Performance of Field and Numerical Back-Analysis of Floating Stone Columns in Soft Clay Considering the Influence of Dilatancy

D E L Ong¹, Y S Sim² and C F Leung³

Abstract: Ground improvement via stone column has gained popularity around the globe as an effective ground improvement technique to improve the load bearing capacity of soft soils. End-bearing stone columns are commonly used but occasionally floating stone columns are adopted as it has been proven to achieve the necessary load carry capacity and serviceability requirements. However, stone column parameters are often estimated without much basis for simulation and design purpose due to lack of practical research and real-life case studies in this area. Hence, in order to develop fundamental understanding on the load-settlement responses of the floating stone columns in soft clays, detailed back-analysis was performed in this paper. Throughout the back-analysis, all the numerical models were validated by comparing the analysis results with the field measurements. This study highlights (i) the existence of some critical interpreted parameters, namely soil cohesion, c' , friction angle, ϕ' and dilation angle, ψ' as well as (ii) soil-structure interaction mechanism that influences the load-settlement responses of the floating stone columns. The successful characterization of the stone column materials and the modeling technique of stone columns in finite element analyses have shown great potential in assessing the real-life performance of floating stone columns in soft clays.

Keywords: Ground improvement; floating stone column; dilation angle; settlement

¹Senior Lecturer, School of Engineering and Built Environment, Griffith Univ., 170 Kessels Rd., Nathan, QLD 4111, Australia (corresponding author). Email: d.ong@griffith.edu.au

²Geotechnical Engineer, Keller (M) Sdn Bhd, B5-10, Block B, Plaza Dwtasik, Bandar Sri Permaisuri, Off Jalan Tasik Permaisuri 1, Kuala Lumpur 56000, Malaysia; formerly, Research Scholar, Research Centre for Sustainable Technologies, Swinburne Univ. of Technology Sarawak Campus, Kuching, Sarawak 93350, Malaysia. Email: yusheng.sim@keller.com.my

³Professor, National University of Singapore, 21 Lower Kent Ridge Road, Singapore 119077, E-mail: ceelcf@nus.edu.sg

INTRODUCTION

Installation of stone column via vibro replacement technique is an effective ground improvement method to speed up soft soil consolidation process by shortening the drainage path thus, reducing settlement of the soft soils. This form of ground improvement is often adopted for structures that can tolerate acceptable minor settlement, for instance, embankment, storage tanks and light buildings (Khabbazian et al. 2010). Stone columns are commonly designed to rest on hard stratum but floating stone columns have been widely used in a handful of recent case studies due to its cheaper construction cost and sometimes limitation of machinery in the installation process (Ng and Tan 2014).

The performance of floating stone columns has been studied using numerical method, field study and laboratory experiments. Many researchers had conducted laboratory experiments (Afshar and Ghazavi 2014; Cimentada and Costa 2008; Shivashankar et al. 2011); field studies (Kirsch 2006; Shao and Gularte 2007); numerical studies (Choobbasti et al. 2011; Mohanty and Samanta 2015; Six et al. 2012) and theoretical studies (Balaam and Booker 1981; Barksdale and Bachus 1983) to understand the behavior and interaction of stone columns. Black et al. (2007) highlighted that the performance of stone columns would be affected by several factors namely; the installation process, soil engineering properties, installation geometric layout and the area replacement ratio. In the direct shear test data of Herle et al. (2008), the densely compacted granular specimens consisted high friction angle, ϕ' beyond the value of 50° in most cases, which often render conventional design values of $\phi' = 42^\circ$ too conservative.

Kirsch (2008) conducted several full-scale load tests on the instrumented group of floating stone columns loaded by a square platform where the columns were installed to 9 m length with diameter of 0.8 m floating within the underlying soft soils. In order to examine the real

behavior of the stone columns, finite element simulation was used in conjunction with advanced constitutive models to examine the settlement performance of the composite foundation. The numerical model used was successfully benchmarked against the field settlements and good agreement has been obtained, which indicated the effectiveness of this approach. Many researchers had investigated the behavior of stone columns in different geometric layout and stiffness ratios. However, the effect of shear strength parameters (c' and ϕ') of the column materials with respect to its settlement performance has not been widely investigated.

This paper investigates the influence of shear strength and dilation parameters on the load-settlement performance of the floating stone columns in soft alluvial soils at working load levels. Well-calibrated finite element models with the consideration of consolidation analysis were employed through the use of three-dimensional (3D) commercially available geotechnical finite element software, Plaxis 3D.

FIELD CASE STUDIES

Ground improvement via floating vibro stone columns was adopted to support crane loads up to 500 kPa for an offshore rig fabrication yard located at Vung Tau, Vietnam. This innovative form of ground improvement method was chosen due to the presence of average 12 m thick soft compressible soil layers with $N \leq 4$, followed by about 33 m of stiff clay with $N < 30$, before dense sand with $N \geq 50$ was encountered. If the concept of floating columns was not adopted, vibro stone column technique could not be used at all since SPT $N \geq 50$ was only found at depth of about 55 m, which was way deeper than the installation limit of about 35 m typically. In order to achieve a levelled and stable load tray working platform after the installation of the floating stone columns, the original ground was excavated so that a layer of 0.5 m thick well-compacted granular platform could be laid above the reinforced ground. This

layer was properly compacted to maximum dry density (MDD) of 90%. Subsequently, two layers of bi-directional geogrid of tensile strength 100 kN/m each were placed on top of the compacted gravel bed before the remaining 0.25 m thick compacted gravel was placed. These two components were essential in spreading the loads uniformly throughout the load transfer platform before the loads were finally transferred to the soft soils.

Soil profile

A total of three case studies will be discussed in this paper where the first case study T1 was located near to borehole BH 4 and the other two case studies T2 and T3 were adjacent to borehole BH 2. The soil investigation outcome showed that the subsurface consisted granular fill followed by a layer of very soft organic clay deposits up to 12 m. The SPT N value of the subsequent soil increased gradually with depth, indicating an increasing soil strength profile. The physical soil properties can be found in Figs. 1 and 2, while the underlying interpreted soil properties are summarized in Tables 1 and 2 for boreholes BH 4 and 2, respectively.

