# MODEL TESTS ON GEOGRID ENCASED STONE COLUMNS

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**Abstract:** Conventional stone columns can be used to improve the engineering performance of soft cohesive soils for construction and site development. In addition, for sites where the presence of extremely soft soils has prevented the use of stone columns in the past, geosynthetic encasement of columns is now being used to provide the lateral support necessary to make the method feasible. This paper investigates the behaviour of geogrid encased stone columns installed in soft clay through a series of laboratory model tests. Particular emphasis has been placed on comparing the behaviour of partially encased columns to fully encased columns. Test results indicate a significant improvement in load-settlement behaviour with the use of geogrid encasement, even for partially encased columns.

**Keywords:** geogrid, geogrid reinforcement, stone column, encasement, ground improvement, scale models

## INTRODUCTION

Conventional stone columns can be installed within deep deposits of soft compressible soil to create a composite material with improved stiffness and strength. This ground improvement technique has been used extensively over the last forty years to support lightly and moderately loaded structures such as embankments and storage tanks founded on soft soils. The practice is also commonly adopted for land reclamation projects. Conventional stone columns are most often used to improve the behaviour of soils with undrained cohesion, $c_u$, in the range of 15 – 50 kPa (Greenwood & Kirsch 1983). Below this strength, the lateral support provided by the surrounding soil may be insufficient to prevent excessive radial expansion (bulging), resulting in column failure. Despite this, some authors have reported using conventional stone columns in soils with $c_u$ as low as 6 kPa (Barksdale & Bachus 1983; Raju 1997).

In order to extend the use of stone columns to very soft soils ($c_u$<15 kPa), recent research has focussed on using geosynthetics to provide additional lateral support to columns, thereby reducing radial expansion. In many cases, the technique provides a unique and cost-effective ground improvement solution compared to options such as piling. Although several methods of geosynthetic reinforcement have been developed, the method of geotextile encasement developed by Huesker Synthetic GmbH is the only technique regularly implemented on ground improvement projects. Raithel et al (2005) provides a review of the technique with reference to its use on recent projects. Numerous other papers have also been published outlining the success of this method on projects in Europe and South America.

Despite the obvious suitability of geotextile encasement to specific projects, its use may result in more settlement when compared to stiffer geosynthetics such as geogrid. Geotextile encased columns generally receive minimal compaction during installation in order to prevent damaging the geotextile, with subsequent loading resulting in significant column densification and therefore vertical settlement. Furthermore, geotextile tends to strain more than geogrid, thereby increasing radial expansion and contributing to additional vertical settlement. Although comparatively little research into geogrid encasement has been undertaken, it is expected that the use of geogrid will result in stiffer columns with lower compressibility. Ongoing research into geogrid encased columns is being undertaken at Monash University, with the following aims:

- To better understand the behaviour of geogrid encasement and its interaction with the column and in-situ soil.
- To optimise the type and size of geogrid suited to a particular application with consideration also to cost.
- To better identify sites suited to the technique.

The research comprises several components with significant time devoted to understanding encased column behaviour using small-scale laboratory testing of model sand columns. This paper summarises some of the laboratory testing undertaken, including discussion of results and recommendations for further assessment and research.

## BACKGROUND

For the development of an accurate laboratory scale-model, all dimensions and stress conditions need to be reduced by an appropriate scale factor. While it was possible to scale many of the variables relating to stone column behaviour, it was not practical to scale aspects such as in-situ earth pressure without using a centrifuge. In this research, the use of a centrifuge was deemed time consuming and prohibitively expensive. Despite scale effects, many authors including Hughes & Withers (1974) and Muir-Wood et al (2000) have used small-scale testing to successfully model stone column behaviour and derive methods for predicting full-scale behaviour. In this research, it was considered that a well designed small-scale testing program would allow observation of key aspects of encased column behaviour. Particular attention was paid to accurately scaling down aspects such as the ratio of column length to column diameter, ratio of column diameter to cell diameter, ratio of column diameter to aggregate size and ratio of geogrid bar spacing to aggregate size. A $c_u \approx 5$ kPa was proposed for the bed of very soft clay to replicate the type of soil strength in which full-scale columns may be installed.

