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Abstract

A stone column typically consists of particles whose influence has largely been overlooked in design practice in terms of stress transfer, pattern of deformation, and intrusion of fines (clogging). This article presents an experimental study on the load-deformation behavior of a model stone column installed in soft clay with a particular emphasis on the influence of particle gradation and shape under undrained loading. The results show that particle gradation and shape have a significant influence on the load-deformation behavior and the extent of fines intrusion into the stone columns. Relatively well-graded particle sizes favor the development of higher peak shear stresses accompanied by lateral bulging, whereas more uniform grading results in the development of distinct shear planes and smaller peak shear stresses. Deformed columns were also examined using computed tomography, and the porosity profiles at the end of the test were determined using micrographs. Maximum porosity typically occurred in the zone of extreme lateral deformation, with the results suggesting that the extent of fines intrusion was influenced by particle morphology.

Keywords

stone, clay, performance, soft, shape, gradation, particle, influence, columns

Disciplines

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Influence of particle gradation and shape on the performance of stone columns in soft clay

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Abstract: A stone column typically consists of particles whose influence has largely been overlooked in design practice in terms of stress transfer, pattern of deformation, and intrusion of fines (clogging). This paper presents an experimental study on the load-deformation behaviour of a model stone column installed in soft clay with a particular emphasis on the influence of particle gradation and shape under undrained loading. The results show that particle gradation and shape have a significant influence of the load-deformation behavior and the extent of fines intrusion into stone columns. Relatively well graded particles sizes favour the development of higher peak shear stresses accompanied by lateral bulging whereas more uniform grading results in the development of distinct shear planes and smaller peak shear stresses. Deformed columns were also examined using Computed Tomography and the porosity profiles at the end of the test were determined using micrographs. Maximum porosity typically occurred in the zone of extreme lateral deformation, with the results suggesting that the extent of fines intrusion were influenced by particle morphology.

subject headings: Ground Improvement; Soft soils; Stone columns; Micromechanics; Computed Tomography.

1 INTRODUCTION

2 Reducing the settlement of infrastructure and providing cost-effective foundations with
3 sufficient load-bearing capacities are priorities when developing infrastructure (Han and Ye
4 2001, Indraratna et al. 2013). In recent years, a large amount of infrastructure has been built
5 on soft soils due to space restrictions and other factors such as socio-economic,
6 environmental, and flood control. Soft soil foundations can cause excessive settlement which
7 can lead to failure of infrastructure unless there is adequate ground improvement. Stone
8 columns are commonly used as a soil improvement technique to: (i) increase the bearing
9 capacity, (ii) reduce the total and differential settlements of infrastructure, and (iii) accelerate
10 the rate of consolidation (Han and Ye, 2001).

11 Reinforcing the ground by installing stone columns is one of the most well-
12 established and effective techniques practiced worldwide (Black et al. 2006, Murugesan and
13 Rajagopal 2009, Yoo 2010, Fattah et al. 2011), among others. Upon external loading, stone
14 columns distribute the applied stress and deform laterally, especially in their upper zones,
15 rather than transfer the stresses into the deeper layers (Tan et al. 2008, Chai et al. 2010). The
16 deformation of stone columns placed in clay has been the subject of an extensive number of
17 experimental and numerical modelling studies (Blewett and Woodward 2002, McCabe and
18 Egan 2010, Ngo et al. 2016), among others. These researches indicated that stone columns
19 decrease the drainage paths in soft clays, which accelerate consolidation and increases the
20 load carrying capacity due to the subsequent reduction in settlement. While Castro and
21 Sagaseta (2011) reported that vertical stress in a column increases as time-dependent
22 consolidation occurs, this only true if the stone column has not yielded or failed. In some
23 cases placing a large load such as a high embankment, over a relatively short construction
24 period is necessary, in which case the stone columns may yield or fail before the surrounding
25 clay has become sufficiently consolidated (Barksdale and Bachus, 1983). The yielding or
26 failure of stone column influences its load-carrying capacity and interaction with the

surrounding clay (Balaam and Booker, 1985). While past research studies examined the consolidation of a soft clay-stone column system in model tests (e.g. Sivakumar et al. 2003), the effects of short term loading and associated primary settlement of stone column has been overlooked. These effects should be considered because the physical changes within the stone column during short-term loading (e.g. column yielding), can impact the subsequent column performance, particularly its bearing and drainage capacity.

