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Behavior of Stone Columns Based on Experimental and FEM Analysis

A. P. Ambily, Ph.D.¹ and Shailesh R. Gandhi²

Abstract: A detailed experimental study on behavior of single column and group of seven columns is carried out by varying parameters like spacing between the columns, shear strength of soft clay, and loading condition. Laboratory tests are carried out on a column of 100 mm diameter surrounded by soft clay of different consistency. The tests are carried out either with an equivalent area loaded to estimate the stiffness of improved ground or only a column loaded to estimate the limiting axial capacity. During the group experiments, the actual stress on column and clay were measured by fixing pressure cells in the loading plate. Finite-element analyses have also been performed using 15-noded triangular elements with the software package PLAXIS. A drained analysis was carried out using Mohr-Coulomb’s criterion for soft clay, stones, and sand. The numerical results from the FEM are compared with the experimental results which showed good agreement between the results. Columns arranged with spacings more than 3 times the diameter of the column does not give any significant improvement. Based on the results, design charts are developed and a design procedure is suggested.

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CE Database subject headings: Stones; Columns; Finite element method; Shear strength; Clays.

Introduction

Vast areas, particularly along the coast, are covered with thick soft marine clay deposits having very low shear strength and high compressibility. In view of the increasing developments on coastal areas in the recent past, a number of ports, industries, and other infrastructure facilities are being built. This necessitated the use of land with such weak strata, wherein the designers are challenged by the presence of thick soft clay deposits. Although the use of pile foundation can meet all the design requirements, negative drag force and large length of the pile often result in prohibitive costs. On the other hand, ground improvement techniques are normally preferred for economical considerations. Out of several techniques available, stone columns (also known as granular column or granular pile) have been widely used. This ground improvement technique has been successfully applied to increase the bearing capacity and to reduce the settlement for foundation of structures like liquid storage tanks, earthen embankments, raft foundations, etc., where a relatively large settlement is permissible. Stone columns have also been used to improve slope stability of embankments on soft ground. In spite of the wide use of stone columns and developments in construction methods/equipments, present design methods are empirical and only limited information is available on the design of stone columns in codes/textbooks.

The stone column technique was adopted in European countries in the early 1960s and thereafter it has been used successfully. Stone columns in compressive loads fail in different modes, such as bulging (Hughes and Withers 1974; Hughes et al. 1976), general shear failure (Madhav and Vitkar 1978), and sliding (Aboshi et al. 1979). A long stone column having a length greater than its critical length (i.e., about 4 times the diameter of the column) fails by bulging irrespective of whether it is end bearing or floating (IS 2003). McKelvey et al. (2004) carried out experimental studies on a group of five stone columns and reported that the central column deformed or bulged uniformly, whereas the edge columns bulged away from the neighboring columns.

Many of the researchers have developed theoretical solutions for estimating bearing capacity and settlement of reinforced foundations by stone columns (Greenwood 1970; Hughes et al. 1976; Madhav and Vitkar 1978; Aboshi et al. 1979). A homogenization assumption (improved soil is assumed as a homogeneous material with equivalent properties) to estimate the ultimate bearing capacity and settlement is presented by Jellali et al. (2005). Priebe (1995) proposed a method to estimate the settlement of foundation resting on the infinite grid of stone columns based on unit cell concept. In this concept, the soil around a stone column for area represented by a single column, depending on column spacing, is considered for the analysis. As all the columns are simultaneously loaded, it is assumed that lateral deformations in soil at the boundary of unit cell is zero. The settlement improvement factor is derived as a function of area ratio and angle of internal friction of column material. Except near the edges of the loaded area, the behavior of all column soil units is the same and thus only one column soil unit needs to be analyzed (Balaam et al. 1978). The unit cell concept has also been used by Abhijit and Das (2000), Goughnour (1983), and Sathish et al. (1997). Alamgar et al. (1996) proposed an elastic approach to predict the load sharing and resulting settlement of ground improved by stone columns assuming free strain condition. Shahu et al. (2000)

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brought out the effects of a granular mat over the improved ground on its overall response within the framework of equal strain theory and unit cell concept.

Balaam et al. (1978) proposed a finite-element approach for soft clay treated with granular piles and reported the effect of stiffness of granular pile on load deformation behavior. Mitchell and Huber (1985) compared the field performance of stone columns by an axisymmetric finite-element model with groups of columns surrounding the central column replaced by a ring of stone material having equivalent thickness.

Field observations showed that stone columns could also accelerate the rate of consolidation of soft clays (Han and Ye 1992). Han and Ye (2001) developed a simplified and closed form solution for estimating the rate of consolidation of the stone column reinforced foundations accounting for the stone column soil modular ratio. It is also reported in the paper that during the process of consolidation the stress on stone column increases with time, whereas the stress on soil decreases. At the end of consolidation, a steady stress concentration ratio is approached.

The stone column improves the ground mainly due to the higher stiffness of the columns compared to the soil. Hence the most critical factor which controls the design of the stone column improved ground is the stiffness of the column and load sharing between column and soil. Hence in the present work an attempt is made to develop design procedure considering the load sharing between column and soil. The behavior of interior stone columns among a group of large numbers of columns is analyzed by varying parameters such as spacing between the columns, shear strength of the clay, angle of internal friction of stones, etc. The analyses have been carried out assuming a unit cell concept for columns arranged in a triangular pattern where the deformations in clay are restrained within the unit cell represented by the equivalent area of each column. The behavior of stone column along the periphery of the group can have lateral deformations even beyond the unit cell and have a different behavior which is not covered in the present study. Similarly, sand columns, sand drains, gravel drains, and rammed aggregate piers are not covered under the present study.

