod) and compacted granular backfill. The tendon is used to transmit the applied load to the column base via the circular base plate, which compresses the granular material to form the anchor. The load can be applied to the anchor immediately after its construction and drainage is also provided, via the granular column, to the soil surrounding the anchor. Granular anchors can have much wider applications in the construction industry, not only to enhance the stability of retaining structures, rock faces or sheet piles but also to act as an effective drainage system in order to prevent excessive build-up of pore water pressure, particularly in slope stabilization.

The focus of the investigation was to compare the ultimate pullout capacity of granular anchors in direct pullout against that of conventional cast *in-situ* concrete anchors. The ultimate pullout capacity is the load at which the anchor is pulled out of the ground, either by failure in shaft resistance mobilised between the granular/concrete column and surrounding soil or alternatively, in the case of granular columns, by localised end-bulging of the column itself (Hughes and Withers, 1974). The tests were performed at Queen's University Belfast (QUB), with the experimental programme considering the assessment of two variables: namely anchor lengths (L) of 0.5, 1.0 and 1.5 m, and anchor diameters (D) of 0.07 and 0.15 m. Incremental loading of the anchors in direct tension was achieved using a custom-built loading device (Figure 23). The load–displacement response of the ground anchor system was measured using load cells and long-stroke displacement transducers. The vertical displacement of the ground surface was also measured at a distance of 0.3 m radially from the anchor tendon by a displacement transducer mounted on an independent reference beam.

The granular anchors were installed in made ground that had been placed about 50 years previously, and was classified as firm to stiff clayey silty sand with occasional gravel. Mean values of c_u of 55 kPa were measured for depths greater than 0.5 m below the ground surface, with slightly higher c_u determined for shallow depths. In addition, tests were also performed on concrete anchors.

Uniformly-graded basalt gravel (nominally 10-mm in size and with an angle of shearing resistance ϕ'_g of 45° for the density achieved in the anchor setups) was used as backfill for granular anchors and also as aggregate in forming concrete anchors. In constructing the anchors, the steel base plate with the tendon (threaded steel rod) was inserted to the base of the borehole. In the case of the granular anchors, the borehole was backfilled by pouring the gravel into the bore cavity to form ~0.12 m thick layers, which were individually compacted to achieve maximum density using a special hammer, comprising an annular compaction-plate and hollow tube. The mass of the hammer was ~2.5 kg and the gravel layers were compacted, in turn, by dropping the hammer 27 times through a free-fall distance of 0.7 m, which produced a bulk unit weight for the gravel of 22 kN/m^3. The concrete anchors were allowed to cure for 7 days before performing the tension/pullout load tests.

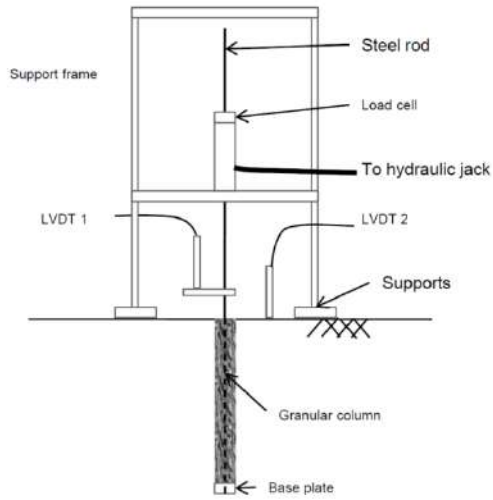


Figure 23 Schematic of loading frame

The experimental results are reported as tension load against vertical anchor displacement in Figure 24. The pullout capacities of the granular and concrete anchors of L x D = 0.5 x 0.07 m were 5.5 and 5.2 kN respectively (Figure 24a). The granular anchor displaced significantly (> 40 mm upward movement of the top surface of the gravel column) during the course of loading compared with the concrete anchor, although the displacement of the latter at the time of failure was considerable (i.e. sudden pullout occurred), implying both of these anchors failed on resistance mobilised along the column shaft.