A stone column typically includes aggregates of different sizes and particle angularity, depending on where the material comes from and the quarrying method. The shear strength of angular materials increases with the increase in particle angularity due to a greater degree of particle interlocking (Guo and Su, 2007). The effect of particle angularity on the strength and dilation of granular materials are a subject of previous research. Guo and Su (2007) carried out a series of small-scale triaxial test (50 mm in diameter and 100 mm high) on Ottawa sand and crush limestone in a drained condition and presented that the effect of interparticle locking associated with particle angularity should be considered when determining the shear strength of granular materials. In this study, the Authors conducted large-scale triaxial tests (300 mm diameter, 600 mm high) with a particular emphasis on the influence of particle gradation and shape under undrained loading that has not been reported in Guo and Su (2007)'s study. Guo and Su (2007) conducted element tests, in which the mechanism of shearing of a stone column cannot be adequately reproduced, e.g. the role of the intrusion was not investigated. While a lot of effort has gone into understanding the behaviour of stone columns in model tests (e.g. Muir Wood et al. 2000, Ambily and Gandhi, 2007, McKelvey et al. 2006 Black et al. 2007, Sivakumar et al. 2003, Shivasankar et al. 2010, Indraratna et al. 2015), very few studies have investigated how the size and shape of particles affects the stress-strain behaviour and physical changes of columns under shearing (Cho et al. 2006; Guo and Su 2007; Thom and Brown 1998). This assessment is important because previous studies indicate that the angle of shearing resistance and the moduli ratio of stone

column which is influenced by particle morphology, could affect its overall performance in reinforcing soft soil (e.g. Hanna et al. 2013, Etezzad et al. 2014).

This paper presents a series of undrained triaxial tests performed to study the short-term load-deformation behaviour of a model stone column; it includes a macroscopic examination of deformed columns and the corresponding changes using the X-ray CT-scan technique, and also studies the stress-strain behaviour associated with columns having different particles sizes and angularity. In addition, the testing conditions adopted in this study, i.e. undrained condition with measurement of excess pore water pressure differs from the results reported in previous studies conducted mainly under drained conditions. For instance, Castro and Sagaseta (2011), Balaam and Booker (1985), Sivakumar et al. (2003), among others carried out tests on model columns in drained conditions where pore water pressure was allowed to dissipate; and therefore the yielding of stone column did not commonly occur in a short term loading.

EXPERIMENTAL PROGRAM

A number of Consolidated Undrained (CU) triaxial tests were carried out on a model unit cell consisting of a model stone column and surrounding soft clay. The first series were to enable X-ray CT-scan imaging, and the second series were performed on the fully-instrumented unit cell in the laboratory. Three out of four tests from the first series were repeated in the second series to assess the repeatability of the results because the particle morphology and size distributions varied. All the tests are summarised in Table 1. The influence that particle size distribution (PSD) and particle shape has on the stress-strain behaviour and deformation patterns observed in the model unit cell were assessed in two sets of testing, T3A/T3 and T4A/T4, and T4A/T4 and T5A/T5, respectively. Table 1 includes the relevant morphological properties of column particles adopted in these tests. Other general index properties of clay

and column materials are shown in Table 2. A number of triaxial tests using 100-mm diameter clay specimens were carried out under Unconsolidated Undrained (UU) condition to evaluate the undrained shear strength of the clay slurry. The samples were 200mm high, thus the aspect ratio typically adopted for triaxial testing ($H=2D$) is maintained. While similar results are to be expected for smaller diameter specimens (38mm or 50mm), in this study large diameter specimens were preferred to replicate the clay behavior likely to be obtained in the 300 by 600mm model columns.

Sample Preparation and Material Properties

A typical test setup for the pre-consolidation stage has been described by Siahaan et al. (2014). Here, the soft clay was prepared by mixing kaolin clay at 1.2 times its liquid limit, and then placing the slurry into a rigid 300 mm diameter cylinder that was wrapped internally with a 3 mm thick rubber membrane with a polished surface. Initially, the sample was 750 mm high and 300 mm in diameter, but during placement, the 100mm thick slurry was lightly compacted by vibration to eliminate any air bubbles. It was then pre-consolidated under a vertical pressure of 65 kPa until it reached a 95% degree of consolidation, after which the sample is approximately 600 mm high. The unconsolidated undrained (UU) triaxial tests of kaolin clay show an average undrained shear strength of 15.5 kPa, and an average peak deviatoric stress of 31 kPa. The index properties of kaolin clay are given in Table 2. The materials used for the stone column can be divided into 2 types, as shown in Figure 1a, where the material M1 generally consisted of angular igneous rock particles of basaltic origin, and material M2 consists of polished river pebbles which are generally sub-rounded in shape. Each column specimen has a specific type of material with a specified PSD. In the first series the angular particles (M1) were prepared in two sets of PSDs, as shown in Figure 1b, while the sub-rounded particles (M2) were prepared for one specified PSD. It is noted that the materials M1 and M2 selected have different shear strength and stiffness. However the shear

strength behavior of the different column materials adopted, determined in independent direct shear tests (Figure 2a-d), show that they are comparable, i.e. having comparable peak and constant volume friction angles. Thus, while the materials are different comparisons still can be made. The PSD sets were determined such that the ratio of column diameter (d) to particle diameter (d_{p50}) is less than 20 to allow a shear band to form without undue restraint (Roscoe, 1970). The grading composition of column PSD T4 is similar to a typical stone column in the field and this test was used as a benchmark for comparison. Despite particle scaling, the main material type (M1) in the model column could not differ from the type used in a typical prototype column in order to maintain the same inter-particle friction, which is a function of surface roughness. Details of the materials used in each test are listed in Table 1.