Field load tests

Static load tests were carried out on a strip of rigid concrete cap measuring 1.5 m by 11.5 m over the 5, 9 and 5 numbers of floating stone columns in case studies T1, T2 and T3, respectively. Approximately 1,100 tonnes of vertical load or an anticipated maximum uniform pressure totalling 600 kPa was applied via six hydraulic jacks on the concrete cap for the respective cases via a kentledge system.

The applied load was increased at 100 kPa increment every 60 minutes interval until it reached the maximum test load of 600 kPa. Once the maximum load test had been achieved, it was then maintained for at least an hour. However, the column would be considered fail if settlement exceeded 100 mm. In order to measure the settlement of the floating stone columns, six dial gauges with 0.01 mm accuracy were installed on the concrete cap. The location of the

load tests is shown in Fig. 3 whereas Figs. 4 and 5 show the typical arrangement of the kentledge load test setup at site.

LABORATORY TESTING

Petrographic test

Thin-section petrography was conducted on the stone column material to determine its mineral contents. The tested specimen was classified as an igneous rock (microtonalite) and mainly comprised fine quartz crystals and alkali feldspar, with presence of hornblende crystals, which were mostly prismatic as shown in Fig. 6.

Particles size distribution

Particle size distribution test was conducted on the scalped microtonalite based on the testing standard ASTM D422-63 (ASTM 2002). Scalping is defined as the removal of particle sizes beyond the allowable particle size range for the direct shear test specimen. The stone column materials collected on-site were crushed using an unconfined compressive strength (UCS) machine so as to obtain smaller particles (2.36 mm to 75 μ m) which would be used for direct shear test. The crushed sample was then scalped by means of sieving to obtain only the sand-sized particles and the oversize particles were removed from the sample (Bagherzadeh-Khalkhali and Mirghasemi 2009). Fig. 7 shows the particle size distribution for the original sample sizes and their scalped sizes. The results indicated that the original material comprised coarse-grained gravels while the scalped material consisted sand-sized grains. Based on the Australian soil classification system (Standards Australia. 1993), the original and scalped material was classified as well-graded gravel (GW). The sieving results of the scalped stone column material are tabulated in Table 3.

Direct shear testing

In order to characterize the material properties, laboratory direct shear tests were conducted on the scalped microtonalite at different normal stresses to determine the strength parameters

of the test specimens. The minimum particle size allowed for a shear box is such that the minimum specimen width should not be less than 10 times the maximum particle size diameter, d_{max} and the minimum specimen height should not be less than $6d_{max}$ (ASTM 2011). The crushed microtonalite was scalped to obtain a distribution of particle sizes passing through 2.36 mm and retained by 75 μ m to satisfy the direct shear test requirements in accordance with ASTM D3080-03 (ASTM 2003), based on the dimensions of the shear box. Scalping method has been previously used by Choo and Ong (2015) during shearing of reconstituted tunnelling rock spoils. Head (1992) emphasized that the maximum size of particles should not exceed 1/10th of the shear box height. The specimen dimension for the direct shear test was of 63.4 mm diameter and 31.5 mm height. The scalped specimens were then prepared within the shear box into three layers of similar thickness, where each layer was compacted with 27 blows using a tamping rod (Peerun et al. 2015). The applied normal stresses are shown in Table 3. The shearing rate is defined as

$$d_r = \frac{d_f}{t_f} \quad (1)$$

where d_f is the estimated horizontal displacement rate at failure, 5 mm; t_f is the total estimate elapsed time to failure, 600 s for clean dense sand; and d_r is the displacement rate in mm/s (Choo and Ong 2015) (ASTM 2011). A shearing rate of 0.0017 mm/s was adopted for the direct shear tests with a maximum horizontal shear displacement of 15 mm (Peerun et al. 2016).

Direct Shear Test Results

Figs. 8 (a) and (b) show the shear stress and volumetric plots for the scalped microtonalite, respectively. At the start of the test, the shear stress increased until a peak shear stress was achieved. Further on during shearing, a reduction in shear stress was observed until residual state was achieved with constant shear stresses. Fig 8 (b) shows an initial compression of the specimen at the start of the test, followed by dilation until almost no vertical displacement was recorded. The initial compression occurred due to the rearrangement of the particles within

existing voids in the specimen. With increasing shear displacement, the particles interlocked among one another, causing other producing the specimen to dilate until a peak shear stress was achieved. Further on, the particles would overcome the interlocking and the shear band would develop into a looser layer by either rolling or rotating and causing a reduction in shear stress (Oda and Kunishi 1974; Fukuoka et al. 2006). At the end of shearing, a steady shear band was produced where the particles slid along the upper and lower halves of the shear interface with almost no vertical displacement. Hence, no significant dilation was observed and hence residual state was achieved (Li and Aydin 2010). With increasing normal stresses, the tested specimens were subjected to relatively larger compression at the initial shearing stage and a relatively reduced dilation with increasing shear displacements.

Fig. 8 (c) shows the strength envelope for the scalped microtonalite. For non-linear strength materials, it was found that by adopting a best-fit Mohr-Coulomb (MC) strength criterion, an over-estimation might occur at lower and higher normal stresses while an under-estimation might occur at intermediate stresses (Choo and Ong 2015). To prevent such inaccuracies, a Power Law function suggested by De Mello (1977) was used to obtain the equivalent non-linear strength characteristics of the tested microtonalite. The Power Law function is defined as

$$\tau = A \cdot (\sigma')^B \quad (2)$$

where A is a dimensionless constant governing the magnitude of the power function and B is a dimensionless constant governing the curvature of the power function (Choo and Ong 2017). Several researchers had adopted this method and it was found that the B value fell within the range of 0.5 to 1.0 for granular materials (Anyaegebunam 2015; Choo and Ong 2015; Serrano et al. 2015). It was also observed that the shear strength of the tested specimen experienced a reduction from peak phase to residual. Choo and Ong (2015) stressed that the reduction of A value and the increase in B value indicate the reduction of curvature of the nonlinear power function. The A and B values tabulated in Table 4 were analyzed based on the test results obtained from the tested specimens using Eq. 2.