## TEST CONSIDERATIONS

In geomechanics, laboratory model testing is a popular and relatively inexpensive method of investigating various behavioural characteristics of engineering practice without the cost and inconvenience associated with site testing. The primary aim of the model testing presented in this paper was to investigate column groups and whether the full length of the column needed to be encased with geogrid or whether functionality could be maintained by encasing just the upper section of the column. Research has shown that as a column that acts independently is loaded, it bulges in the upper section of the column. Although this bulging is reduced in groups by confinement provided by adjacent columns, radial column expansion is still observed to be greater in the upper section of the column. This is in part due to lower lateral soil stresses providing less confinement to the upper section of the column and increasing soil stiffness with depth providing additional confinement to the base of the column.

Based on these observations, the need to encase the full length of the column was questioned, at least for soils with adequate stiffness to provide the required confinement to the non-encased section of column. Although the stress conditions of a natural soil deposit could not be replicated in small-scale testing, it was hoped that trends in encased column behaviour could be identified. The schedule of testing was developed to investigate the behaviour of columns with different fractions of encased length, including the upper 25%, 50%, 75% and full length.

## MATERIALS

Laboratory model testing was proposed on the basis that a bed of homogeneous clay was an adequate representation of a very soft natural soil deposit. It was also assumed that a sand column installed within the clay would adequately represent a stone column. The materials used in testing are discussed below.

### Kaolin Clay

A very soft bed of clay with $c_u \approx 5$ kPa was required in which to test model sand columns. The clay needed to provide homogeneous and repeatable samples with a compressibility similar to natural soil. Commercially available powdered kaolin (Grade HR1F, supplied by Unimin Australia Ltd) was used because of its suitable properties. To produce very soft clay, the kaolin was mixed with water to form a slurry with an initial moisture content of about 115% and then consolidated. The high initial moisture content ensured workability and prevented the occurrence of visible air bubbles in the slurry matrix.

Characteristic testing was undertaken to determine the engineering properties of the kaolin slurry presented in Table 1. Consolidation testing indicated that a vertical pressure of 50 – 55 kPa (resulting in about 35% strain) produced samples of very soft clay with $c_u \approx 5$ kPa. Cohesion was measured using laboratory vane shear and pocket penetrometer testing. Permeability was indirectly measured from consolidation tests. The compressibility determined from consolidation testing was used to design the small-scale test apparatus. The engineering properties of the very soft kaolin clay, consolidated from slurry at a vertical pressure of about 50 kPa are provided in Table 2.

**Table 1.** Summary of geotechnical parameters for kaolin slurry

| Parameter | Unit | Value |
|---|---|---|
| Specific gravity | - | 2.64 |
| Approximate clay fraction (<0.002 mm) | % | 85% |
| Approximate silt fraction (<0.02 mm) | % | 12% |
| Plastic Limit | % | 29 |
| Liquid Limit | % | 62 |
| Initial moisture content | % | 115 |
| Initial void ratio | - | 3.04 |
| Compression index, $C_c$ | - | 0.8 |
| Recompression index, $C_r$ | - | 0.09 |

**Table 2.** Summary of geotechnical parameters for consolidated kaolin clay ($\sigma_v$=50 kPa)

| Parameter | Unit | Value/Range |
|---|---|---|
| Saturated density | kN/m$^3$ | 16.2 |
| Moisture content | % | 60 – 66 |
| Void ratio | - | 1.6 – 1.7 |
| Undrained cohesion | kPa | 3 – 5 |
| Angle of internal friction | ° | 18 |
| Dilation | ° | 0 |
| Average permeability | m/s | $1.4\times10^{-9}$ |

