Experimental and Numerical Analyses of the Behavior of Rammed Stone Columns Installed in a South African Soft Soil

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Abstract—This study investigated the behaviour of single rammed stone columns. Prior to this assessment, the load – settlement and bulging responses of the columns were evaluated by conducting laboratory tests using a specially designed bench scale tank. The latter was a mild steel cylindrical tank which represented unit cells reinforced with single stone columns. The testing technique enabled the evaluation of the influence of variation of column diameter on stone column deformation responses and also provided data that was compared with the numerical results. In the numerical analysis, Mohr-Coulomb and modified Drucker-Prager models were used in the idealization of the behaviour of the column and soil materials respectively. Results revealed that the bigger the column diameter, the higher the loading capacity of the columns. Increasing the column diameter D by 1.5, 2 and 3 times its initial size generally led to improved load carrying capacity by approximately 2, 4 and 10 times the initial strength respectively. The radial expansion of the columns was prominent in their upper parts with the highest value being experienced at a depth of about 0.5D from the ground surface.

Index Terms—South African, soft soil, stone columns, unit cell, load carrying capacity.

I. INTRODUCTION

According to a recent report on the “State of African Cities” published by the UN Habitat [20], urbanization is occurring at a furious rate in most African cities. This is mainly due to human migration from rural areas to urban cities which is rapidly happening on the continent; and the development booms linked to current foreign investment and increased trade. Consequently, areas exhibiting weak soil strata such as swamps which were previously considered unsuitable for construction are being developed to meet the high demand in housing and service facilities. In addition, areas along the coast which usually consist of soft marine clay are also under development since they present environments with high cultural interest and rich biodiversity which greatly attract tourists. Although pile foundations may be deployed in those areas to meet the design requirements for stability, integrity and serviceability of the infrastructures, they are costly. Hence other ground improvement techniques may be preferable because they are cheaper. Amongst the wide range of ground improvement techniques are stone columns, which are often considered as cost-effective and environmentally friendly. Stone columns are installed in pre-bored openings in marginal soils such as cohesive soils and silty sands [1],[17]; and consist of coarse materials such as gravels or crushed aggregates with better properties. This technique was apparently first implemented in France in 1830[3]. Since then, it has been extensively used in numerous construction projects realized in marginal areas throughout the world especially in America, Asia and Europe for increasing the bearing capacity of marginal soil and reducing total and differential settlements. Stone columns are also used to facilitate the consolidation process and to improve the resistance to liquefaction [1], [18], [21]. Given the many advantages associated with the use of stone columns, the performance of single stone columns installed in a South African soft soil was investigated in this study. Using the approach of a unit cell comprised of a single column whose tributary area is surrounded by soft soil, the behaviour of single columns was studied through a numerical analysis. Numerical analysis enables the resolution of complex problems where analytical solutions and experimental measurements are sometimes cumbersome to be established. Prior to the extensive use of numerical models developed with the finite element analysis software, Abacus 6.10, laboratory tests were performed for the calibration and validation of the models.
II. RESEARCH MATERIALS AND METHODOLOGY

A. Material properties

1) Durbanville clay
Durbanville clay, a fine reddish-brown clay (Fig. 1a), was used as the base material. It was obtained from the Durbanville area of the Western Cape Province in South Africa. It was classified as lean clay (CL) with liquid and plastic limits of 43% and 26% respectively. Its properties determined during the laboratory tests are summarized in Table I.

2) Crushed aggregates
The crushed aggregates were grey and angular in shape (Fig. 1b). They were sourced from the Peninsula quarry in the Western Cape Province in South Africa. Particle distribution tests revealed that their grain sizes range from 1.18 to 9.50 mm with a mean grain size of 5.5 mm. Their coefficients of uniformity and conformity were 1.46 and 1.04 respectively. Based on their particle distribution and uniformity coefficients, the crushed aggregates were classified as poorly graded gravel (GP). Properties of the crushed aggregates determined during the laboratory tests are summarized in Table I. Its shear strength was determined by carrying out a large shear box test on specimens compacted to an average density of 1.6 Mg/m³ which would be achieved during installation of the columns.

![Durbanville clay](image1.jpg)  ![Crushed aggregates](image2.jpg)

(a) Durbanville clay  (b) Crushed aggregates

Fig. 1: Research materials

<table>
<thead>
<tr>
<th>Properties</th>
<th>Unit</th>
<th>Durbanville Clay</th>
<th>Crushed Aggregates</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity</td>
<td>---</td>
<td>2.70</td>
<td>2.68</td>
</tr>
<tr>
<td>Liquid limit</td>
<td>%</td>
<td>43</td>
<td>N/A</td>
</tr>
<tr>
<td>Plastic limit</td>
<td>%</td>
<td>26</td>
<td>N/A</td>
</tr>
<tr>
<td>Unified soil classification</td>
<td>---</td>
<td>CL</td>
<td>GP</td>
</tr>
<tr>
<td>Optimum moisture content</td>
<td>%</td>
<td>20.0</td>
<td>N/A</td>
</tr>
<tr>
<td>Maximum dry density</td>
<td>Mg/m³</td>
<td>1.67</td>
<td>N/A</td>
</tr>
<tr>
<td>Loosest density</td>
<td>Mg/m³</td>
<td>N/A</td>
<td>1.43</td>
</tr>
<tr>
<td>Densest density</td>
<td>Mg/m³</td>
<td>N/A</td>
<td>1.79</td>
</tr>
<tr>
<td>Cohesion</td>
<td>kN/m²</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Angle of internal friction</td>
<td>°</td>
<td>28</td>
<td>48</td>
</tr>
</tbody>
</table>