Large-scale direct shear tests (300 mm long x 300 mm wide x 200 mm high) have been carried out on stone column materials to determine their apparent inter-particle friction angles. It is noted that triaxial element tests are indeed superior but quite limited in the maximum particle size they can examine. In this study, the maximum particle size adopted was 16mm, thus element tests were not possible. As these tests aimed at determining the apparent inter-particle friction angles of materials used to make stone columns, the use a large scale shear box that could readily accommodate testing of larger particles sizes was preferred as it was less time consuming compared to large scale triaxial tests (300 by 600mm). Furthermore, past studies on the shear strength behavior of granular materials demonstrate comparable results between direct shear and triaxial states as presented by Rowe (1969). The specimens were then placed and compacted by similar levels of compaction energy to achieve approximately 85% relative density, after which normal pressures of 28 kPa, 54 kPa and 78kPa were applied. The resulting friction and constant volume friction angles are shown in Figure 2d. The constant volume friction angle (ϕ_{cv}') was obtained by subtracting the dilation angle (ψ) from the peak friction angle ϕ' .

The variation of properties pertinent to particle angularity and PSD are determined by ensuring that any variation on the continuum friction angles is appreciable. Cho et al. (2006) presented a correlation between particle roundness (R_I) and the critical state friction angle (ϕ'_{cv}), as shown by Eq. 1 below.

$$\phi'_{cv} = 42 - 17R_I \quad (1)$$

The results from direct shear testing (Fig. 2) and Eq. 1, for the same PSD and R_I values can be back-calculated for materials M1 and M2, and they are approximately 0.25 and 0.70, respectively. Particle roundness denotes the average radius of the curvature of surface features relative to the radius of the maximum sphere that can be inscribed for a given particle (Cho et al. 2006). Based on the roundness chart provided by previous studies (e.g. Powers 1953, Krumbein and Sloss 1963), the materials M1 and M2 shown in Fig. 1a can be classified as angular and sub-rounded materials, respectively.

The friction angle of typical materials derived from latite basalt (igneous) varies slightly due to differences in packing (Indraratna et al. 2011). The grading parameter “ n ” (Thom and Brown 1988 and Indraratna et al. 2011) expressed in Eq. 2 can be used to qualitatively assess the influence of PSD on the peak friction angle.

$$\% \text{ Finer} = 100 \left(\frac{d_p}{d_{p100}} \right)^n \quad (2)$$

where, d_p and d_{p100} denote a particle at any given finer percentage and at maximum size, respectively. PSD sets PSDT3 and PSDT4 have average “ n ” values of approximately 0.32 and 1.05 respectively, and the variation in the peak friction angle of about 3° between PSDT3 and PSDT4 (Fig. 2) is consistent with the variation shown by Thom and Brown (1988).

The model stone column in this study was installed using the replacement method. To install a stone column, a thin-walled tube with an area ratio of less than 10 % (La Rochelle et al. 1981) was used to create a 102.5 mm hole in the centre of the specimen; this corresponds to an area ratio (A_r) of 11.7%. A smaller tube was then used to remove excess clay inside the first tube, while the first tube supported the surrounding clay. Column particles were then placed in several, 100mm thick layers where their weights were measured regularly to ensure that a relative density of 83 – 87% was achieved. The ratio of column length (L) to diameter (d) is constant for all tests and a value of 6.0 is adopted. An “ L/d ” ratio of 6.0 and an “ A_r ” of 11.7% were considered to be enough to remove any constraints that could prevent any localised column deformation such as bulging, shearing, and buckling (McKelvey et al. 2004, McCabe et al. 2007, Black et al. 2011).

Large-scale Triaxial Testing

Once the stone column was installed, the unit cell specimen was transferred to a large-scale triaxial apparatus in a process that involved removing the wrapped-around casing and fitting the triaxial cell. To minimise any disturbance, a small vacuum pressure of 5 kPa was applied through the bottom drainage valve and the weight of the top piston was suspended before applying any confining pressure. A 10-mm thick layer of sand sandwiched between two geofabric layers was also added at the top and bottom of the sample to provide a flexible support at both ends. The flexible support was used to eliminate the influence of the rigid boundaries and to provide better stress distribution (i.e. applied loads are uniformly distributed across the sample). A typical setup of the large-scale triaxial apparatus is shown in Figure 3.

The unit cell specimen was then saturated by applying 35 kPa of back pressure to the drainage valve while maintaining a vacuum pressure of 30 kPa onto the top valve (Bishop and

Henkel 1964, Head 1998). When saturation finished, a minimum Skempton's B value approaching of 0.95 (ASTM, 2011) was achieved. Saturation was primarily for the stone column and the adjacent column-clay interface because the undisturbed clay portion was prepared in a wet condition (i.e. 1.2 times liquid limit) before pre-consolidation.