### Sand

An aim of the proposed testing was to understand the stress-strain and radial deformation behaviour of geogrid encased stone columns. A commercially available, poorly-graded sand (Grade 8/16, supplied by Unimin Australia Ltd) with similar scaled properties to the crushed rock used in stone columns was used for the model sand columns. The diameter of crushed rock used for stone columns is typically in the range of 25 – 75 mm, equating to a scaled particle size in the range of 1 – 4 mm. Particle size testing indicated that Grade 8/16 sand comprised poorly graded quartz sand with >98% of particles in the range of 1 – 3 mm. Sand particles were generally sub-angular, similar to the shape of crushed rock.

Conventional stone columns are typically installed with a relative density in the range of 60 – 100%, depending on method. For the laboratory testing, columns were installed at a relative density of 90%. As such, density testing was undertaken to determine the minimum and maximum dry density of the sand. Samples were then compacted to a relative density of 90% and tested in a conventional shear box to determine the shear properties of the sand. Constant head permeability testing was also undertaken. These properties are presented in Table 3.

**Table 3.** Summary of geotechnical parameters for Grade 8/16 sand

| Parameter | Unit | Value/Range |
|---|---|---|
| Minimim dry density | $kN/m^3$ | 13.9 |
| Maximum dry density | $kN/m^3$ | 16.5 |
| **Properties at 90% relative density** | | |
| Dry density | $kN/m^3$ | 16.2 |
| Saturated density | $kN/m^3$ | 20.2 |
| Angle of internal friction | ° | 35 |
| Dilation | ° | 8 – 11 |
| Average permeability | m/s | $4.5 \times 10^{-3}$ |

**Geogrid**

Commercially available 'window mesh' was used to replicate the geogrid. The mesh was fabricated from fibreglass with a thin plastic coating. The fibreglass mesh had suitable scaled dimensions such that the scaled-down ratio of geogrid bar spacing to average aggregate size was in proportion to what would be expected in full-scale. Mesh tensile stiffness and strength were determined from uniaxial tensile testing of 25 mm wide strip samples. The mesh was formed into a cylinder with diameter of about 50.5 mm, using an epoxy-resin to bond a 10 mm wide section of overlap. This method was used in preference to stitching the mesh because testing showed that stitching induced points of weakness. The resin was cured to full strength over 3 days and tested in uniaxial tension, verifying that the overlap strength was as strong as the mesh itself. Samples soaked for 28 days were also tested, showing that although there was some loss of tensile strength with saturation, there was little reduction in stiffness. The mesh properties are presented in Table 4.

**Table 4.** Summary of geotechnical parameters for fibreglass mesh

| Parameter | Unit | Value |
|---|---|---|
| Average bar diameter | *mm* | 0.2 |
| Average bar spacing | *mm* | 1.55 |
| Tensile strength | *kN/m* | 13.0 |
| Soaked tensile strength | *kN/m* | 8.4 |
| Soaked strain at failure | *%* | 3.6 |
| Elastic axial stiffness, EA | *kN/m* | 330 |

## CONSOLIDATION CELL

The primary consideration in the development of the consolidation cell proposed for testing was to make it as large as possible while still remaining economical in terms of fabrication cost and operation time. The proposed cell was required to consolidate the kaolin slurry to the consistency of a very soft clay in which model sand columns could be installed. The apparatus also needed to be capable of further loading the columns in order to simulate group loading. Design of the apparatus was loosely based on those of previous authors including Hughes & Withers (1974), with the aim of functioning as a large oedometer. Kaolin slurry was to be placed inside a 550 mm high and 155 mm diameter cylinder. The cylinder would be mounted to a base plate comprising a porous stone and drainage channel. A piston within the cylinder would load the slurry using air pressure. The piston would be bored through the centre, allowing two-directional drainage within the cell as the sample consolidated.