In order to back-analyze the field load test results, the equivalent strength parameters obtained from the MC failure criterion is required. Hence, to obtain the internal equivalent friction angle, ϕ' and cohesion, c' , the generalized tangential approach developed by Yang and Yin (2004) was applied to the Power Law function as shown in Fig. 8 (c). This method consists of drawing a tangent to the power law strength envelope at a desired confining pressure to obtain the respective equivalent MC parameters. Tangents were drawn at normal stress of 200 kPa, being the average confining pressure, for peak and residual states and the values are tabulated in Table 5.

From the equivalent MC parameters, a reduction of residual shear strength parameters was observed. This phenomenon is due to strain softening and dilative behavior of the tested specimen, which caused a significant loss in cohesion (Choo and Ong 2015). After achieving the greatest interlocking state among particles, known as the peak state, the shear stress reduced due to the development of the shear band where particles would rearrange themselves by rolling or rotating until a constant shear stress level is achieved. Hence, with a reduced shear stress at residual state, a relatively lower ϕ'_r and c'_r were obtained as opposed to the peak state. In the subsequent analysis of stone column, only the peak equivalent strength values are used.

FIELD PERFORMANCE OF FLOATING STONE COLUMNS

Fig. 9 shows the typical configurations of the load tested stone columns for the three case studies T1, T2 and T3. A layer of 0.5 m thick well-compacted gravel load transfer platform was created at the platform level. Below this load transfer platform, two layers of geogrid were embedded within an additional 250 mm thick bedding. Overall, the 750 mm thick reinforced gravel platform was designed to control surface settlements and to prevent the ground surface from experiencing bearing capacity failure during construction stage. It was expected that the reinforced ground would be able to support the live load of 500 kPa with eventual settlements

not exceeding 100 mm. Ng and Tan (2014) mentioned that settlement would tend to reduce with the increase of granular bed thickness, thus minimizing any serviceability issue.

In case study T1, the stone columns with 1 m diameter and 10 m long (base supported on SPT $N = 1$) were installed in square grid of 2.3 m centre-to-centre (c/c) where shorter columns of 3 m length and 0.6 m diameter were installed at the centre of the square grid. However, stone columns in case study T2 were installed to 10 m length (base supported on SPT $N = 2$) and 1 m diameter with the spacing 1.3 m square grid. Although case study T3 has a similar spacing dimension with case study T1, the length and diameter of the centre columns were increased to 6 m and 1 m, respectively. At 6 m and 10 m lengths, the stone columns were resting on SPT $N = 1$ and $N = 2$, respectively.

Fig. 10 shows the field measured load-settlement responses of case studies T1, T2 and T3. It was apparent that case study T2 showed a stiffer load-settlement response and hence much lesser rebound settlement than T1 and T3 because the former contained a group of 9 floating stone columns being loaded at the rigid concrete cap as opposed to only 5 columns for the latter case studies as shown in Fig. 9. The measured load-settlement response of T3 was more efficient than that of T1 because the installed secondary columns in T3 were twice the length at 6 m, compared to the columns in T1. Besides, T3 also had larger secondary column diameter at 1.0 m.

BACK-ANALYSIS

The parameters used to back-analyze the soil and column properties were important because they would affect the serviceability performance of the floating stone columns. In the back-analysis exercise, the use of the following scenarios and parameters were considered:

- Typical design parameters (ϕ' , $c'=0$ and $\psi'=0$)
- Interpreted parameters (ϕ' , c' and ψ') from this study

The summary of these design and interpreted parameters can be found in Table 6.

3-Dimensional FEM

The advanced hyperbolic hardening soil model developed by Duncan and Chang (1970) was adopted for all soft cohesive soils while Mohr-Coulomb failure criterion associated with the dilation angle was adopted for the remaining soils and column material. The equivalent strength parameters of the column material interpreted from the direct shear results as discussed earlier were adopted in the finite element (FE) analysis. These strength parameters were adopted to back-analyze the performance of the floating stone column group using the geotechnical finite element software Plaxis 3D. The numerical results were validated against the field measurements before performing further analyses using the calibrated numerical model.

In the FE numerical models (see Fig. 11), all granular materials were modelled as drained condition whereas the clay layers were simulated as undrained condition. The boundary conditions used in all the numerical models in this study were laterally restrained along the vertical boundaries while the bottom boundary of the model was fully fixed. The bonding between the granular particles and the native soil is assumed to be nearly perfect along the interface due to consistent compaction of the material during installation of the floating stone columns, where the interface element can be disregarded (Ambily and Gandhi 2004; Tan et al. 2014).

On the other hand, the FE mesh of the models (see Fig. 11) was simulated as continuum elements using 10-node tetrahedral elements. Refinement of mesh was made particularly to the upper portion of the column group and cap as the load intensity was expected to be higher at the surface compared to the lower region of the numerical model. The earth pressure coefficients at rest of all materials were determined using Jacky's equation ($K_0 = 1 - \sin \phi'$) (Elshazly et al. 2008; Tan et al. 2014). The extent of reinforced ground was simulated using a one-quarter model of 30 m depth with the boundary extent of 15 m² to prevent boundary effects. Consolidation analyses were performed to study the settlement performance of stone columns

floating in the weak cohesive soils. However, the installation process of the columns was not considered in the model where the columns were simulated as “wished in place”, which was deemed acceptable (Tan et al. 2014).