The unit cell specimen underwent isotropic consolidation under an effective cell pressure of 25 kPa and then anisotropic consolidation followed by incrementing the cell pressure and axial pressure to maintain a stress ratio (K) of approximately 0.7 (Head, 1998). The value of $K = 0.7$ is considered to be appropriate based on typical values used in the literature to reflect typical field conditions for column installation, as reported by Black et al. (2011).

At the end of consolidation the sample was then sheared under undrained conditions at a relatively slow rate of 3% axial strain per hour to enough time for the excess pore pressure in the sample to equalize during testing, but can still remain an acceptable time required to complete one tests (i.e. approximately 7 hours). The shearing stage continued until an axial strain of about 20% was attained (tests T3A, T4A and T5A). In test T3B, a smaller axial strain of 7.5%, based on the stress-strain results from test T3A, was used to investigate the physical condition of the column at its peak strength. The membrane has also been corrected in accordance with ASTM D4767-11 (ASTM, 2011) and Head (1994, 1998), and an additional test ST1, where only the clay sample was sheared, was also carried out for comparison purposes.

Computed Tomography (CT) Scanning

To quantify the influence of particle shape and angularity it is important to understand the changes that occur in the soil structure surrounding the column, and the associated stress-strain behaviour. X-ray Computed Tomography (CT) techniques are normally used to study

the structure of soil specimens under various conditions (e.g. Oda et al. 2004, Heitor et al. 2013), and a CT scan has been used here to investigate column deformation and the extent of fines intrusion.

After shearing, the central portion of the deformed specimen was extracted for CT Scanning tests using a thin-walled PVC tube (Fig. 4a). Thickness of the thin-walled PVC tube used in the laboratory was 2.5 mm and this thickness was in accordance with the range of the ratios between sample diameters to sampler thickness provided by Rochelle et al. (1981). To minimise any disturbance to the stone column and clay adjacent to the column-clay interface, the tube has an inside diameter of 225 mm and a tapered end with an area ratio of 10 % (La Rochelle et al. 1981). Furthermore, the inner wall of the tube was lubricated to prevent any compression induced by friction between the sample and the wall. As a sample is being extracted, a 30mm to 50mm long section was trimmed from the sample so that a flat plate can be inserted at the base to keep the granular particles intact. Both sides of the tube were sealed and semi-rigid cushions were inserted into the remaining voids at both ends. The typical CT-scanning setup is shown in Figure 4(b). The tests were carried out in a high-voltage X-ray CT scanner using procedures similar to those described in Heitor et al. (2013) and Heitor (2013). The reconstruction function used in this study enables image artefacts to be corrected, that might result from having lower energy X-rays. The voltage and current of the X-ray tube were 135 kV, and 350 mA, respectively. The X-ray beam was 7 mm wide (i.e. slice thickness), the exposure time was 1 second, and the field of view (FOV) was 21 cm.

The cross sections obtained from CT-scanning were processed using DicomWorks v 1.3.5 (Puech et al. 2007) and suitable window ranges to ensure the column materials, water-filled voids, and clay, with inherently different densities, can be distinguished. Further analyses using the cross sections were also carried out using the Image Processing toolbox of Matlab v7.14 R2012a, where area of pixels representing the column particles and voids ($A_{p,void}$) can

be captured by selecting the appropriate threshold values represented by different colours in greyscale images, as shown in the histogram plot in Fig. 5a. The area of pixels covering the entire cross-section ($A_{p,total}$) excluding the zone with fines intrusion was calculated by superimposing the entire cross-section with a single white colour (Fig. 5c). The zone with fines intrusion was trimmed and excluded because the calculated porosity refers to the uncontaminated zone of the stone column (Fig. 5b).

It is acknowledged that the porosity calculated directly from the pixels area of the cross-section is generally accurate in two dimension (2D), so this 2D porosity value was converted to porosity (n_{3d}) using a conversion factor ($x_{2d \rightarrow 3d}$):

$$n_{3d} = x_{2d \rightarrow 3d} \left(\frac{A_{p,void}}{A_{p,total}} \right) \quad (3)$$

In this study the porosity obtained by weight-volume relationships is used as a comparison measure among the different model columns tested, thus it is necessary to convert the CT-scan based 2D porosity into 3D. Typically to determine the 3D porosity, the pore skeleton needs to be imaged, but due to the limitations of the CT-scanner used, the pore skeleton reconstruction was not possible and thus conversions factors were adopted. The conversion factors were determined by comparing the 2D porosity values obtained for cross-sections that did not undergo any change in the porosity and the initial porosity obtained via weight-volume relationships. Certainly although the computed 3-D porosity values are influenced by the conversion factors, the values obtained are appropriate for qualitative comparisons between different model tests. The conversion factor has been obtained by comparing the two-dimensional porosity values of cross-sections which did not undergo any actual change in porosity (n_{3d}). The values of this factor ($x_{2d \rightarrow 3d}$) are determined as 0.81, 0.71 and 0.72 for columns T3A, T4A, and T5A, respectively. The resulting porosity values are then checked against the initial porosity values pertinent to the minimum and maximum dry densities for