**Manufacture**

Based on oedometer test results, drained column tests were expected to take up to two months to complete. Two stainless steel cells were therefore manufactured to investigate group loading of encased columns with a third Perspex cell constructed for use in other areas of the research. A photograph of the three cells in operation is presented in Figure 1. The piston of each cell was constructed from aluminium with most other components constructed from stainless steel or brass. All cells were manufactured in the Civil Engineering Workshop at Monash University.

**Figure 1.** Photograph of consolidation cells used to test encased model sand columns

**Theoretical Considerations**

Unit cell idealisation was used in testing to simplify the behaviour of column groups installed beneath a wide uniformly loaded area to that of a single column installed at the centre of a cylinder of soil, which represented the column's zone of influence. This concept is illustrated in Figure 2 and described in detail by Barksdale & Bachus (1983). Based on a 51 mm column diameter, the unit cell testing represented a soil replacement ratio, $A_r$ (ratio of column cross-sectional area to loaded area), of about 11%. This was at the lower end of values typically adopted on site but was considered sufficient for comparison of encased column behaviour.

Although research such as that undertaken by Muir-Wood et al (2000) has shown that the behaviour of columns within small groups can be different to that assumed from the 'unit-cell', it was expected that 'unit-cell' idealisation would adequately represent stone column behaviour beneath the centre of a wide loaded area such as an embankment.

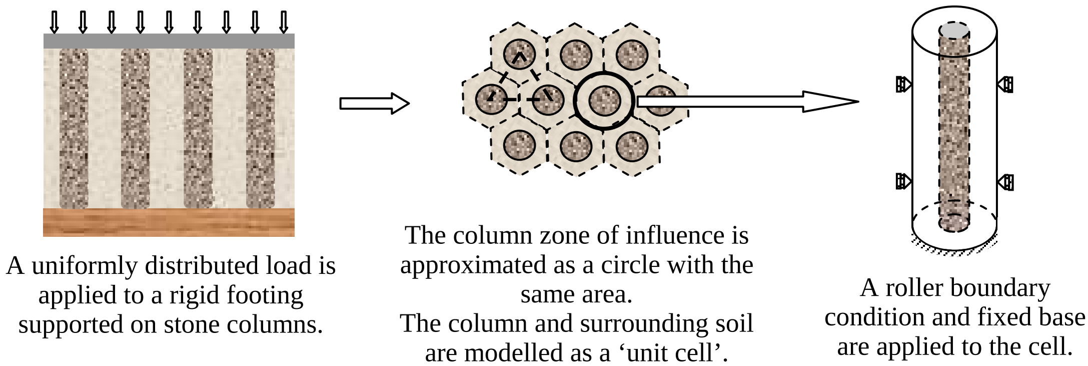


**Figure 2.** Illustration of unit cell idealisation

**Operation**

The cells were designed to operate in two stages. The first stage comprised feeding air pressure to the upper cell chamber, forcing the piston downwards and consolidating the slurry to the consistency of a very soft clay. After unloading and column installation, the column and surrounding clay were then further loaded using the piston, creating the 'unit-cell' condition. Loading was advanced in stages with consolidation taken to practical completion between stages. The cell operation is illustrated in Figure 3.