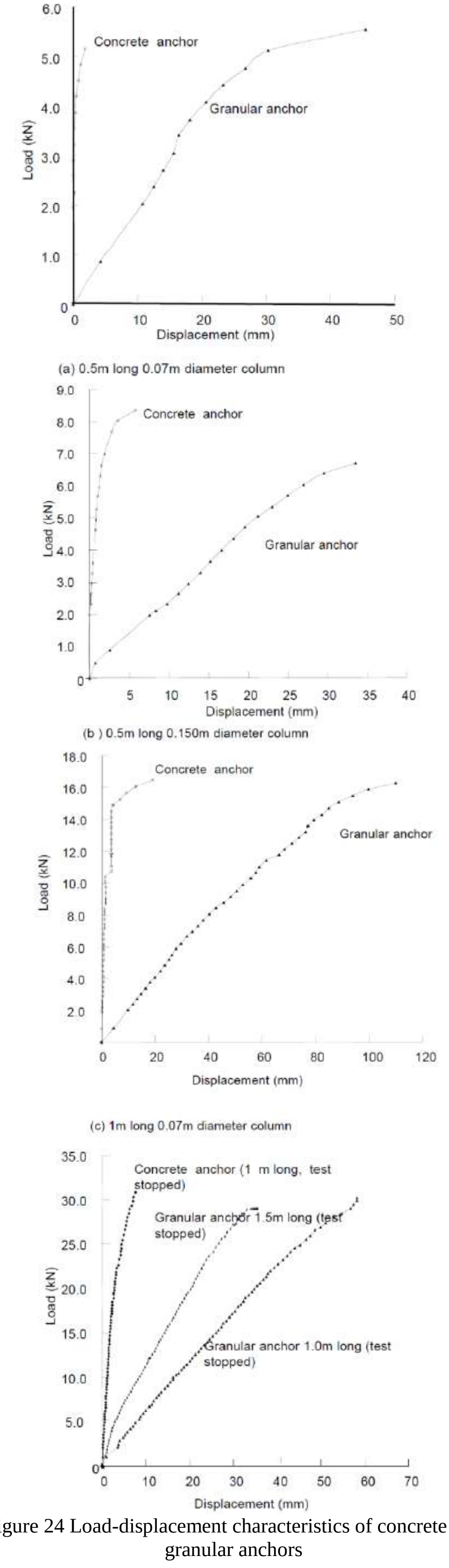


Figure 24 Load-displacement characteristics of concrete and granular anchors

The soil surrounding the concrete anchor did not undergo any significant displacement (either heave or subsidence) until the failure state was achieved. However the soil surrounding the granular anchor progressively heaved as the anchor was incrementally loaded to failure. Anchors of L x D = 0.5 x 0.15 m also failed on shaft resistance (Figure 24(b)), experiencing ductile and sudden pullout behaviour for granular and concrete constructions, respectively, with mobilised pullout capacities of 6.7 and 8.0 kN respectively. The granular anchor of L x D = 1.0 x 0.07 m experiencing ductile failure, underwent localised end-bulging (Figure 24(c)), whereas the concrete anchor experienced sudden pullout, failing in shaft resistance. Pullout capacities of 16.1 and 16.3 kN were mobilized for these granular and concrete columns respectively. During the early loading stage, the surrounding ground barely moved, although ground heave started to occur as the anchors approached pullout capacity. The 1.0 and 1.5 m long anchors of 0.15 m diameter (Figure 24(d)) could not be taken to true failure since this exceeded the capacity of the loading system used in performing these series of tests. Nevertheless, it would appear from the load–displacement responses in Figure 24d that failure of both concrete and granular anchors was imminent at the time when the loading had to be terminated prematurely, particularly in the case of the 1.0 m long anchors.

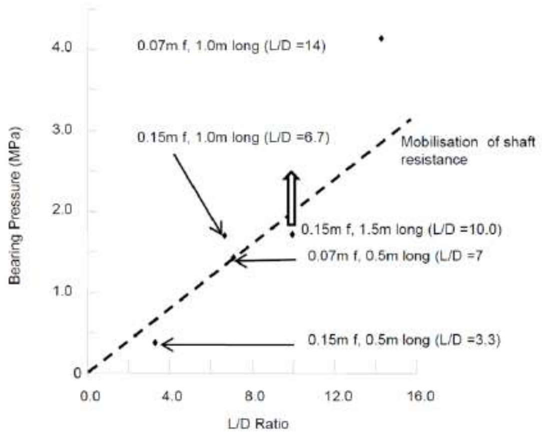


Figure 25 Bearing pressure vs L/D ratio QUB site

In granular column applications for ground improvement, the column can fail by one of two distinct mechanisms. As the load increases on the granular column, shaft resistance develops along the cylindrical surface and end bearing resistance at the base of the granular column are mobilized gradually. Shaft failure is typical for short columns and for values of L/D ratio < ~6–7. In contrast, longer columns fail in localized bulging in the vicinity of the column head since the shaft resistance and end bearing capacities exceed the bulging capacity. This analogy was extended to granular anchors with the proviso that bulging in granular anchors occurs close to the bottom of the column. Bulging capacity at the column base σ_v was estimated using the relationship proposed by Hughes and Withers (1974):