A concrete cap having a dimension of 1.5 m by 11.5 m (see Fig. 9) was placed over the selected number of columns of 1 m diameter and 10 m long installed in the square grid such that the columns were floating in soft soils resting on a clay layer ($\text{SPT } N \leq 2$). The average groundwater table was 2.5 m below the ground surface throughout the project site. The applied maximum load of 600 kPa was assumed to be uniformly distributed over the concrete cap. Time rate consolidation analysis was conducted to understand the deformation behavior of the floating stone columns subjected to loading.

Verification of influence of Young's modulus and dilation angle

Castro (2014) suggested that the highly frictional column material whose friction angle, ϕ' and dilation angle, ψ' are generally estimated because these parameters were often difficult to measure in the field. In view of the practical challenges, Fatahi et al. (2012) recommended that the unloading phase was more appropriate and suitable to verify the Young's modulus, E of the stone columns while the loading curve was more appropriate to verify the ψ' of the highly compacted column material. Fatahi et al. (2012) continued to emphasise that the E and ψ' of stone columns were two critical parameters that would adversely affect the accuracy of the numerical results if they were not correctly estimated. Nonetheless, in this paper, these parameters had been calibrated using FE analysis and benchmarked against the field measurements derived from the load test results as shown in Fig. 12, taking after the recommendations of Fatahi et al. (2012).

Fig. 12a illustrates that a reliable estimate of E can be achieved when the unloading gradients at various stages of the FE results become comparable to the corresponding unloading gradients calculated based on the field measured readings. In this case, E of 120,000 kPa, which was used

consistently in the FE model, was found to be a reliable value to use in order to yield a R^2 value of 0.9973. It was recommended by Tan et al. (2014) that the results could be considered reliable if the coefficient of determination, R^2 is 0.9 or greater. Similarly, in Fig. 12b, $\psi' = 20^\circ$ was found to be reliable as the R^2 value was 0.9947, when the settlements derived from the FE model during loading phase showed comparable results to the corresponding field measured values.

The summary of the linear regression of E and ψ' for each case study is tabulated in Table 7. Most of the values show R^2 greater than 0.9, except for the R^2 value of the ψ' for case T2 which is slightly lower than 0.9. This may due to the stiffening effect of the closely spaced floating stone columns installed in the field, which resulted in relatively higher ψ' and lesser settlements on site. Overall, it is quite convincing that the calibrated values ($E=120,000$ kPa and $\psi'=20^\circ$) can be considered reliable for use in this back-analysis process. This observation further verifies that $\psi' = \phi'_p - 30^\circ$ as proposed by Mohanty and Samanta (2015) can be reliably used.

DISCUSSION ON BACK-ANALYSIS RESULTS

Rebound settlements were compared amongst case studies T1, T2 and T3 after they were successfully back-analyzed and validated against the field measured results as shown in Table 8. It is observed that the rebound settlements have been grossly over-predicted when typical design parameters (ϕ' , $c'=0$ and $\psi'=0$) were used. In contrast, if the interpreted parameters (ϕ' , c' and ψ') from this study were used instead, the rebound settlements show much better prediction especially for case study T3, which matched the measured value of 67.8 mm.

Case study T1

The floating stone columns in case study T1 were arranged in 2.3 m square grid where the secondary 0.6 m diameter columns were installed at the centre of the square grid (see Table 9). The granular platform was laid above the reinforced ground to create a stable load transfer

platform that was safe against bearing failure. The applied test load was distributed uniformly to the 5 floating stone columns via the concrete cap (see Fig. 9a). The back-analysis results using interpreted parameters (ϕ' , c' and ψ') from this study showed reasonable agreement with the field measurements especially during loading phase (see Fig. 11a).

In contrast, when the back-analysis was carried out using typical design parameters (ϕ' , $c'=0$ and $\psi'=0$), the load-settlement responses during the loading phase could not trace the observed field results (see Fig. 11a), even though the rebound settlement was somewhat more comparable. The use of lower ϕ' , $c'=0$ and $\psi'=0$ have resulted in a more plastic response than the measured response especially after the load application of 400 kPa.

Case study T2

In this case study, the floating stone columns of 10 m length with 1 m diameter were embedded within the soft soil with SPT N value not more than 2. Unlike case study T1, the floating stone columns in case study T2 were arranged in a closer spacing of 1.3 m square grid (see Table 9). The applied test load was distributed uniformly to the 9 floating stone columns via the concrete cap (see Fig. 9b). It was observed that the field measured load-settlement responses showed some form of stiffening effect due to the closely spaced columns installed and thus resulted in relatively small settlements. The back-analysis results using interpreted parameters (ϕ' , c' and ψ') from this study indicated similar stiffening phenomenon in general when loaded. The FE response showed similar general trend but could not successfully trace the measured load-settlement behavior more accurately (see Fig. 11b).

In contrast, when back-analysis was carried out using typical design parameters (ϕ' , $c'=0$ and $\psi'=0$), the load-settlement responses during both the loading and unloading phases showed grossly over-predicted responses and thus could not trace the observed field results altogether. The use of lower ϕ' , $c'=0$ and $\psi'=0$ have also resulted in a more plastic response than the

measured response especially after the load application of 400 kPa and somewhat portrayed similar responses to those of case study T1.

Case study T3

The 1.0 m diameter primary and secondary floating stone columns in case study T3 were arranged in 2.3 m square grid (see Table 9). Similarly to all the other case studies, the granular load transfer platform was formed to control surface settlements and to increase bearing capacity. The applied test load was distributed uniformly to the 5 floating stone columns via the concrete cap (see Fig. 9c). The back-analysis results using interpreted parameters (ϕ' , c' and ψ') from this study showed good agreement with the field measurements (see Fig. 11c), therefore validating the use of the interpreted column shear strength parameters (ϕ' and c') based on the direct shear box test results as discussed earlier, as well as the use of ψ' .