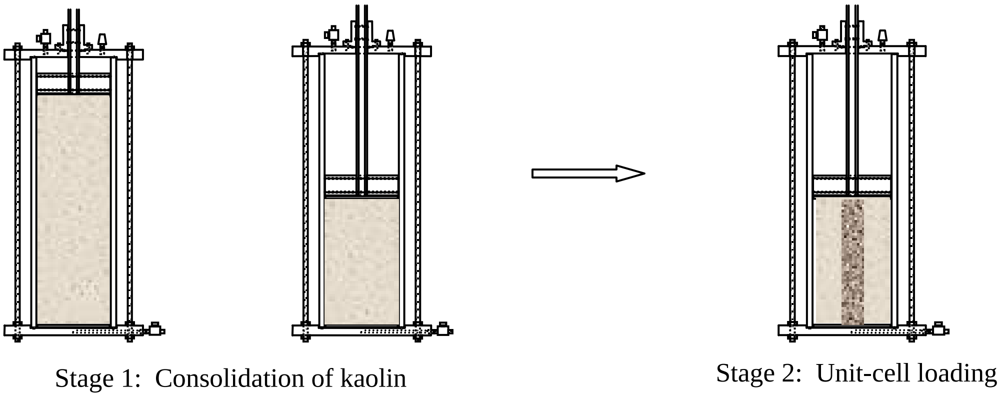


**Figure 3.** The two stages of consolidation cell operation

**Instrumentation**

A string potentiometer (stringpot) transducer with a travel of about 380 mm was attached to the top plate of each cell and used to measure piston displacement and therefore sample settlement. Two pore pressure transducers were mounted to the cylinder wall of each cell, at distances of 100 mm and 150 mm from the cell base. The pore pressure transducers measured sample load and excess pore pressure dissipation. In one of the cells, two miniature stress diaphragm gauges measuring 6.5 mm in diameter were attached to the underside of the piston. The transducers, supplied by TML Ltd. (Japan), had a range of 3 MPa and 500 kPa and measured stress in the sand column and clay, respectively. A 10 channel Datataker and desktop computer provided the necessary data acquisition system.

**Serviceability**

Three types of internal friction were considered to act on the cell, comprising (i) resistance between the o-rings and piston shaft (ii) resistance between the piston o-rings and side-wall of the cylinder, and (iii) friction between the clay sample and the side-wall of the cylinder. It was considered impractical to reduce the resistance acting on the piston shaft, with calibration tests undertaken to quantify its magnitude instead. The internal surface of the cells were bored and polished, and following research by Gachet et al. (2003), were coated with a thin layer of silicone to reduce the magnitude of side-wall friction. Using silicone, test results indicated that about 90% of applied load was transferred to the sample. In addition, consolidation and shear box testing indicated that side-wall friction acting on the sample was reduced by >90%, bringing the load-settlement behaviour of the cell in line with that of a conventional oedometer.

**EXPERIMENTAL PROCEDURE**

Kaolin slurry with an initial height of about 480 mm was consolidated using a pressure of about 54 kPa. At completion, the sample of very soft kaolin clay had an undrained cohesion of 3 – 5 kPa and a height of about 310 mm.

**Geogrid construction:** The geogrid encasement was manufactured by forming the fibreglass mesh into a cylinder of diameter 50.5 mm with 10 mm overlap. Lengths corresponding to 25%, 50%, 75% and 100% of the sample height were constructed. The overlap was then bonded with resin and cured for 3 days.

**Column construction:** Based on past research, it was considered that a replacement method would provide the most reliable and repeatable method of column installation. In-situ compaction of the model column was also avoided in the very soft clay because the potential for inconsistencies in column density was considered high. Model sand columns were therefore constructed using the method of freezing used by Sivukumar et al. (2004). Sand was compacted to a relative density of 90% in a plastic mould measuring 50.5 mm in internal diameter. The sand column was equal in length to the sample height. The sand was saturated with water and frozen at a temperature of -5°C for 24 hours. The intact frozen sand column was then extruded from the mould. Where geogrid encasement was used, the encasement was placed within the mould prior to filling with sand.

**Column installation:** A thin-walled aluminium tube measuring 51 mm in diameter was pushed slowly through the sample to the base of the cell. Cell centrality was achieved by using a guide attached to the top of the cylinder. Following this, the tube was slowly removed leaving a full-height void of 51mm diameter in the centre of the sample. The sample within the extracted tube was used for strength and moisture testing. The extruded frozen sand column was then placed by hand inside the void and allowed to thaw for several hours prior to loading.