$$\sigma_v = \left[\frac{1+\sin\phi'_g}{1-\sin\phi'_g}\right]\left[\sigma_{vc} + N_c^* c_u\right] \tag{1}$$

where σ_{vc} is the overburden pressure caused by the surrounding soil at the point of bulging; ϕ'_g is the angle of shearing resistance of the granular column and N_c^* is a bearing capacity factor that considers local shear failure and is assumed to be approximately = 4.6. The undrained shear strength against depth profile of the surrounding soil is the crucial piece of information required for the prediction of anchor performance/mode of failure, with shaft resistance mobilized along the full length of the column shaft whereas bulging occurs locally in the vicinity of the column base. The experimental programme at the QUB site considered the assessment of anchor length (0.5, 1.0 and 1.5 m) and diameter (0.07 and 0.15 m) on ultimate pullout capacity. The strength against depth profile of the ground determined using a hand vane indicated an average undrained strength of 55 kPa for depths greater than 0.5-m below the ground surface. The anchor dimensions are shown in Table 1 and the measured and predicted pullout load is shown in Table 2.

Table 1 Anchor dimensions

Test no.	Bore diameter (m)	Diameter base plate (m)	Column length (m)	L/D ratio
1	0.07	0.07	0.5	7.0
2	0.07	0.07	1.0	14.0
3	0.15	0.15	0.5	3.3
4	0.15	0.15	1.0	6.7
5	0.15	0.15	1.5	10.0

Table 2 Measured and predicted pullout load

Test No	Shaft capacity kN	Bulging capacity kN	Measured pullout capacity kN
1	6.1	5.9	5.2
2	12.2	6.1	16.5
3	13.2	27.0	7.5
4	26.3	28.1	30.7
5	39.4	29.1	30.8**

** Test terminated due to load cell maximum load capacity

Failure over the column length would occur due to a shear zone developing within the remoulded soil next to the bore sidewall and not along the granular/soil interface since no distinct granular surface forms, with the confined granular material intruding slightly into the adjacent soil under pullout loading. Hence $\alpha = 1$ (adhesion factor) is assumed in determining the shaft resistance. This is also supported by back-calculating the value of α from the observed performance of the concrete anchors. Tables 1 and 2 list values of predicted shaft resistance and bulging capacities together with measured pullout loads at the termination of each test, including the anchor dimensions. Note that loading was terminated at 30 kN load for one of the anchors on account of the load cell capacity being reached. Based on available information, it can be concluded that the 0.07 and 0.15-m diameter by 0.5-m long anchors failed in shaft resistance whereas the 0.07 and 0.15-m diameter by 1.0-m long anchors may have failed by localized end-bulging. This postulation is further illustrated by plotting bearing pressure acting on the column base against L/D ratio (see Figure 25). Included in this figure and indicated by a broken line, is the mobilization of shaft resistance for an average undrained shear strength of 55 kPa over the column length. Based on the results obtained, it can be concluded that the 0.07 and 0.15-m diameter by 0.5-m long anchors failed on shaft resistance whereas the other anchors may have failed on bulging. Furthermore the work clearly suggests that the L/D ratio which distinguishes whether pullout failure occurs in shaft resistance or localized end-bulging is about 7. Further information can be found in Sivakumar *et al.* 2014 and O'Kelly et.al. (2015).

10. STABILIZATION OF SLOPES

Figure 26 illustrates a cross-section of the model box where the slope was constructed using a moist gravelly sand. The key dimensions of the test arrangement are also shown in Figure 26. The material was mixed with 5% kaolin to constitute the soil test bed. The gravelly sand had the following particle size distribution, D_{10} = 0.1mm, D_{30} = 0.25mm and D_{50} = 0.7mm (D_{60}/D_{10} = 7 & D_{30}^2/ $D_{60}\times D_{10}$ = 0.9). The resulting composite material was mixed at 5% water content using a large rotating drum mixer. Post mixing, the material was stored in large sealed plastic bins. A total of 10 tests were conducted. Each time the soil bed material was re-used, to ensure uniformity of composition within the soil mass. When using the granular anchors, the anchor material was carefully extracted from the failed mass before re-using to limit contamination in further tests. During the tests, the soil bed was covered with cling film to avoid moisture loss. For each test, the box was filled in a series of 5 layers; each layer was compacted uniformly using a vibrating plate. The final average bulk density of the compacted soil was 1850kg/m^3. Upon filling of the box, a 45^o slope was formed which was cut back to leave a 60^o slope if needed, depending on the testing requirements.