In contrast, when the back-analysis was carried out using typical design parameters (ϕ' , $c'=0$ and $\psi'=0$), the load-settlement responses during the loading phase could not trace the observed field results (see Fig. 11c). Besides, the rebound settlement was also grossly over-predicted. The use of lower ϕ' , $c'=0$ and $\psi'=0$ had resulted in a more plastic response than the actual field measured response especially after the load application of 400 kPa. The phenomenon of premature plastic response was consistent to the earlier two case studies and hence proven to be detrimental in trying to benchmark against the field measured responses.

Overall assessment

It is to be mentioned that throughout the back-analysis of the three case studies T1, T2 and T3, similar interpreted strength parameters ($\phi'=50^\circ$ and $c'=45$ kPa) from the direct shear test have been used. Besides, dilation angle, $\psi'=20^\circ$ has also been used consistently in the FEM in view that Raju et al. (2004) stated that the installation of stone columns was aided by the vibrations and thrusting action of the vibrator. This technique partially replaces the weaker surrounding soils with granular materials, which upon being vibrated will result in tightly

interlocking of materials with the surrounding soils. Greater interlocking effect of the granular materials will thus result in greater apparent cohesion, c' due to the reorientation of particles during compaction.

From case studies T1, T2 and T3, it was observed that reasonable and realistic load-settlement response curves could be obtained when the interpreted parameters ($\phi'=50^\circ$, $c'=45$ kPa and $\psi'=20^\circ$) from this study were adopted in the FEM back-analyses (see Fig. 11), instead of the typical design parameters (ϕ' , $c'=0$ and $\psi'=0$). Therefore, this important observation lends further credence to the interpreted equivalent strength parameters based on the direct shear testing, whose results were then interpreted based on the Power Law function and the generalized tangential approach developed by Yang and Yin (2004). Besides, the use of dilation angle, ψ' has been found to be important in preventing premature plastic responses in the FE analysis during the loading phase of the floating stone columns, which if not addressed, will subsequently lead to over-prediction of the rebound settlements.

Influence of ϕ' , c' , and ψ'

An extensive parametric study was conducted on the shear strength parameters (c' and ϕ') as well as the dilation angle, ψ' of the stone column material using a validated FE model to back-analyze its load-settlement performance when subjected to loadings.

Fig. 13 illustrates a typical case study T3 whose load-settlement curves were obtained via parametric study using the benchmarked FE analyses performed earlier. The parametric study was conducted on the friction angle, ϕ' and apparent cohesion, c' of the column material ranging from 25° to 50° and 0.1 kPa to 50 kPa, respectively. A total of six different ϕ' and c' values were adopted in the parametric study to investigate their respective impact on the load-settlement performance of the floating stone columns in soft clays.

A total number of 36 FE runs were carried out to study the effect of shear strength (c' and ϕ') and ψ' parameters on the settlement performance of the floating stone columns. Based on the parametric results, the gradual improvement of the apparent cohesion, c' of the column material from 0.1 kPa to 50 kPa will significantly reduce the load-settlement behavior. This is most likely due to the constant compaction experienced by the column materials during the installation process, which resulted in better interlocking and interaction of the particles, thereby, reducing the settlement potential of the floating columns.

The ϕ' and thus the ψ' of the column material are important parameters that can minimize the development of plastic strains during the shearing process, when ϕ' is beyond 30° (Walton et al. 2014). By being able to minimize the development of plastic strains, ψ' is a major parameter that will significantly affect the settlement results in frictional materials (Houlsby 1991). True enough as demonstrated by the load-settlement responses as shown in Figs. 13a - c, plastic strains seem to quickly manifest the behavior at low friction and dilation angles, especially beyond 400 kPa loads. As the magnitudes of the friction and dilation angles increase (see Figs. 13c - f), the manifestation and influence of plastic strains reduce in influence at higher load applications. Another important observation is that when the friction and dilation angles increase, the influence of apparent cohesion reduces, thus easily matching the measured load-settlement responses at most values.

In summary, for the case study T3 shown in Fig. 13f, the back-analyzed parameters that best suit the measured load-settlement behavior are when the stone columns inherit material properties of $\phi'=50^\circ$, $c'=45$ kPa and $\psi'=20^\circ$, as back-analyzed in this paper. In fact, these derived parameters have been consistently used in the other case studies T1 and T2, whose load-settlement responses are generally comparable to their corresponding measured load-settlement responses as evidenced in Fig. 11. Therefore, it has been successfully shown that if the influence of stone column parameters can be better understood and subsequent design

optimization that includes provision of a load transfer platform can be implemented, then floating stone columns instead of end-bearing ones, could be the optimal ground improvement solution in areas manifested with thick, soft clay deposits.

CONCLUSIONS

This research was motivated by the fact that there exists some gaps in the detailed understanding of the load-settlement responses of floating stone columns installed in soft clays ($\text{SPT } N \leq 2$).

This paper presents 3 cases studies where static load tests were carried out on the installed group of stone columns with different configurations. In order to investigate the load-settlement responses of the three case studies and to characterize the critical parameters that affect the load-settlement responses, the following activities had been successfully carried out:

(i) The scalped column materials were subjected to direct shear testing and the equivalent shear strength parameters, c' and ϕ' were interpreted using the established Power Law function and generalized tangential method.

(ii) The value of the dilation angle, ψ' was selected based on the relationship $\psi' = \phi'_p - 30^\circ$, where ϕ'_p is the interpreted friction angle derived from (i) above

(iii) The study then used 3-dimensional (3D) finite element analyses to investigate the load-settlement responses of the loaded groups of floating stone columns based on the established interpreted parameters derived from this study. Throughout the study, all the numerical models were validated by comparing the analysis results with the field measured readings

(iv) Detailed parametric study was performed to develop fundamental understanding of the measured load-settlement responses for stone columns floating in soft clays and the roles of the critical parameters namely c' , ϕ' and ψ'

From the three case studies investigated, it was observed that reasonable and realistic load-settlement response curves could be obtained when the interpreted parameters ($\phi'=50^\circ$, $c'=45$ kPa and $\psi'=20^\circ$) from this study were adopted in the finite element back-analyses, instead of the

typical design parameters (ϕ' , $c'=0$ and $\psi'=0$). Therefore, this important observation lends further credence to the interpreted equivalent strength parameters based on the direct shear testing, whose results were then interpreted based on the Power Law function and the generalized tangential approach. Besides, the use of dilation angle, ψ' has been found to be important in preventing premature plastic responses in the finite element analysis during the loading phase of the floating stone columns, which if not addressed, will subsequently lead to over-prediction of the rebound settlements. Therefore, it has been successfully shown that if the influence of stone column parameters can be better understood and subsequent design optimization that includes provision of a load transfer platform can be implemented, then floating stone columns instead of end-bearing ones, could be the optimal ground improvement solution in areas manifested with thick, soft clay deposits.