Table I: Properties of the Durbanville clay and crushed aggregates
B. Methodology

1) Overview and schedule

The compression tests were performed using the Zwick 1445, an automated universal tensile and compression testing machine. All clay beds were prepared at a consistency corresponding to the liquid limit water content since soft soils are naturally found at or above their liquid limit [5], [12]. A specially designed bench scale mild steel cylindrical tank 300 mm wide and 550 mm high represented the unit cell containing a clay bed 400 mm high. Single end bearing stone columns with 70 and 100 mm diameters respectively were installed at the center of the clay bed. The length to diameter ratios were greater or equal to 4.0 which is the minimum recommended in order to mobilize the full axial capacity of the stone columns [2]. Reinforcing columns of 70 and 100 mm diameter had a ratio of 5.7 and 4.0 to length respectively. The cylindrical tank diameter of 300 mm was deemed adequate to represent a field scenario where stone columns are installed in a triangular pattern with spacing equal to twice the diameter of the columns, thereby bringing the maximum tributary diameter of the single stone column to 210 mm (2.1D, D: column diameter) which was less than the diameter of the cylindrical tank. Fig.2 and Fig.3 show a grid of the triangular pattern of the installation of stone columns and the configuration of the tests done respectively. In total, six bench scale tests were conducted during the experiments. That is, three unreinforced and reinforced tests respectively. The experiments included one repeated test in each case so as to confirm that the experimental methodology was repeatable. Table provides a summary of the testing schedule.

Table II: Testing schedule

<table>
<thead>
<tr>
<th>Test description</th>
<th>Loading Plate</th>
<th>Test label</th>
<th>Number of tests</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>PT1 = 70</td>
<td>PT2 = 100</td>
<td>DVC-PT1</td>
</tr>
<tr>
<td>Clay bed only</td>
<td>●</td>
<td>●</td>
<td>DVC-PT2</td>
</tr>
<tr>
<td>Clay bed only</td>
<td>●</td>
<td>●</td>
<td>DVC-CA-D1-PT1</td>
</tr>
<tr>
<td>Stone column with D1 = 70</td>
<td>●</td>
<td>●</td>
<td>DVC-CA-D2-PT2</td>
</tr>
<tr>
<td>Stone column with D2= 100</td>
<td>●</td>
<td>●</td>
<td></td>
</tr>
</tbody>
</table>

CA: Crushed Aggregates  D: Diameter  DVC: Durbanville Clay  PT: Plate

Fig. 3 shows a grid of the triangular pattern of the installation of stone columns and the configuration of the tests done respectively. In total, six bench scale tests were conducted during the experiments. That is, three unreinforced and reinforced tests respectively. The experiments included one repeated test in each case so as to confirm that the experimental methodology was repeatable. Table provides a summary of the testing schedule.
Fig. 2: Triangular pattern of the stone column installation in the field

![Triangular pattern of the stone column installation in the field](image)

Fig. 3: Sketches showing the configuration of the tests performed

![Sketches showing the configuration of the tests performed](image)

2) **Preparation of the clay bed**

The clay was initially sufficiently air-dried and the moisture content of the air-dried soil determined. Using a Tyrone Industrial mechanical mixer, 10 kg of air-dried soil were mixed with a known volume of water to produce a blended sample at 43% moisture content. The blended sample was immediately transferred into a sealed box. This process was repeated 6-8 times to ensure that the quantity of blended clay prepared was sufficient for forming the clay bed in the tank. Samples were then taken for moisture content control and the mixture was kept in the sealed box for 24-48 hours to ensure uniform moisture distribution. For each compression test, the inner surface of the cylindrical tank was first smeared with a thin coat of grease in order to reduce friction between the clay and the tank wall. Using a suitable weighing scale, approximately 6.875 kg of the prepared moist clay material were measured and afterwards gently poured into the cylindrical tank. A wooden board was then placed onto the soil specimen and compacted using a 2 kg rammer falling 12 times through a height of 180 mm onto the board in order to form a layer 50 mm thick. This process was repeated 8 times until a base soil 400 mm high was formed in the tank. The soil surface was thereafter leveled using a metal scraper and a spirit level. The average bulk density achieved during preparation of the clay bed was 1.945 Mg/m³. Fig. 4 shows the plan view of the soft clay inside the tank.
3) **Stone column installation**