each column. Although the computed three dimension (3D) porosity values are influenced by the conversion factors, the values obtained are appropriate for qualitative comparisons between the aforementioned tests. Moreover, this approach enables a direct evaluation of the porosity profile and to identify locations where relative changes occurred along the stone column. The porosity profile for each column was evaluated using 15 – 17 cross sections spaced evenly along the column length. Each column diameter was determined by averaging 6 measurements in a cross section, while the thickness of the fines intrusion band was an average of 12 measurements taken at equal distances along the perimeter of the given cross-section.

In test T4, a number of optical Fibre Bragg Grating (FBG) sensors and miniature pore pressure transducers were installed to measure the lateral strain of the column and induced pore pressure in the clay located adjacent to the column, respectively. The details of strain measurement using FBG are presented in Hussaini et al (2015). The FBG sensors were glued on a flexible and grooved rubber strip placed along the length of stone column at an offset of approximately 10 mm from the column edge. One of the FBG sensors was positioned at about 200 mm below the top of the column prior to the start of the testing. One of the miniature pore pressure transducers was installed at about 200 mm below the top of the column at an offset of about 25 mm from the column edge.

RESULTS AND DISCUSSION

Load-deformation response and deformation shapes

Fig. 6a shows the stress-strain behaviour of the model columns tested with different gradations (T4A and T3A) and different particle angularity (T4A and T5A). The test conducted with kaolin clay (ST1) is also shown for comparison; as expected, including the

column resulted in a substantial increase in the peak deviatoric stress attained in the tests, which indicates that the column provides a greater load-bearing capacity to the unit cell, and particle angularity appears to influence the peak shear stress much more than particle gradation. Larger peak deviatoric stresses are obtained for columns with angular particles (T4A); for instance, the column with angular particles has a peak deviatoric stress of 82kPa, whereas the column with sub-rounded particles attained a smaller peak deviatoric stress of 72kPa. However, the column with sub-rounded particles (T5A) has a stiffer initial response, possibly due to the particle interlocking effect, i.e. the initial density attained in this column is larger for the same relative level of compaction, which would indicate a larger number of inter-particle contacts. The development of larger deviatoric stresses and an initially stiffer response is favoured by a less-uniform distribution (T3A), i.e. a peak stress of 90kPa is attained for 6% to 8% axial strain compared to 82kPa at 8% - 9% axial strain for the column with a more-uniform gradation (T4A). This may be attributed to the greater particle interlocking and lower void ratio observed in the less-uniform gradation column (T3A). It is also noted that the deviatoric stress in material M1 is greater than that in material M2 and this is partly associated with the angularity of the particles as clearly illustrated by the extend of fine intrusion.

The pore pressure measured at the base (Fig. 6b) indicates that the entire unit cell is contracting, hence the generation of positive excess pore pressure. Upon applied loads, the upper parts of stone column tend to bulge laterally (i.e. dilating) that causes the contracting of the surrounding clays; and this results in an increase in stresses measured in clays. Black et al. (2007) conducted series of tests to study the performance of clay samples reinforced with vertical granular columns and also observed that the increased lateral stresses within the surrounding clay lead to further consolidation and enhanced resistances to column bulging. In fact, the soft clay that forms the bulk of this specimen is generally contracting while the stone

column is dilating, especially at the latter stage of testing. This is consistent with the observations made by Black et al. (2007), who reported that the dilating stone column material contributed to a decrease in the pore pressure measured during testing. As expected, the greatest magnitude of positive excess pore pressures occurred in the test where no stone column has been installed (ST1) (Fig. 6(b)).

The images of deformed columns obtained at the end of Tests T3A, T3B, T4A (angular particles but different gradations) and T5A (sub-rounded particles) are shown in Figure 7 while the corresponding magnitudes of lateral strain are shown in Figure 8. The lateral strain can be calculated as a ratio between the change in average radius at the end of the test (Δr) and the initial column radius (r_o). The extreme lateral deformation in tests T3A, T4A, and T5A typically occurs near the middle of the columns, but is still within three to four times the diameter of the column when measured from the top of the columns. The column from Test T4A with more-uniform particles shows the beginning of shearing within its bulging zone at a depth of 100 – 200mm (Fig. 7), but the shear band is not evident in columns T3A (less-uniform particles) and T5A (sub-rounded particles). This shear band may have been caused by the stress state in the column where a higher stress ratio and lower magnitude of mean stress due to lower particle interlocking could encourage localisation (Muir Wood et al. 2000). Figure 7 also depicts the deformed shapes of columns T3A and T3B, which consist of less-uniform and angular particles, at axial strains of 20% and 7.5%, respectively. Note that significant lateral deformation (bulging) presumably occurs once the peak stress is exceeded for axial strains larger than about 6% to 8% (Fig. 6a) because there is no significant lateral deformation in T3B. This is consistent with the results of sand columns tested in triaxial conditions by Sivakumar et al. (2003) where the axial strains did not cause any bulging before reaching peak stresses. At the same axial strain of about 20%, the bulging in Test T5A (sub-rounded particles) is greater in magnitude than in T4A (angular particles) due to particle