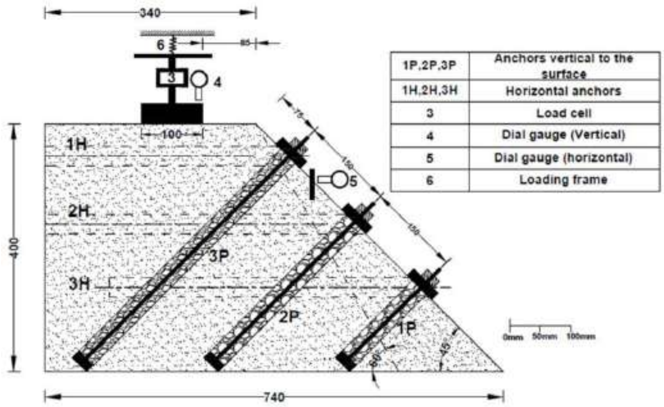
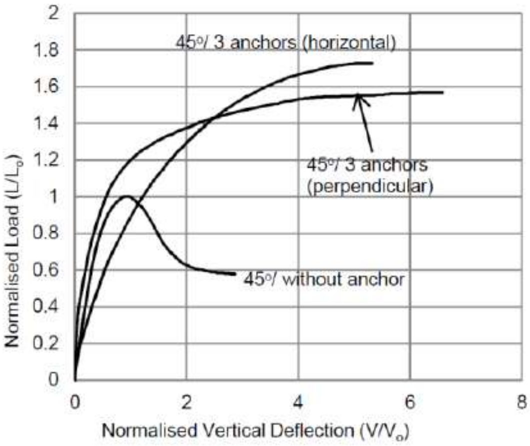


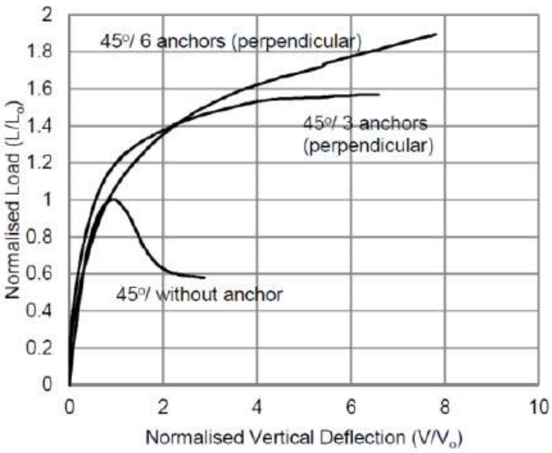
Figure 26 Cross-sectional view of the slope included with granular column anchors

A hand auger was used to bore holes (32 mm diameter & 500mm long space permitting) in the soil bed through to the back or base of the box, whichever was reached first depending on the orientation of the anchors (Figure 26, P stands for perpendicular column and H stands for horizontal column). Following this, 500mm long steel rods (3mm diameter) connected to a 32mm diameter base plate were inserted into the holes. Crushed basalt with particle sizes ranging between 2.36 mm to 3.35 mm and a friction angle of 42^o, was fed into the hole in small layers (~ 100mm deep) before being compacted using a special tamping rod. Each layer was compacted 12 times in order to achieve a uniform density throughout the length of the anchor and between all anchors respectively. In the case of horizontal column, the model box was tilted by 45 degrees to facilitate an easy construction of the columns. Upon completion of each column, a 250×40×5mm steel capping plate was bolted to the protruding steel rods. Pressure was applied to the top of the test bed by the use of a compression frame as shown in Figure 26. The required slope failure was induced by the use of a rigid loading plate (100mm wide and 200mm deep) which was applied to the test arrangement at a rate of 1 mm/min.