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NOTATION

The following symbols are used in this paper:

A = dimensionless constant

B = dimensionless constant

c' = cohesion

C_u = Uniformity coefficient

d_f = estimated horizontal displacement rate at failure

d_{max} = maximum particle size diameter

d_r = shearing rate

E = Young's modulus

ϕ' = friction angle

R^2 = Coefficient of determination

γ = density

σ' = normal stress

τ = shear stress

ψ' = Dilation angle

REFERENCES

- Afshar, J. N., and Ghazavi, M. (2014). "Experimental Studies on Bearing Capacity of Geosynthetic Reinforced Stone Columns." *Arabian Journal for Science and Engineering*, 39(3), 1559-1571.
- Ambily, A. P., and Gandhi, S. R. (2004). "Experimental And Theoretical Evaluation Of Stone Column In Soft Clay." *ICGGE* pp. 201- 206.
- Anyaegbunam, A. (2015). "Nonlinear Power-Type Failure Laws for Geomaterials: Synthesis from Triaxial Data, Properties, and Applications." *International Journal of Geomechanics*, 15(1), 04014036.
- ASTM (2002). "Standard Test Method for Particle-Size Analysis of Soils." *D422-63*, West Conshohocken, PA, United States.
- ASTM (2003). "Standard test method for direct shear test of soils under consolidated drained conditions." *D3080-03*, West Conshohocken, PA, United States.
- ASTM. (2011). "Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions", *D3080M-11*, West Conshohocken, PA.
- Bagherzadeh-Khalkhali, A., and Mirghasemi, A. A. (2009). "Numerical and experimental direct shear tests for coarse-grained soils." *Particuology*, 7(1), 83-91.
- Balaam, N. P., and Booker, J. R. (1981). "Analysis of rigid rafts supported by granular piles " *International Journal for Numerical and Analytical Methods in Geomechanics*, 5(4), 379-403.
- Barksdale, R. D., and Bachus, R. C. (1983). "Design and construction of stone columns." *Rep. No. FHWA/RD 83/026*, , Federal Highway Administration, Washington, DC.
- Black, J. A., Sivakumar, V., Madhav, M. R., and Hamill, G. A. (2007). "Reinforced Stone Columns in Weak Deposits: Laboratory Model Study." *Journal of Geotechnical and Geoenvironmental Engineering*, 133(9), 1154-1161.

- Castro, J. (2014). "Numerical modelling of stone columns beneath a rigid footing." *Computers and Geotechnics*, 60, 77-87.
- Choo, C., and Ong, D. (2015). "Evaluation of Pipe-Jacking Forces Based on Direct Shear Testing of Reconstituted Tunneling Rock Spoils." *Journal of Geotechnical and Geoenvironmental Engineering*, 141(10), 04015044.
- Choo, C. S., and Ong, D. E. L. (2017). "Back-analysis and numerical modelling of pipe-jacking forces in the highly fractured and highly weathered Tuang Formation". Geotechnical Research, Institution of Civil Engineers, UK. doi:http://dx.doi.org/10.1680/jgere.16.00022
- Choobbasti, A. J., Zahmatkesh, A., and Noorzad, R. (2011). "Performance of Stone Columns in Soft Clay: Numerical Evaluation." *Geotechnical and Geological Engineering*, 29(5), 675-684.
- Cimentada, A., and Costa, A. D. (2008). "Laboratory experimental analysis of radial consolidation around a stone column." *Geotechnics of Soft Soils: Focus on Ground Improvement*, 213 -217.
- De Mello, V. F. B. (1977). "Reflections on design decisions of practical significance to embankment dams." *Géotechnique*, 27(3), 281-355.
- Duncan, J. M., and Chang, C. Y. (1970). "Nonlinear analysis of stress strain in soils." *Journal of Soil Mech Found Div ASCE*, 96(SM5), 1629-1653.
- Elshazly, H. A., Hafez, D. H., and Mossaad, M. E. (2008). "Reliability of Conventional Settlement Evaluation for Circular Foundations on Stone Columns." *Geotechnical and Geological Engineering*, 26(3), 323-334.
- Fatahi, B., Basack, S., Premananda, S., and Khabbaz, H. (2012). "Settlement prediction and back analysis of Young's modulus and dilation angle of stone columns." *Australian Journal of Civil Engineering* 10(1), 67-80.
- Fukuoka, H., Sassa, K., Wang, G., and Sasaki, R. (2006). "Observation of shear zone development in ring-shear apparatus with a transparent shear box". *Landslides*, Springer-Verlag, 3(3), 239-251. doi:10.1007/s10346-006-0043-2
- Head, K. H. (1992). "Manual of soil laboratory testing." *Permeability, shear strength and compressibility tests*, Halsted Press, New York.
- Herle, I., Wehr, J., and Arnold, M. (2008). "Soil improvement with vibrated stone columns?influence of pressure level and relative density on friction angle." *Geotechnics of Soft Soils: Focus on Ground Improvement*, Taylor & Francis, 235-240.
- Houlsby, G. T. (1991). "How the Dilatancy of Soils Affects Their Behavior." *Proceedings of the Tenth European Conference on Soil Mechanics and Foundation Engineering*, Invited Theme Lecture, Florence, pp. 1189-1202.
- Khabbazian, M., Meehan, C., and Kaliakin, V. (2010). "Numerical Study of Effect of Encasement on Stone Column Performance." *Proc., GeoFlorida 2010*, pp. 184-193.
- Kirsch, F. (2006). "Vibro Stone Column Installation and its Effect on Ground Improvement " *Proceedings of the International Conference on Numerical Modelling of*

Construction Processes in Geotechnical Engineering for Urban Environment, Taylor and Francis, Bochum, Germany, pp. 115– 124.