Single stone columns were centrally installed in the base soil by replacement method. For each installation, a thin open-ended cylinder of desired diameter whose inner and outer surfaces were slightly smeared with grease was used. This was done to ease its penetration and withdrawal thereby minimizing the disturbance caused to surrounding soil during installation. The cylinder was pushed into the clay bed up to the bottom of the tank through a collar fixed to the tank. The clay inside the cylinder was thereafter scooped out using a helical auger whose diameter was slightly less than the cylinder’s. Upon completing the scooping process, the inner surface of the cylinder was neatly cleaned to prevent the column material from sticking to the cylinder surface. The crushed aggregates were thereafter inserted into the cylinder in measured portions; and then manually compacted using a 2 kg rammer falling 12 times through a height of 180 mm in order to achieve a compacted height of 50 mm at a time. The cylinder was then lifted by 45 mm at a time ensuring a small penetration of the cylinder below the top surface of the crushed aggregate layer formed. This process was repeated 8 times until the full length of column was formed, after which the soil’s top surface was leveled. The average density at which the stone column was installed was 1.6 Mg/m$^3$. Fig. 5 shows the stone column formed in the soft clay.

4) **Procedure for the load test**

Fig.6a shows the schematic of the specimen containing a stone column completely set up for the compression test. Before applying the compressive load onto the test specimen, necessary checks were done to ensure the centering of the tank. The loading rate was set to 0.0625 mm/min. This displacement rate was adopted by other researchers such as Ambilyand Ghandi [1] while conducting similar studies, and was mainly used to simulate the medium to long term behaviour of the composites formed. Additionally, it avoided the development of excess pore water pressure during the experiments [1],[11]. With this loading rate, it took over 10 hours for the test to be completed. At the end of each test, the failure that occurred within the compressed specimen was carefully examined as the tank was emptied. Another sample was then prepared and tested following the procedure described in sections 2 and 3 of the methodology. For the unreinforced soil test, no stone column was installed (Fig.6b).
III. NUMERICAL MODEL DEVELOPMENT

A. Problem geometry, boundary conditions and discretization

The problem of an axially loaded stone column surrounded with tributary soil is an axisymmetric one, if the materials are homogenous. Axisymmetric finite element analysis was therefore carried out for the assessment of the load – settlement and bulging behaviour of the single stone columns. Abacus 6.10 software, one of the most used software packages in geotechnical engineering, was employed for finite element analysis of the problem under investigation. Owing to the symmetry along the centre of the column or clay bed in case of reinforced and unreinforced soil specimens respectively, only half portion of the vertical cross-sectional shown in Fig.3 was modeled during the analyses. Based on the geometrical characteristics of the clay bed and the installed columns, the basic axisymmetric idealizations of the laboratory models with their boundary conditions are depicted in Fig.7. Along the vertical edge of the model, only vertical displacement was permitted. Both horizontal and vertical displacements were not allowed along the bottom edge of the model. For the analyses, 8-node axisymmetric quadrilateral, biquadratic displacement and bilinear pore pressure, reduced integration elements (CAX8RP) were used for meshing since they are suitable for drained analysis and have a higher convergence rate compared to triangle elements. Besides, they are capable of capturing stress concentrations more effectively [7]. An example of the domain meshes comprising 600 elements and 1992 nodes generated for the analyses is shown in Fig.8.

(a) Reinforced soil   (b) Unreinforced soil

Fig.6: Specimens completely set up for the compression test

Unreinforced

No horizontal movement allowed

35/50 mm

482 mm

Reinforced

No horizontal movement allowed

35/50 mm

No horizontal movement allowed
B. Constitutive models

1) Mohr-Coulomb
The Mohr-Coulomb model is widely used in finite element analysis of geotechnical applications due to its simplicity and sufficient accuracy. The Mohr-Coulomb model is generally suitable for granular materials therefore it was used in the idealization of the behaviour of the column material. An elasto-perfectly plastic behaviour with the Mohr-Coulomb...
failure criterion was adopted for the material response under loading conditions. In terms of the principal stresses ($\sigma_1$ and $\sigma_2$), the failure envelope of the Mohr-Coulomb model represented in Fig.9 is defined using (1).

$$ (\sigma_1 - \sigma_3) + (\sigma_1 + \sigma_3) \sin \phi - 2\sigma \cos \phi = 0 \quad (1) $$

Where $c$ and $\phi$ are the cohesion and the angle of internal friction of the material respectively.

In this study, the Mohr-Coulomb model parameters considered for the crushed aggregates along with their justification are summarized in Table III.