1 shape. Moreover, the larger bulging in the column with sub-rounded particles because it
2 reached peak strength at a lower axial strain ($\varepsilon_a = 5\%$) than T4A ($\varepsilon_a = 8 - 9\%$) with angular
3 particles. However, the shear band formed in column T4A influenced its deformation
4 characteristics, but there is less bulging in T4A than in T3A. At an axial strain of 20%,
5 column T4A with more-uniform gradation had already developed a shear band while column
6 T3A with less-uniform PSD was still undergoing bulging. Oda and Kazama (1998) stated that
7 the shear band and its surrounding zone with a localised large void ratio within a sample of
8 granular soil are typically formed when a series of columnar-like structures that transfer the
9 vertical stresses through particle-to-particle contacts, begin to buckle and rotate. This
10 phenomenon then alters the characteristics and orientation of lateral deformation in the
11 column. Note also that the column with more-uniform grading is more likely to develop a
12 shear band because it has a lower stability and larger void ratio than the column with less-
13 uniform grading.

14 The observations from the CT-images can be corroborated by the lateral strain
15 measurements represented in Fig.8. Although there have been several past research studies
16 reporting that the bulging zone typically occurs within the top third of the stone column,
17 Sivakumar et al. (2007) demonstrated that the ratio of length over the diameter of stone
18 column (L/d) governs the failure mechanism and deformation patterns (e.g. bulging zone). For
19 instance, long columns (length/diameter: $L/d=10$) deformed significantly in the upper region
20 while short columns ($L/d=6$), bulging was less significant in top section. In this study, the
21 Authors used the ratio $L/d=6$; which can be classified as short columns and observed the
22 bulging zones occurs near the middle of the columns, but is still within three to four times the
23 diameter of the column when measured from the top of the columns.

24 Figure 9 shows the test results repeated in the second test series for Columns T3, T4
25 and T5 where a miniature total and pore pressure sensors have been included. As expected,

the differences are consistently within 10%, but the slight differences in their deviatoric stresses are consistent and may be attributed to the reinforcing effect of those sensors; these inferred the repeatability of tests T3A, T4A and T5A. Note also that the images of deformed columns in both series obtained from the CT-scanning closely resemble tests T3 and T4, which are similar (Fig. 10a). For a study of the compression of the surrounding clay during the bulging of the stone column, the lateral strain of the column and induced pore pressure in the surrounding soft clay at approximately 200 mm deep below the top of column are plotted for various axial strain values as presented in Figure 10b. The column starts bulging at an axial strain of about 6 %. However the plot also shows that a substantial increase in the rate of bulging occurs from an axial strain of about 9%. As substantial bulging occurs, the surrounding clay located adjacent to the column undergoes compression as indicated by an increase in the induced pore pressure following an axial strain of 9%.

Observations on Column Deformations

Figure 11 shows the typical cross sections of those parts of the column where the most extreme lateral deformations occur. The zone where the extreme fines (clay) intrusion occurs is thicker in column T5A than in column T4A; this is associated with the expanding of a stone column into the surrounding clay and there is a certain amount of clay may be gradually squeezed into the voids within the stone column due to applied loads. For granular materials, it is well accepted that the load-bearing capacity depends on the compaction level, particle size distribution and angularity. All of these aspects have a role on the granular assembly porosity. In this study, the relationship between the model column load-bearing capacity and the porosity was not examined. However, some conclusions can be drawn in relation to the role of gradation and angularity. For instance, for the model columns having the same

gradation (T4A and T5A), the column having angular particles had a greater load-bearing capacity. Similarly, for the columns having the same angularity (T3A and T4A), the column having a less uniform gradation exhibited the largest load-bearing capacity. While a direct relationship between porosity and load bearing cannot be evaluated, the porosity analysis enabled the identification of sections of the model column exhibiting substantial deformation.

In column T4A where shearing occurs, there is a shear band in the cross-sectional view of the upper end (Fig. 11b) and lower end (Fig. 12) of the column segment. That portion of these cross sections traversed by the shear band shows some discontinuities and an area with localised large voids which are consistent with the shear band structure seen in the sheared sandy specimen by Oda and Kazama (1998) and Oda et al (2004).