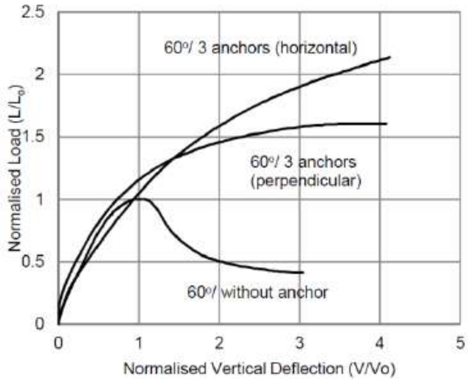
In an effort to mitigate the small scale nature of the study, it was decided to use a normalised presentation to compare the results of the plain and reinforced sections. The normalised results were created by dividing the unreinforced peak load (denoted as L_o in relevant figures) and associated deflection at the time of peak load (denoted as V_o) for vertical deflection against the observed reinforced results. The peak loading capacity corresponds to the failure load within the soil mass, beyond which the integrity of the slope is compromised. Figure 27(a) compares the vertical load-displaced profile of the unreinforced slope with those reinforced with three granular anchors, located either in the horizontal directions or perpendicular to the slope face. The inclusion of granular anchors has clearly enhanced the performance. A comparison of the unreinforced peak strength demonstrated that a similar configuration with three perpendicular anchors produced approximately a 60% increase in capacity. A similar comparison only this time using horizontal anchors produced a 75% increase in capacity. In both anchored cases, there was no evidence of residual stress within the soil mass (failure condition) and the surcharge loading continued to increase at a slow rate even at the termination of test. The slope with anchors positioned in horizontal direction performed better than the other configuration. Figure 27(b) illustrates the influence of increasing the number of horizontal anchors from 3 to 6 on the performance of the same slope profile. The benefits of additional anchors are significant only at large displacement and the relevant performance at the early stage of loading is generally unaffected. The investigation was extended to examine the effectiveness of the granular anchors for stabilizing steepened slopes (i.e. slope angle at 60^o to horizontal). Figure 27(c) shows the load-displacement profile of the steeper slope, reinforced with 3 horizontal or perpendicular anchors. In the case of steepened slopes, the inclusion of anchors in the slope resulted in a 60% and 110% capacity increase for the perpendicular and horizontal anchors respectively, when compared with the plain section. In both cases, the anchors effectively prevented the slopes producing a residual state failure. In all the cases the horizontal movement of the slope was largely restricted by the granular anchors.



(a) Effects of horizontal and perpendicular columns



(b) Effects of column numbers



(c) Steep slope

Figure 27 Normalised load versus normalised vertical displacement of slopes anchored with granular columns

The results of this small scale study have verified the hypothesis that granular anchors can be used to stabilize slopes. Anchors installed in horizontal directions performed better than those installed perpendicular to the slope face. Increasing the number of anchors was shown to augment capacity and ductility at failure, but the enhanced performance must be judged in relation to the cost of installing anchors at full scale. The authors concede that the small scale nature of the tests may have influenced the results, with some of the enhanced performance being attributable to the steel rod running through the anchors, particularly when the anchors are close the loading plate. However in practice (full-scale), such interference would not take place. In terms of full scale applications, installing horizontal anchors is not practical despite offering enhanced drainage within the soil mass. The use of perpendicular anchors will also allow effective drainage and may have wider applications in practice, especially due to ease of installation.

11. CONCLUSION

The following conclusions are made based on the research carried out at the Queen's University Belfast, UK.

- The performance of granular columns in weak deposits can be enhanced significantly by enclosing the columns in geogrids. However an appropriate installation technique should be developed for effective and efficient application in field.
- The interaction between columns and the surrounding clay is dependent on various aspects including the column length and number of columns. The short columns usually fail on punching and long columns fail on bulging, and the direction of bulging is influenced by the configuration of adjacent columns.
- The settlement improvement factor when the columns are employed under strip or pad footing is limited if the boundary conditions away from the footing is flexible. However if the boundary condition is rigid (the settlement improvement factor is significantly high).
- Granular columns also reduce the creep settlement marginally
- Granular columns experience negative skin friction similar to that of solid pile when the surrounding soils undergo consolidation. Under foundation loading, it appears the increase in the vertical pressure in the column is limited beyond L/d of 6.
- Granular columns can be used to limit the settlement of weak sand deposits, however, under dynamic groundwater pressure conditions, the immigration of sand into larger pores of columns generate continuous settlement of the footing.
- Granular columns are effective in providing anchor forces and it can be effectively used to provide ground stabilization process.

12. ACKNOWLEDGMENT

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