![Mohr-Coulomb failure criterion](image)

**Table III: Crushed aggregates parameters**

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Value</th>
<th>Source</th>
<th>Justification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (kg/m$^3$)</td>
<td>1600</td>
<td>Lab</td>
<td>Average bulk density achieved during testing</td>
</tr>
<tr>
<td>Young modulus (kPa)</td>
<td>2500</td>
<td>Lit</td>
<td>References [14] and [15] have used identical value in similar studies.</td>
</tr>
<tr>
<td>Poisson ratio</td>
<td>0.3</td>
<td>Lit</td>
<td>References [1] and [14] have used identical value and it also fell within the range proposed by [4].</td>
</tr>
<tr>
<td>Cohesion (kPa)</td>
<td>0.1</td>
<td>Lit</td>
<td>This small value was set to avoid divergence in the analysis (Abaqus manual recommendations).</td>
</tr>
<tr>
<td>Friction angle (°)</td>
<td>48</td>
<td>Lab</td>
<td>Determined by running the large shear box test according to ASTM D3080.</td>
</tr>
<tr>
<td>Dilatancy angle (°)</td>
<td>16</td>
<td>Lit</td>
<td>Value taken in the range of 1/3 to 2/3$\phi$. Reference[14] considered identical range while conducting similar study.</td>
</tr>
<tr>
<td>Permeability (m/s)</td>
<td>$1.2 \times 10^{-5}$</td>
<td>Lit</td>
<td>Reference [14] has used identical value and it also fell within the range proposed by[4].</td>
</tr>
<tr>
<td>Initial void ratio</td>
<td>1.2</td>
<td>Lit</td>
<td>$G_2 = 2.6\varepsilon$ for crushed aggregates (Table I)</td>
</tr>
</tbody>
</table>
2) Modified Drucker-Prager

The non-linear behaviour of the clay material was represented by the modified Drucker-Prager model. This model is extensively described in [11]. Also called, Cap plasticity model, it is appropriate for soil behaviour since it is capable of considering the effect of stress history, stress path, dilatancy and the effect of the intermediate principal stress. As shown in

Fig.10, the yield surface of this constitutive model consists of three parts which are the Drucker-Prager shear failure surface, $F_{\tau}$, the elliptical cap intercepting the mean effective stress axis at a right angle, $F_{\lambda}$, and the smooth transition region between the shear failure and cap surfaces and which is tangential to both surfaces, $F_{\eta}$. In the model, the elastic behaviour of the material is governed by the generalized Hooke’s law. However, the onset plastic behaviour is determined by the Drucker-Prager failure and the cap yield surfaces. For the analyses, the Cap plasticity model parameters adopted for the Durbanville clay along with justification are summarized in Table IV.

![Fig.10: Yield surfaces of the modified Drucker-Prager / Cap plasticity model in the p-t plane [11]](image)

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Value</th>
<th>Source</th>
<th>Justification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (kg/m$^3$)</td>
<td>1945</td>
<td>Lab</td>
<td>Average bulk density achieved during testing.</td>
</tr>
<tr>
<td>Young modulus (kPa)</td>
<td>40</td>
<td>Lit</td>
<td>Although the minimum value of the Young modulus is 2000 kPa according to [4], values as low as 600 kPa, 300 kPa and 60 kPa have been used in studies conducted by [8], [9] and [10] respectively. In this study, the value of 40 kPa was selected to account for the small loads necessary to induce settlements in the base soil.</td>
</tr>
<tr>
<td>Poisson ratio</td>
<td>0.2</td>
<td>Lit</td>
<td>Reference [14] has used identical value and it also fell within the range proposed by [4].</td>
</tr>
<tr>
<td>Permeability (m/s)</td>
<td>7x10$^{-3}$</td>
<td>Lit</td>
<td>Reference [14] has used identical value and it also fell within the range proposed by [4].</td>
</tr>
<tr>
<td>Initial void ratio</td>
<td>1.2</td>
<td>Lit</td>
<td>$G_s = 2.7$ for clay soil (Table I)</td>
</tr>
<tr>
<td>Coefficient of earth pressure at rest</td>
<td>0.53</td>
<td>Lit</td>
<td>$1 - \sin \phi = 1 - \sin 28 \approx 0.53$</td>
</tr>
</tbody>
</table>

Table IV: Durbanville clay parameters
Material cohesion, $\sigma_0$ (kPa) & 1 & Lit & This small value was set to avoid divergence in the analysis (Abaqus manual recommendations).  
Friction angle, $\phi$ & 28 & Lab & Determined during the small shear box test  
Cap eccentricity, $R$ & 0.4 & Lit & Assumed value. However, $R$ varies between 0.0001 and 1000 (Abaqus manual).  
Initial yield surface position & 0 & Lit & Recommended value (Abaqus manual).  
Transition surface radius & 0.01 & Lit & Minimum value was set to avoid divergence in the analysis (Abaqus manual recommendations). However, $R$ varies between 0.01 and 0.05 (Abaqus manual).  
Flow stress ratio & 0.778 & Lit & Recommended value. However, it varies between 0.778 and 1 (Abaqus manual).  
Yield stresses (kPa) & 1, 2, 4, 8, 16, 32, 64, 128, 356, 712 & Lit & These values were derived by performing considerable trial runs to optimize model performance to fit the experimental results. Reference [6] has deployed identical technique to derive some parameters of the Cap Plasticity model during their investigation.  
Volumetric plastic strains corresponding to the yield stresses & 0, 0.001, 0.002, 0.004, 0.008, 0.01, 0.02, 0.04, 0.06, 0.08 & Lit & These values were derived by performing considerable trial runs to optimize model performance to fit the experimental results.  