Kirsch, F. (2008). "Evaluation of ground improvement by groups of vibro stone columns using field measurements and numerical analysis." *Geotechnics of Soft Soils: Focus on Ground Improvement*, Taylor & Francis, 241-247.

Li, Y. R., and Aydin, A. (2010). "Behavior of rounded granular materials in direct shear: Mechanisms and quantification of fluctuations." *Engineering Geology*, 115(1–2), 96-104.

Mohanty, P., and Samanta, M. (2015). "Experimental and Numerical Studies on Response of the Stone Column in Layered Soil." *International Journal of Geosynthetics and Ground Engineering*, 1(3), 1-14.

Ng, K. S., and Tan, S. A. (2014). "Design and analyses of floating stone columns." *Soils and Foundations*, 54(3), 478-487.

Oda, M., and Kunishi, J. (1974). "*Microscopic deformation mechanism of granular material in simple shear*". Japan Society of Soil Mech Foundation, 14(4), 25-38.

Peerun, I., Ong, D. E. L., and Choo, C. S. (2015). "Behavior of reconstituted sand-sized particles in direct shear tests using PIV technology." *The 15th Asian Regional Conference on Soil Mechanics and Geotechnical Engineering*, Japanese Geotechnical Society Special Publication, Fukuoka, Japan, pp. 354-359.

Peerun, M.I., Ong, D.E.L. and Choo, C.S. (2016), "*Effect of Particle Shapes on Shear Strength during Direct Shear Testing using GeoPIV Technology*", in SH Chan, TA Ooi, WH Ting, SF Chan & DEL Ong (eds.), *Deep Excavation and Ground Improvement – Proceedings of the 19th SEAGC and 2nd AGSSEAC*, 31 May – 3 June 2016, Subang Jaya, Malaysia, ISBN 978-983-40616-4-7

Raju, V. R., Krishna, R. H., and Wegner, R. (2004). "Ground Improvement using Vibro Replacement in ASIA 1994 to 2004 - A 10 Year Review.", Keller (M) Sdn. Bhd., Malaysia, Malaysia.

Serrano, A., Olalla, C., and Jimenez, R. (2015). "Analytical Bearing Capacity of Strip Footings in Weightless Materials with Power-Law Failure Criteria." *International Journal of Geomechanics*, 16(1), 04015010.

Shao, L., and Gularte, F. (2007). "Instrumentation for Vibro Stone Column Soil Improvement." *7th FMGM 2007*, 1-12.

Shivashankar, R., Dheerendra Babu, M. R., Nayak, S., and Rajathkumar, V. (2011). "Experimental Studies on Behavior of Stone Columns in Layered Soils." *Geotechnical and Geological Engineering*, 29(5), 749-757.

Six, V., Mroueh, H., Shahrour, I., and Bouassida, M. (2012). "Numerical Analysis of Elastoplastic Behavior of Stone Column Foundation." *Geotechnical and Geological Engineering*, 30(4), 813-825.

Standards Australia. (1993). "Geotechnical site investigations." *AS 1726-1993*, Australia.

- 562 Tan, S. A., Ng, K. S., and Sun, J. (2014). "Column Groups Analyses for Stone Column
563 Reinforced Foundation." *Proc., Geo-Congress 2014*, 597-608.
- 564 Walton, G., Diederichs, M. S., Alejano, L. R., and Arzúa, J. (2014). "Verification of a
565 laboratory-based dilation model for in situ conditions using continuum models." *Journal*
566 *of Rock Mechanics and Geotechnical Engineering*, 6(6), 522-534.
- 567 Yang, X., and Yin, J. (2004). "Slope Stability Analysis with Nonlinear Failure Criterion."
568 *Journal of Engineering Mechanics*, 130(3), 267-273.
569

Table 1. Measured and interpreted properties of granular fill material, column material and underlying soils for BH 4 (Case study T1)

Granular fill material		Thickness (m)	E (kPa)	c _u (kPa)	ϕ' (°)	c' (kPa)	ψ (°)
Compacted gravel		0.5	20,000	-	50.0	45.0	20.0
Load transfer platform		1.0	60,000	-	50.0	45.0	20.0
Column material		Length (m)					
Stone column material (microtonalite)		10.0	120,000	-	50.0	45.0	20.0
Soil	SPT	Thickness					
	N _{ave}	(m)					
Loose coarse SAND	4	1.1	8,000	-	35.0	-	5.0
Very soft CLAY	1	8.6	4,200	12.0	-	-	-
Stiff sandy CLAY	14	7.9	28,000	-	15.0	-	25.0
Stiff to very stiff sandy CLAY	15	4.4	30,000	-	15.0	-	24.0
Stiff to very stiff CLAY with sand	17	6.5	34,000	-	15.0	-	34.0

Table 2. Measured and interpreted properties of granular fill material, column material and underlying soils for BH 2 (Case studies T2 and T3)

Granular fill material		Thickness (m)	E (kPa)	c _u (kPa)	ϕ' (°)	c' (kPa)	ψ (°)
Compacted gravel		0.5	20,000	-	50.0	45.0	20.0
Load transfer platform		1.0	60,000	-	50.0	45.0	20.0
Column material		Length (m)					
Stone column material (microtonalite)		10.0	120,000	-	50.0	45.0	20.0
Soil	SPT N _{ave}	Thickness (m)					
Loose coarse SAND	4	1.1	8,000	-	35.0	-	5.0
Very soft CLAY	1	12.1	4,200	12.0	-	-	-
Stiff sandy CLAY	13	2.4	26,000	-	15.0	-	21.0
Stiff to very stiff sandy CLAY	11	6.0	22,000	-	15.0	-	26.0
Stiff to very stiff CLAY with sand	21	7.5	42,000	-	14.11	-	37.0