The porosity profiles from all four tests are shown in Figure 13. It is noted that the porosity profile shows a non-uniform distribution along the depth where compression and expansion are observed in different sections of the column. In tests T3A, T4A, and T5A, the depths at which bulging occurred had more porosity than the other parts of the columns; in fact the porosity of the bulging zones is usually more than the initial porosity achieved installing the column(s). Column T3A increased in porosity by almost 28% in its bulging zone compared to columns T4A and T5A, although the bulging in column T3A is slightly lower than in column T5A. However, Fig. 13 shows that the bulging zone in column T5A is more localised and covers no more than 20% of the total column length, while the bulging zone in column T3A covers about 30% of the total column length. In Figure 13, the increase in porosity of about 11% of the bulging zone of column T5A with sub-rounded particles is less than in column T3A with angular particles. Since some sub-rounded particles of column T5A are expected to dilate radially, the adjacent sub-rounded particles can rearrange themselves to partially refill parts of the void because the rounded particles typically have lower interlocking and inter-particle friction. As a result, the total increase in porosity in the

bulging zone of column T5A is lower than column T3A. Unlike column T4A, column T3A shows continual stable lateral deformation, so it is envisaged that this increase in porosity within the bulging zone of column T3A occurs evenly across the cross section as the column expands. However, the almost 10% increase in porosity in the bulging zone of column T4A, which is more uniform PSD than column T3A, is possibly due to the occurrence of a shear band which may have induced a localised change in porosity. The porosity profiles were determined by the computed tomography (CT) scanning and showed a non-uniformly distributed along the depth of stone columns. It is observed some reduction of void ratio due to applied load within the top part of the columns and this phenomenon could be related to high stress concentrations at the upper part of stone column and due to the intrusion of clay. By comparing the porosity profiles of columns T3B ($\varepsilon_a = 7.5\%$) and T3A ($\varepsilon_a = 20\%$), it can be deduced that a localised extreme compression has initially taken place within the eventual bulging zone prior to any lateral deformation and as a consequence, the porosity has also decreased.

Figure 14 shows profiles of the ratio of the thickness of the intrusion zone (t_c) to the magnitude of lateral deformation (Δr) of a column, which can be conveniently called the penetration ratio. The values of t_c and Δr are measured from the cross-sections obtained from the CT-scanning. A penetration ratio greater than 1.0 represents a reduction in the diameter of the uncontaminated zone of the column. The upper and lower parts of column T5A generally experienced reduction in diameter due to the intrusion of clay. In the upper part of column T5A, the clay intrusion is about 25% thicker than the actual magnitude of lateral deformation. The lower inter-particle friction and weak particle interlocking in column T5A may have contributed to the weaker resistance of rounded particles against the intrusion of clay under an applied load, despite having lower porosity than column T4A. The profile of the penetration ratio between columns T3A and T4A where the PSD varied does not differ very much, but

1 the penetration ratio within the zone of extreme lateral deformation is smaller in column T3A
2 (less-uniform particles) than in column T4A (more-uniform particles). Apart from the
3 influence of shearing in column T4A which led to a localised large void ratio and subsequent
4 clay intrusion, it is also due to the lower void ratio in column T3A where its resistance against
5 clay intrusion had increased. Fines can intrude into a column as the surrounding clay is
6 remoulded by the column exerting lateral pressure during its expansion. This well-known
7 phenomenon typically occurs when stone columns are being installed using vibration (Weber
8 et al. 2010). The zone where fines intrude can vary in thickness along the length of the
9 column and also in those tests where the magnitudes of column deformation and particle
10 morphologies varied (Fig. 14). In addition, observations on parts of the column which have
11 not appeared to bulge infer that only minimum clay intrusion occurs due to possible clay
12 expansion into the column. This indicates that the increase in ratio $tc/\Delta r$ becomes more
13 pronounced as the stone column begins to expand into the surrounding clay.

14 The extent of clay intrusion due to clay remoulding is not only influenced by the
15 magnitude of stress, it is also affected by the morphological properties and PSD of the column
16 particles. In reality, the migration of clay particles into the pores of the granular column
17 inevitably occurs, which leads to a clogged zone and results in a significant decrease in the
18 column permeability (Indraratna et al. 2013). This clogging of stone columns would initiate a
19 reduction in the effective radius of the column in terms of drainage, as well as reducing the
20 permeability in the clogged zone where the dissipation of excess pore pressure can be
21 adversely affected by clogging.

22 Within the scope of this study the stress concentration and stress transfer in the
23 column was not considered as this study focused on the testing aspect, and this is as a
24 limitation of the current research and will be investigated in the future. Indeed, another series
25 of large-scale triaxial tests of stone columns where granular aggregates having different

angularity and varied length/diameter ratio, $L/d=5-10$ are being conducted at the University of Wollongong. In addition, extensive field works on the use of stone columns to reinforce soft soil embankments are carried at Ballina, (NSW, Australia) are currently being undertaken. These studies will investigate the stress concentration and stress transfer in the columns in detail.

CONCLUSION

A series of large-scale laboratory tests has been carried out to study the influence of particle gradation and shape on the performance of stone columns in soft clay. The laboratory results showed that the particle morphology and gradation of stone aggregates affected the stress-strain behaviour, the overall column deformation, and the extent of fines intrusion to the stone column. The X-ray Computed Tomography (CT) Scanning was used to examine the deformation of stone column and the extent of fine intrusion. The following conclusions can be drawn from the large-scale laboratory tests.