C. Interface modeling

The master and slave concept was used to define the two surfaces in contact at the interface between the stone column and the surrounding soil. The master surface was chosen on the stone column, and the surrounding soil was chosen as the slave surface since crushed aggregates were stiffer than the clay material. For the description of the frictional behaviour along the interface, the shear behaviour along these two surfaces was governed by the Coulomb frictional law. The Coulomb friction model relates the maximum allowable shear stress ($\tau_{c\theta}$) to the contact pressure ($p_{cont}$) as shown in (2).

$$\tau_{c\theta} = \mu p_{cont}$$

where $\mu$ is the coefficient of friction.

The graphical representation of the Coulomb friction model with no limiting allowable shear stress is shown in Fig 11. When the magnitude of the transmitted frictional shear stress across the interface is less than the critical shear stress, the two surfaces in contact stick together. In such a case, there is no relative movement between the contact nodes. However, there is an occurrence of slippage between the contact nodes at the interface once the critical shear stress is mobilized. In this work, the frictional behaviour at the interface was defined through the penalty interaction module in the software. A default option with no limiting value was set for the critical shear stress and the coefficient of friction ($\mu$) was prescribed as in (3).

$$\mu = \tan \delta$$

where $\delta = \frac{\phi}{2}$ (effective angle of internal friction of the soil).
D. Initial stresses and loading conditions

1) Initial stresses
The initial vertical stress due to the gravity load was taken into account in the analyses. The stress due to column installation was however not considered since this depends greatly on the method of construction. The method deployed in the study for the installation of columns was assumed to induce insignificant disturbance to the surrounding soil. During the experiments, loading of the test specimen was conducted at a slow displacement rate. Hence, the numerical analyses were performed by assuming that sufficient time had lapsed after the application of the load and the stress concentrations as well as the settlement had stabilized based on recommendations by Ambilyand Gandhi [1]. Therefore, a drained analysis was undertaken for the study of the behaviour of the specimen under loading conditions. Thus, the initial stress field consisted of the gravity field stress from the effective self-weight of the soil. The initial vertical effective stresses were then computed using the bulk unit weight of the clay ($\gamma_b$) achieved while preparing the base soil. Equation 4 shows the expression of the effective stress ($\sigma'_z$) at any depth ($z$) in the soil matrix.

$$\sigma'_z = (\gamma_b - \gamma_w)(z - z_0)$$

Where:
- $\gamma_w$ is the unit weight of water.
- $z_0$ is the reference depth in the model.

Alongside these initial vertical effective stresses, are their corresponding horizontal initial effective stresses ($\sigma'_h$). Generally, these are calculated using (5).

$$\sigma'_h = K_0\sigma'_z$$

In the expression, $K_0$ is the coefficient of earth pressure at rest and its value usually ranges between 0.5 and 2 ([13]). For this work, the Jacky’s formula $K_0 = 1 - \sin \phi$ was selected since the installation effects of the stone columns were not taken into account in the study.

2) Loading conditions
The analyses were implemented through initial, secondary and final steps as required by Abaqus 6.10. In the initial step, the boundary conditions defined in section A of numerical model development, were assigned to the numerical model. In the secondary step, the Geostatic step was invoked to ensure equilibrium of the soil matrix with the external applied loads. The initial stress field consisting of the effective self-weight of the soil was mainly prescribed to the model in this step. The initial void ratio of the soil and the horizontal initial stresses were also specified to the model. Lastly, the final step was invoked for the implementation of the drained analysis of the specimen under loading conditions. The loads were applied through a displacement controlled rate technique. A total vertical displacement of 40 mm was assigned to the model at a displacement rate of 0.0625 mm/min. The non-loaded top surface of the clay bed was set permeable by assigning a zero excess pore pressure at that region to allow the dissipation of excess pore pressure in the model. The total time set for this analysis step was 38400 seconds corresponding to the time necessary to induce a settlement of 40 mm at the displacement rate adopted. The maximum settlement was limited to 40 mm (i.e. L/10, where L was the column length) in order to avoid excessive distortion in the model which could have led to the divergence of the solution. This maximum settlement was also beyond the allowable settlement, 25 mm recommended by [19].