Table 3. Properties and test condition for scalped stone column material (microtonalite)

Material	D ₆₀	D ₃₀	D ₁₀	D ₅₀	C _u	C _c	Soil Classification	Applied Normal Stresses (kPa)
Microtonalite	1.5	0.6	0.2	1.1	7.5	1.2	Well graded sand-sized GRAVEL (GW)	50, 90, 120, 150, 200, 300 & 400

Table 4. Power Law function for stone column material (microtonalite) benchmarked against other established case studies

Type of Material	Geology	A	B	Source
Stone Column material	Microtonalite (peak)	3.241	0.847	This study
	Microtonalite (residual)	1.244	0.945	
Tunnelling Rock Spoils	Sandstone (Test 1 – peak)	4.68	0.76	Choo & Ong (2015)
	Sandstone (Test 1 – residual)	2.24	0.84	
	Phyllite (Test 3 – peak)	1.92	0.87	
	Phyllite (Test 3 – residual)	1.07	0.96	
	Shale (Test 4 – peak)	3.86	0.79	
	Shale (Test 4 – residual)	3.37	0.78	
Rockfill	Sandstone	6.8	0.67	Charles and Watts (1980)
	Slate	5.3	0.75	
	Slate	3.0	0.77	
	Basalt	4.4	0.81	
	Basalt	1.54	0.821	De Mello (1977)
	Diorite	1.10	0.870	
	Conglomerate	1.27	0.846	
	Conglomerate	1.19	0.881	
	Conglomerate	1.59	0.808	

Table 5. Analyzed peak and residual shear strength parameters of stone column material (microtonalite)

Geology	Power Law Function				Generalized Tangential Method			
	A_p	B_p	A_r	B_r	c'_p (kPa)	ϕ'_p (°)	c'_r (kPa)	ϕ'_r (°)
Microtonalite	3.241	0.847	1.244	0.945	44.1	50.6	11.4	41.4

Table 6. Typical design parameters against interpreted parameters (from this study) used for back-analysis

Back-analysis Method	Back-analysis using typical design parameters ^a	Back-analysis using interpreted parameters (from this study)
Young's Modulus, E (kPa)	120,000	120,000
Friction angle of column, ϕ' (°)	42.0 ^b	50.0 ^c
Cohesion, c' (kPa)	0.0	45.0 ^c
Dilation angle of column, ψ' (°)	0.0	20.0 ^d

^a Typically $c'=0$ and $\psi'=0$

^b Typical design value during design stage

^c Direct shear test from current study

^d $\psi' = \phi' - 30^\circ$, Mohanty and Samanta (2015)

Table 7. Correlation between field measured values and numerical results for case studies T1, T2 and T3

Case study	Coefficient of determination, R^2	
	Young's modulus, E	Dilation angle, ψ
T1	0.9903	0.9925
T2	0.9847	0.8740
T3	0.9973	0.9947

Table 8. Comparison of measured stone column rebound settlements against back-analyzed rebound settlements

Case study	Rebound settlement (mm)		
	Measured	Back-analysis Using Typical Design Parameters (ϕ' , $c'=0$ and $\psi'=0$)	Back-analysis Using Interpreted Parameters from This Study (ϕ' , c' and ψ')
T1	76.2	92.3	56.3
T2	10.1	72.7	24.3
T3	67.8	120.4	67.8

Table 9. Spacings and diameters of installed stone columns for case studies T1, T2 and T3

Case study	Column spacing (m)	Primary column diameter (m)	Secondary column diameter (m)
T1	2.3	1.0	0.6
T2	1.3	1.0	-
T3	2.3	1.0	1.0

Fig. 1. Interpreted subsurface soil profile and measured soil properties based on BH4 at site

Fig. 2. Interpreted subsurface soil profile and measured soil properties based on BH2 at site

Fig. 3. Plan view showing the test and borehole locations at the site

Fig. 4. Typical kentledge load test setup at site

Fig. 5. Close up view of arrangement of kentledge load test at site

Fig. 6. Result of petrographic testing on microtonalite (stone column material)

Fig. 7. Particle size distribution curves for microtonalite (stone column material) gravels and scalped sand-sized particles

Fig. 8. Direct shear test on scalped microtonalite: (a) shear stress versus horizontal displacement; (b) vertical displacement versus horizontal displacement; (c) non-linear MC failure envelope described by Power Law functions for peak and residual states

Case study	Schematic views of stone column configurations		
	Plan view	Elevation view	Configuration of load tested stone columns
T1			<p>Case T1</p>
T2			<p>Case T2</p>
T3			<p>Case T3</p>

Fig. 9. Schematic views and configuration of load tested stone columns for case studies T1, T2 and T3

Fig. 10. Field measured load-settlement responses for case studies T1, T2 and T3

Fig. 11. Finite element models used and load-settlement responses for case studies (a) T1, (b) T2 and (c) T3

Fig. 12. Typical correlation between field measured and numerical results: verification outcome for (a) Young’s modulus; (b) dilation angle of the column material for case study T3

Fig. 13. Parametric studies on the effect of friction angle and dilation angle on load-settlement responses of stone column group for case study T3: (a) $\phi' = 25^\circ$ ($\psi = 0^\circ$); (b) $\phi' = 30^\circ$ ($\psi = 0^\circ$); (c) $\phi' = 35^\circ$ ($\psi = 5^\circ$); (d) $\phi' = 40^\circ$ ($\psi = 10^\circ$); (e) $\phi' = 45^\circ$ ($\psi = 15^\circ$); (f) $\phi' = 50^\circ$ ($\psi = 20^\circ$)

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