**Column loading:** The column and clay (unit cell) were loaded using the piston. Initially, the cell was loaded by 4 kPa above the clay pre-consolidation pressure to ensure intimate contact between the column and clay. The cell was then loaded in increments ranging from 25 – 70 kPa to a maximum cell pressure of about 350 kPa. Consolidation was completed between load stages, as determined from settlement and pore pressure measurements.

**Sample extrusion:** At test completion, a rigid layer of plaster was cast at the top of the sample. The sample was then extruded from the cylinder using hand pressure and carefully cut in half to observe column deformation.

## RESULTS & DISCUSSION

Kaolin was initially consolidated in one of the cells to provide base data for the column group tests. A non-encased column test was then undertaken in each cell, verifying similar cell performance. Load tests were undertaken on columns with 25%, 50%, 75% and 100% fibreglass encasement. Sample properties for each test are summarised in Table 5. Vertical stress-strain plots for these tests are presented in Figure 4 and the reduction of vertical strain compared to kaolin clay behaviour is presented in Figure 5. As each test was commenced at a slightly different pressure (ranging from 55 kPa – 59 kPa), vertical strain reduction was assessed by adjusting each test to a start point of 59 kPa using fourth order polynomial equations.

**Table 5.** Summary of column properties and results for column group testing

| Test | Initial kaolin properties | | | Column properties | | Test Results | |
|---|---|---|---|---|---|---|---|
| | kaolin moisture content (%) | kaolin $c_u$ (kPa) | sample height (mm) | relative density (%) | encased length (mm) | approx. maximum radial strain (%) | average vertical strain reduction (%) |
| Kaolin clay | 64 | - | 311 | - | - | - | - |
| Non-encased | 62 | 5.5 | 313 | 90 | - | 9 | 25 |
| Non-encased (2) | 62 | 5.7 | 306 | 94 | - | 8 | 25 |
| 25% encased | 62 | 5.1 | 293 | 92 | 78 | 21 | 31 |
| 50% encased | 67 | 3.7 | 317 | 93 | 159 | 30 | 40 |
| 75% encased | 63 | 3.8 | 315 | 93 | 233 | 33 | 49 |
| 100% encased | 59 | 5.3 | 311 | 92 | 311 | 7 | 79 |