IV. RESULTS AND DISCUSSION

A. Repeatability of tests
Repeatability is a crucial part of any experimental procedure as it provides evidence on the consistency of the testing and the reliability of the results obtained. Fig. 12 shows the variation of the settlement against the applied load for the repeatability tests. All the curves illustrated in the figure had identical trends indicating that the
settlement increased with the applied load. In Table V and Table VI, the applied load necessary to induce a specific settlement was recorded for each test, and the relative percentage difference from their mean calculated for assessing the precision of the results. In general, a good correlation was obtained between the repeated tests with most results within ±10% of the mean. This provided assurance of the validity of the results obtained and also indicated that the test procedure was repeatable.

Table V: Repeatability tests conducted for the clay bed only

<table>
<thead>
<tr>
<th>Settlement (mm)</th>
<th>Applied load (N)</th>
<th>Mean (N)</th>
<th>Difference from mean Test A</th>
<th>Difference from mean Test B</th>
<th>Relative percentage difference (%) Test A</th>
<th>Relative percentage difference (%) Test B</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>10.17</td>
<td>9.16</td>
<td>9.66</td>
<td>-0.51</td>
<td>0.51</td>
<td>-5.27</td>
</tr>
<tr>
<td>10</td>
<td>14.19</td>
<td>12.63</td>
<td>13.41</td>
<td>-0.78</td>
<td>0.78</td>
<td>-5.82</td>
</tr>
<tr>
<td>15</td>
<td>17.40</td>
<td>15.58</td>
<td>16.49</td>
<td>-0.91</td>
<td>0.91</td>
<td>-5.52</td>
</tr>
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<td>20</td>
<td>21.42</td>
<td>19.54</td>
<td>20.48</td>
<td>-0.94</td>
<td>0.94</td>
<td>-4.58</td>
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<td>25</td>
<td>24.10</td>
<td>23.03</td>
<td>23.56</td>
<td>-0.54</td>
<td>0.54</td>
<td>-2.27</td>
</tr>
<tr>
<td>30</td>
<td>26.77</td>
<td>24.63</td>
<td>25.70</td>
<td>-1.07</td>
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<td>-4.17</td>
</tr>
<tr>
<td>35</td>
<td>28.65</td>
<td>27.31</td>
<td>27.98</td>
<td>-0.67</td>
<td>0.67</td>
<td>-2.39</td>
</tr>
</tbody>
</table>

Relative percentage difference = 100x(Load for a particular test – mean)/mean

Table VI: Repeatability tests conducted for the stone column of 70 mm diameter

<table>
<thead>
<tr>
<th>Settlement (mm)</th>
<th>Applied load (N)</th>
<th>Mean (N)</th>
<th>Difference for mean Test A</th>
<th>Difference for mean Test B</th>
<th>Relative percentage difference (%) Test A</th>
<th>Relative percentage difference (%) Test B</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>35.02</td>
<td>46.08</td>
<td>40.55</td>
<td>5.53</td>
<td>-5.53</td>
<td>13.65</td>
</tr>
<tr>
<td>10</td>
<td>51.05</td>
<td>67.55</td>
<td>59.30</td>
<td>8.25</td>
<td>-8.25</td>
<td>13.90</td>
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<td>15</td>
<td>71.16</td>
<td>73.64</td>
<td>72.40</td>
<td>1.24</td>
<td>-1.24</td>
<td>1.72</td>
</tr>
<tr>
<td>20</td>
<td>79.74</td>
<td>81.33</td>
<td>80.54</td>
<td>0.79</td>
<td>-0.79</td>
<td>0.98</td>
</tr>
<tr>
<td>25</td>
<td>91.72</td>
<td>85.17</td>
<td>88.44</td>
<td>-3.28</td>
<td>3.28</td>
<td>-3.70</td>
</tr>
<tr>
<td>30</td>
<td>108.43</td>
<td>91.49</td>
<td>99.96</td>
<td>-8.47</td>
<td>8.47</td>
<td>-8.47</td>
</tr>
<tr>
<td>35</td>
<td>106.18</td>
<td>90.14</td>
<td>98.16</td>
<td>-8.02</td>
<td>8.02</td>
<td>-8.17</td>
</tr>
</tbody>
</table>

Relative percentage difference = 100x(Load for a particular test – mean)/mean

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B. Comparison between experimental and numerical results

1) Load – Settlement responses
The experimental load – settlement results of compression of the base soil and the reinforced soil (Exp_U and Exp_R respectively) are plotted along with the numerical results (Num_U and Num_R for the base soil and reinforced soil respectively) in Fig. 13. The settlement increased gradually with increase in the applied load for all graphs. It can be observed that the larger the loading area on the base soil, the greater its load carrying capacity. Similarly, for the composite, larger columns exhibited higher support capacity. Overall, the numerical analysis produced results that correlated to a good extent with the experimental results since each numerical curve and its corresponding experimental curve were well close to each other. In Table VII, the summary of the load carrying capabilities at specific depths for both experimental and numerical analyses is provided and the absolute values of the relative difference from their mean (Abs. RDM) were computed for each case. It can be noticed that few values of Abs. RDM were between 10% and 20%. Otherwise, they were less than 10%. This showed the closeness of the numerical and experimental results, thereby the good correlation between them.