- The shape of column deformation is likely influenced by the particle morphology and size distribution. The use of angular particles and less-uniform particle size distribution helped to increase the load-bearing capacity (i.e. an increase of about 10kPa in peak deviatoric stress). It was also noted that the column with sub-rounded particles had a narrower bulging zone (approximately 200mm) over its length than the column with angular particles (250-400mm), and the occurrence of a shear band influenced the performance of the model stone column in terms of its porosity and intrusion of fines.

- The porosity profiles obtained from the analysed CT-images at the end of the test were used to infer compression-induced densification, expansion (dilation) and shearing. The stone column underwent extreme compression and then expansion, during which time the porosity increased to a maximum value in the zone of extreme lateral deformation (bulging). The magnitude of maximum porosity is also influenced by particle morphology. The CT-Scan images indicated that the intrusion of fines is significant for some of the columns tested, particularly those with sub-rounded particulates. The extent to which fines intrude due to the surrounding soft clay being remoulded was greater in the zone where extreme lateral deformation occurred (approximately between 200-400mm depth below the column). However, the intrusion of fines also increased as particle interlocking and the coefficient of friction decreased. In some parts of the column where sub-rounded particles are present, the intrusion of fines (clay) exceeded the increase in the column radius by up to 25%, which then reduced the uncontaminated zone inside the column.

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1

2 Table 1. Descriptions of Tests and Stone Column Material Properties

Test	Particle Set for Column Materials	Additional Miniature Sensors	Friction Angle of Column Materials in degrees		Roundness Coefficient (Krumbein and Sloss, 1963)	Average “n” parameter for grading
			Peak (ϕ'_p)	Constant Volume (ϕ'_{cv})		
T3A and T3B	PSDT3/M1 (less-uniform and angular)	N/A	57	38	0.25	0.32
T4		Yes				
T4A	PSDT4/M1 (more-uniform and angular)	N/A	54	37	0.25	1.05
T4		Yes				
T5A	PSDT4/M2 (more-uniform and rounded)	N/A	49	31	0.7	1.05
T5		Yes				

3

4 Table 2. Properties of Soft Clay and Stone Column Particles

Property	Value
Kaolin Clay	
Plastic Limit of	27.4 %
Liquid Limit of Kaolin Clay	55.8 %
Undrained Shear Strength at the end of one-dimensional consolidation	15.5 kPa
Compression Index (C_c) of Clay	0.365
Re-compression Index (C_r) of Clay	0.053
Stone Column	
Specific Gravity of Material M1	2.76
Specific Gravity of Material M2	2.79
Dry unit weight of material M1 and PSDT3 (Tests T3A, T3B and T3)	17.2 kN/m ³
Dry unit weight of material M1 and PSDT4 (Tests T4A and T4)	16.9 kN/m ³
Dry unit weight of material M2 and PSDT4 (Tests T5A)	19.5 kN/m ³

5

M1 - Angular

M2 – Sub-rounded and Rounded

(a)

(b)

Figure 1 – (a) Representatives of angular particles (M1) and sub-rounded and rounded particles (M2), and (b) Particle Size Distribution of Column Material

Figure 2: (a)-(c) Shear stress-strain curves of PSDT3 and PSDT4 for angular and rounded materials; (d) Shear strength of stone column materials (Direct Shear Tests)

Fig.3. Schematic Illustration of large-scale triaxial apparatus

(a)

(b)

Fig.4. (a) Triaxial specimen extrusion using thin-walled PVC tube with tapered end, and (b)
CT-scanning process

Note: Void area ($A_{p,void}$) = Total Area ($A_{p,total}$) – Particle Area ($A_{p,particle}$)

Figure 5 – Illustration of Image Processing (a) Histogram showing thresholding process (b) Image for the calculation of particle area, and (c) Image for the calculation of total area

Fig. 6 (a) Stress-strain and (b) Pore Water Pressure responses for tests T3A, T3B, T4A, T5A and ST1

Note: 35 to 50-mm part near the base of each sample is not shown as it was trimmed during sample extraction. All dimensions shown within the images are in millimetres.

Fig. 7 CT-scan images of deformed columns at the end of testing

Fig. 8 Measured lateral strain profiles of deformed model columns

Fig. 9 Stress-strain behaviour of model columns

Fig. 10. (a) Deformed model columns and corresponding CT-images; (b) Changes in Lateral Strain of the Stone Column and Induced Pore Pressure for Test T4

Fig. 11. Cross-sectional views at depths where maximum lateral deformation occurred: (a) Test T3A, (b) Test T4A and (c) Test T5A

Fig. 12. CT-scanned image showing the cross-section of the lower end of the shear band in
Test T4A

Fig.13. End-of-test Porosity profiles for Model Columns

Fig. 14. Profile of Fines Penetration Ratio for Model Columns