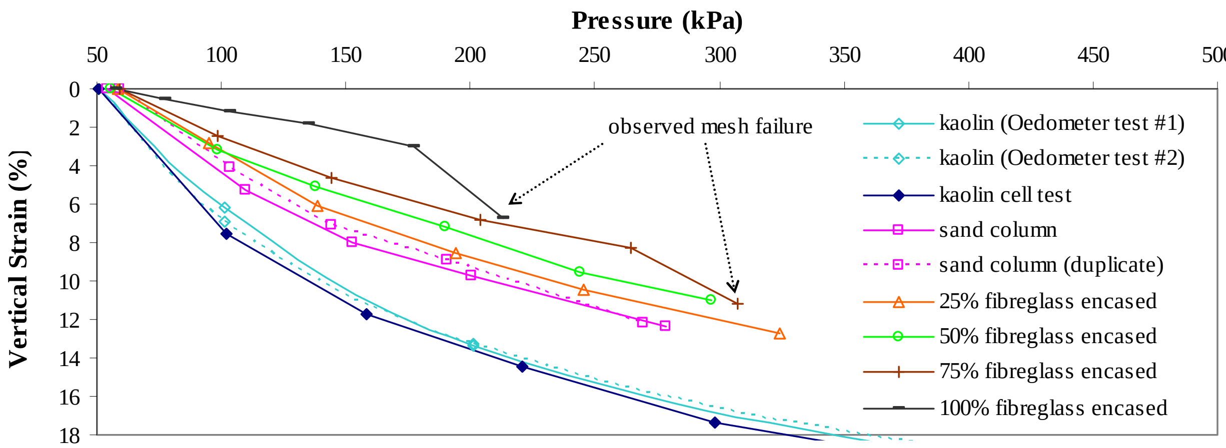


**Figure 4.** Vertical stress-strain behaviour of column tests compared to kaolin cell and oedometer tests

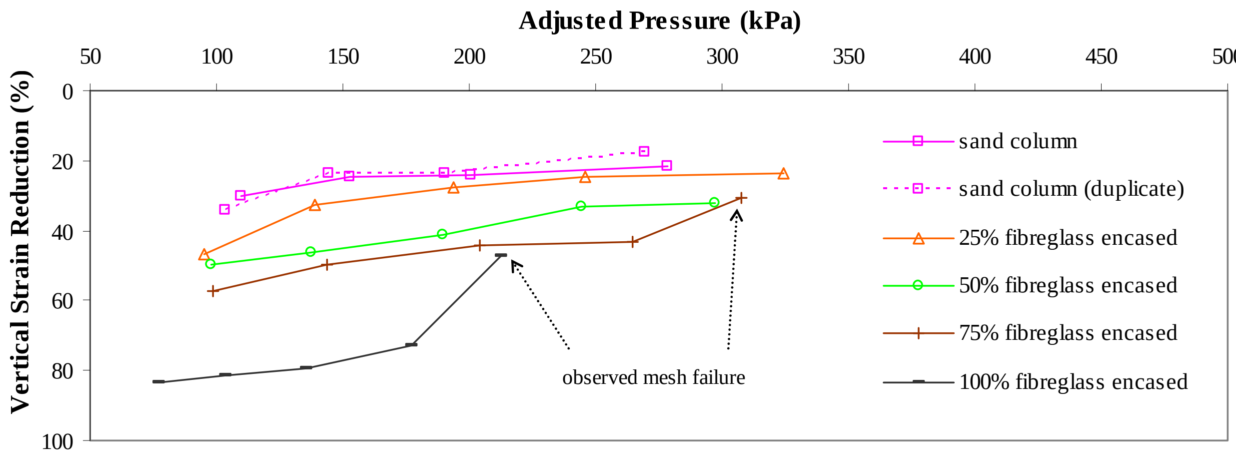


**Figure 5.** Vertical strain reduction with varying percentages of geogrid encasement compared to kaolin alone

### Vertical stress-strain behaviour

As illustrated in Figure 4, the vertical strain of the kaolin cell test closely matched that of the oedometer tests, suggesting that the silicone side-wall lubrication worked very effectively. Test results indicated a significant reduction in vertical sample strain through the use of non-encased model sand columns when compared to clay behaviour alone. For non-encased columns, the average strain reduction across the range of applied stresses was about 25%, a result typical for a low replacement ratio of $A_r$=11%. The observed behaviour of decreasing strain reduction with increasing pressure, outlined in Figure 5, can be attributed to the clay becoming stiffer during consolidation. The stiffer clay results in the clay supporting more vertical load, thereby reducing the impact of the stiffening column.

For the 25%, 50% and 75% encased tests, the average strain reduction was about 30%, 40% and 50%, respectively. The partially encased tests indicated a similar trend to the non-encased tests in that as the clay became more stiff through consolidation, the impact of the stiffening column on vertical strain was reduced.

For the 75% encased column test, the mesh was observed to fail in the upper section of the column. The cell stress at failure was similar to the maximum cell stress for the 50% encased test, which remained intact. This may have indicated that peak hoop forces in the mesh were higher for the 75% encased column or at least that the peak forces acted along a greater length of mesh, increasing the likelihood of encountering zones of weakness.

For the 100% encased column test, strain reduction averaged about 80% prior to failure, which again occurred in the upper section of the column. Failure occurred at a lower vertical stress than for previous tests, a further indication that peak hoop forces for the model columns may have increased with greater encased length. Photographs of test cross-sections following extrusion are presented in Figure 6.


**Figure 6.** Photographs of column cross-sections following sample extrusion

**Radial column expansion**

Extruded samples were carefully bisected to enable measurement of column deformation. Given the coarse grain size of the sand, deformations were measured to an accuracy of about 0.5 mm. Non-encased columns were observed to laterally expand evenly along the entire column length. This was expected as there was little difference in lateral in-situ stresses along the length of the column due to the small sample size.