Table VII: Comparison of load carrying capacity at specific depths

<table>
<thead>
<tr>
<th>Depth (mm)</th>
<th>Load carrying capacity of the base soil (N)</th>
<th>Load carrying capacity of the stone column (N)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>PT = 70 mm</td>
<td>PT = 100 mm</td>
</tr>
<tr>
<td></td>
<td>Exp</td>
<td>Num</td>
</tr>
<tr>
<td>5</td>
<td>10.17</td>
<td>14.66</td>
</tr>
<tr>
<td>10</td>
<td>14.19</td>
<td>19.18</td>
</tr>
</tbody>
</table>
2) Bulging responses
At the completion of each experimental test, the failed stone column was carefully examined. By removing layers of the soil material slowly around the column and taking measurements of the deformed diameter of the column and linking them to the corresponding depths, it was possible to establish an approximate shape of the failed stone column which was initially 100 mm in diameter. While Fig. 14 shows photographs of the diameter of the failed column at a depth of 90 mm below the initial soil surface, the approximate shape of the deformed column was established in Fig. 15 based on the measurements taken. As shown in Fig. 15, the deformation was more prominent in the upper region of the column over a length of about 2D. The maximum radial expansion of the column was observed at 50 mm (0.5D) approximately below the top surface of the failed column and the enlarged diameter was 130 mm approximately.

Fig. 14: Experimental assessment of the bulging of the end bearing column of 100 mm diameter

The bulging of the columns was also examined through numerical analysis. Under the same vertical displacement constraint of 40 mm, both columns of 70 mm and 100 mm diameter respectively deformed radially in the numerical model as depicted in Fig. 16. The bulging effect was more pronounced in the upper third of the columns. The detailed values of the horizontal displacement at the outer face of the columns - the column bulging, are plotted against the ratio of the length to diameter of each column in Fig. 17. In the figure, it was evident that column bulging was greater near the column tops and reduced with depth. Both columns experienced their maximum bulging at a depth of about 0.5D. It was 12.34 mm for the column of 100 mm diameter, thereby bringing the maximum enlarged diameter to approximately 125 mm which compared well with the experimental observations. The column lengths mainly affected by bulging were 3D and 2.5D in the case of the 70 mm and 100 mm column diameter respectively. These results were in line with the findings of earlier observations made by [1] and [14].

Abs. RDM: Absolute Relative Difference from Mean
Abs. RDM = absolute (100x(Exp-Mean)/Mean) = absolute (100x(Num-Mean)/Mean)
Mean = (Exp+Num)/2
Using the numerical model successfully developed, a parametric study was conducted in order to provide more insight into the behaviour of singular stone columns. The influence of variation of the column diameter, column length and friction angle of the column material on deformation responses was carefully examined under specific loading conditions.

1) Influence of the increase in column diameter

Fig.18 shows the horizontal and vertical cross-sectional areas of the loading configuration with the varying column diameters considered in this assessment. The column diameter varied between 50 and 150 mm within the unit cell, and the entire end bearing single columns were 400 mm long. A total vertical displacement constraint of 30 mm was assigned to all models using the prescribed displacement controlled rate technique. The material properties adopted were those summarized in Table III and Table IV. The geometrical models with their boundary conditions are depicted in...
Fig. 7 with the column diameter varying accordingly.

Fig. 18: Variation of the diameter D within the unit cell

- **Load – Settlement responses**

For all stone columns, the settlement increased gradually with the increase in the applied load as shown in Fig. 19. However, the rate of increase in settlement decreased with enlargement of the column size. This shows that larger columns can withstand higher compressive loads in marginal soil. In Table VIII, the load carrying capacities (LCCs) at various depths are summarized for all stone columns. The ratio of the LCC for column diameters of 75 mm, 100 mm and 150 mm to that of the 50 mm column diameter respectively is graphically shown in Fig. 20. It can be inferred that when the diameter of the stone column in the base soil increases by 1.5, 2 and 3 times of its initial diameter, its load carrying capacity is approximately 2, 4 and 10 times higher than the initial strength respectively.
Table VIII: Load carrying capacity of the stone columns with varying diameter

<table>
<thead>
<tr>
<th>Depth (mm)</th>
<th>Load carrying capacity (N)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>D = 50</td>
</tr>
<tr>
<td>5</td>
<td>20.63</td>
</tr>
<tr>
<td>10</td>
<td>30.41</td>
</tr>
<tr>
<td>15</td>
<td>36.95</td>
</tr>
<tr>
<td>20</td>
<td>43.53</td>
</tr>
<tr>
<td>25</td>
<td>48.94</td>
</tr>
<tr>
<td>30</td>
<td>54.39</td>
</tr>
</tbody>
</table>

Fig. 19: Load – Settlement response for various diameters within the unit cell

Fig. 20: Load carrying capacity ratio

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Bulging of the columns

The variation of the radial expansion of all the columns is plotted in Fig. 21. Generally, column bulging was more pronounced in the upper third of the column with the greatest deformation being experienced closer to the ground surface. The maximum bulging was 4.68 mm, 7.74 mm, 8.19 mm and 14.94 mm for stone columns of 50 mm, 75 mm, 100 mm and 150 mm respectively. The ratio of radial expansion for column diameters of 75 mm, 100 mm and 150 mm to that of the 50 mm column diameter was 1.65, 1.75 and 3.19 respectively. Thus, increasing the diameter of the column by 1.5, 2 and 3 times of its initial diameter in the base soil resulted in an increase in column bulging in more or less the same manner approximately.