Although the mesh was too fine to practically instrument with strain gauges, the deformed shape of the clay in areas of mesh encasement indicated some radial straining of the mesh during loading. In areas where the mesh had not failed, this radial strain was roughly measured to be in the range of 1% to 4%, a similar magnitude to the failure strain measured in tensile testing. For partially encased columns, the entire non-encased length was observed to expand laterally, with the magnitude of radial expansion being much greater than for the sections of encased column. For the 75% encased column, the radial strain was measured to be as high as about 33%. Larger lateral deformations occurred near the base of the encasement and generally increased in magnitude with increasing encased length.

**Stress concentration**

Vertical column stresses were only measured for the fully encased column test. A column stress of >1000 kPa was measured prior to column failure, indicating that the ratio of column stress to clay stress (stress concentration ratio) was >10. As expected, the stress concentration ratio was significantly higher than the range of 2 – 4 expected for non-encased columns, further demonstrating that full length encasement significantly increased column stiffness.

**Cell drainage**

Although assessment of the time rate of consolidation was not a primary aim of this research, much information was collected on this component as part of testing. The tests confirmed the expectation that columns acted as effective vertical drains. Consolidation time rates for most column load stages were typically reduced by a factor of about 3 – 4 when compared to kaolin alone. Deconstruction of the samples following testing did reveal significant ingress of clay fines into the model columns (by a margin of up to 6 mm), possibly reducing the drainage efficiency of the column. This observation will be further explored through testing and numerical modelling as part of ongoing research.

**Summary of test behaviour**

The results of model testing of encased column groups can be summarised as follows:

- Column vertical strain is steadily decreased with increasing percent encased length.
- Foundation stresses are, in part, translated to geogrid hoop stresses, thereby reducing vertical column strain.

- Column radial strain is significantly reduced by encasement, although for partially encased columns, bulging tends to occur directly below the base of the encasement.
- For columns with a greater percent encased length, more of the foundation stress is translated to geogrid hoop stress, as evidenced by mesh failure and vertical strain reduction.
- Model columns tend to act as effective vertical drains.

## CONCLUSIONS

This research aimed to investigate the impact of geogrid encasement on stone column behaviour using laboratory testing. The testing focussed on comparing the behaviour of partially encased columns to fully encased columns using model sand columns to replicate stone columns and fibreglass mesh to simulate geogrid. Test results indicated a significant reduction in column vertical and radial strain with increasing percent encased length. For all partially encased tests, bulging occurred directly below the geogrid.

Although scale effects will undoubtedly result in differences between model and full-scale column behaviour, it is expected that the load transfer mechanisms between column, geogrid and clay observed in testing will be similar to full-scale columns. As such, it is expected that full-length geogrid encasement can extend the use of stone columns to extremely soft soils in the same way geotextile has been used in the past.

For partial encasement, soil stiffness and lateral stress below the base of the encasement is expected to govern column behaviour. It is therefore likely that partially encased columns could be used in soils of lower strength than conventional stone columns but probably not to the same extent as fully encased columns. In addition, model testing has shown that partially encased columns may be an innovative technique for some of the following applications:

- Partial encasement may be well suited to projects where stricter settlement control is required than can be provided by conventional stone columns.
- Partial encasement may be adopted to reduce the number of stone columns used on a project (by increasing their spacing) while delivering the same reduction in vertical settlement, providing a cost saving.
- Settlement could be 'tailored' to design requirements by varying the length of encasement.
- Partially encased columns could be used to soften the 'step' in differential settlement that often occurs between areas of compressible soil and rigid structures such as piled abutments. This could be achieved by progressively increasing the encased length in the approach to the structure.
- Partial encasement may be used on sites with soils of layered stiffness, such as extremely soft soil overlying firm soil.

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