2) Influence of the variation of the angle of internal friction

- **Load – Settlement responses**
  The effect of variation of the friction angle of the column material was assessed for a stone column diameter of 100 mm. As shown in Fig. 22, all composites exhibited the same trend in load – settlement response. The settlement increased continuously with the increase in applied load. However, stone columns with greater friction angle possess higher load carrying capacity than the ones in the looser state (i.e. column material with smaller friction angle). This may be due to the high interlocking of the column material arising from the denser state.

- **Bulging of the columns**
  The variation of the radial expansion of the columns is graphically plotted for the various angles of internal friction in Fig. 23. For all stone columns, this radial expansion increased from the ground surface up to a depth of about 0.5D, and then decreased with depth. The bulging effect was significant in the upper half of the columns. It was also evident that the higher the angle of friction of the material, the lesser the bulging of the column in the region mainly affected. This may be due to the high resistance to deformation which the columns in the denser state possess.
3) **Influence of the variation of column length**

To numerically study the influence of the variation of column length, floating and end bearing stone columns were constructed and loaded individually within the unit cell. All constructed columns had the same diameter (100 mm). However, the lengths of columns varied as 200 mm, 300 mm and 400 mm.

Fig. 24 shows the various stone columns and loading configurations considered. All columns were subjected to the same applied pressure of 4 kPa, 8 kPa and 12 kPa and the settlements at the column bases were recorded to assess the punching behaviour of the columns. The radial deformations of the columns were also recorded to assess the bulging behaviour of the columns. In addition, the effect of increasing the applied pressure on column bulging was also examined for all cases.

- **Punching of the columns**

The vertical displacement at the center of the base of the stone columns shown in
Fig. 24, is plotted against the applied pressure in Fig. 25. The end bearing column exhibited no punching behaviour as it rested on a hard stratum. However, the floating columns showed punching behaviour in the marginal soil. It was evident that the higher the applied pressure, the greater the punching effect. It was also noticed that the shorter the floating column, the greater the punching effect in the marginal soil.

![Diagram showing vertical displacement at the base of the column for different applied pressures]

- **Bulging of the columns**

The variations of radial expansion of the three stone columns shown in Fig. 24, are displayed in Fig. 26. In general, the radial deformation of the columns increased with depth, reaching its maximum value at a depth of approximately 0.5D and thereafter decreased with depth. The bulging effect was mainly observed up to an approximate depth of 2D. Beyond this, the horizontal displacement became less significant as it can be noticed in Fig. 26 (b) and (c). It was also evident that the horizontal displacement increased with applied pressure for all cases since the greater the applied pressure, the bigger the column bulging. All these findings confirm the observations made in earlier studies reporting that floating columns fail by punching and bulging whereas end bearing columns are mainly prone to bulging failure.
Fig. 26: Variation of the horizontal displacement for each loading configuration under different applied pressures

V. CONCLUSIONS AND RECOMMENDATIONS
This study investigated the behaviour of single rammed stone columns installed in a South African soft soil. Prior to the numerical investigation, the research materials were characterized; and the load – settlement and bulging responses of the single columns evaluated by conducting experiments using a specially designed bench.
scale tank. The testing technique enabled the evaluation of the influence of the stone column diameter on the deformation responses and also provided data which correlated to a good extent with the numerical solution. The findings of this study revealed the following:

- The load carrying capacities of the columns were always greater than the base material’s, indicating that stone columns could be used to improve the bearing capacity of soft soils.
- When varying the diameter of the column in the base soil by 1.5, 2 and 3 times of its initial diameter, it was found that its load carrying capacity increased by approximately 2, 4 and 10 times of the initial strength. This suggested that larger columns could withstand higher load while exhibiting smaller settlement.
- Bulging constituted one of the main failure modes of the columns. The radial expansion of the columns was more prominent over a length of about 2D to 3D from the initial surface. The maximum radial expansion of the stone columns was obtained at a depth of about 0.5D from the top of the columns.
- When varying the length of the column, it was found that the shorter the column, the greater its punching behaviour.

Based on the results obtained, it is recommended that future work should explore the use of an encasement material such as geosynthetic to minimize the excessive bulging observed in this study. A field study could also be conducted in order to provide full scale evidence of the improvement achieved with the installation of rammed stone columns in typical South African soft soils.

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