A detailed experimental investigation is carried out on a single column and groups of seven columns to study the improvement achieved. Also, the stress intensity on the column and the soil is measured using pressure cells attached to the loading plate and the results are compared with that obtained based on the finite-element package PLAXIS (Plaxis BV, The Netherlands). The analysis and parametric study have been carried out assuming a drained condition of the soft clay surrounding the column. The paper describes details of experimental works carried out, numerical analysis using finite-element package, comparison of results, design charts developed, and procedure proposed for design of stone columns.

### Experimental Program

All experiments were carried out on a 100 mm diameter stone column surrounded by soft clay in cylindrical tanks of 500 mm high and a diameter varying from 210 to 835 mm to represent the required unit cell area of soft clay around each column assuming triangular pattern of installation of columns. For single column tests the diameter of the tank was varied from 210 to 420 mm and for group tests on 7 columns, 835 mm diameter was used. Tests had been carried out with shear strength of 30, 14, and 7 kPa as summarized in Table 1. Tests with column area alone loaded were used to find the limiting axial stress and tests with entire area loaded were used to study the stiffness of improved ground.

### Test Setup

A typical test arrangement for a single column test is shown in Fig. 1. The stone column was extended to the full depth of the clay placed in the tank for a height of 450 mm so that $l/d$ ratio (length of the column/diameter of the column) is a minimum of 4.5, which is required to develop the full limiting axial stress on the column (Mitra and Chattopadhyay 1999). Vertical stress was applied either over the entire tank area or only over the stone column. The load was applied through a proving ring at a constant displacement rate of 0.0625 mm/min. A proving ring is a steel ring of approximately 200 diameter, the deformation of which is measured using a mechanical displacement gauge to arrive at the axial load applied through the ring. A sand layer of 30 mm thick was placed at the top to serve as a blanket for the case where the entire area is loaded.

### Table 1. Test Program and Results

<table>
<thead>
<tr>
<th>Type of test</th>
<th>$s/d$</th>
<th>$c_u$ (kPa)</th>
<th>Stiffness (kPa)</th>
<th>$c_u$ (kPa)</th>
<th>Limiting stress (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>vane shear</td>
<td>Test</td>
<td>FEM</td>
<td>Test</td>
</tr>
<tr>
<td>Single column</td>
<td>2</td>
<td>32</td>
<td>15,500</td>
<td>14,500</td>
<td>31</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>14</td>
<td>8,900</td>
<td>8,230</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>8</td>
<td>6,400</td>
<td>5,700</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>30</td>
<td>9,900</td>
<td>9,000</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>15</td>
<td>5,700</td>
<td>5,100</td>
<td>13</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>7</td>
<td>4,000</td>
<td>3,500</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>29</td>
<td>7,500</td>
<td>7,200</td>
<td>29</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>13</td>
<td>4,300</td>
<td>4,050</td>
<td>13</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>7</td>
<td>3,000</td>
<td>2,800</td>
<td>7</td>
</tr>
<tr>
<td>Group test</td>
<td>3</td>
<td>30</td>
<td>8,700</td>
<td>9,050</td>
<td>—</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>14</td>
<td>5,100</td>
<td>4,900</td>
<td>—</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>8</td>
<td>3,900</td>
<td>3,450</td>
<td>—</td>
</tr>
</tbody>
</table>
Tests were also carried out on a group of seven columns arranged in a triangular pattern where the clay area corresponding to an equivalent area of seven unit cells is represented by the tank area as shown in Fig. 2 to compare the behavior with a single column and to check the stress distribution between stone column and clay. The load was applied through a 16 mm thick mild steel plate with stiffeners to ensure negligible structural deformation. The locations of pressure cells fixed along the bottom surface of the loading plate are shown in Fig. 2.

Properties of Materials

Three basic materials used for this study are clay, stones, and sand having the following properties:

Clay
The clay used is of CH classification, excavated from the IIT Madras campus. Surface clay was excavated after removal of vegetation, air-dried, and pulverized. The clay was sieved through 4.75 mm sieve to remove the coarser fraction. Particle size distribution is shown in Fig. 3. The other properties are specific gravity=2.6, liquid limit=52%, plastic limit=21%, maximum dry density=16.63 kN/m$^3$, and optimum moisture content=19.26%. Other properties of clay at different moisture contents are given in Table 2. Modulus of elasticity of the clay reported is the inverse of coefficient of volume compressibility obtained from a consolidation test corresponding to a pressure range of 100–200 kPa. As the analysis is intended for the behavior of interior columns among a large group of columns, the confinement for surrounding columns justify the constrained modulus value obtained from a consolidation test. Poisson’s ratio was determined by conducting a drained triaxial test at the respective moisture content.

Stones
Crushed stones (aggregates) of sizes between 10 and 2 mm have been used to form stone column with particle size distribution as shown in Fig. 3. The $\gamma_{\text{max}}$ and $\gamma_{\text{min}}$ of the aggregate is 17.3 and 15 kN/m$^3$, respectively. Other properties of the aggregate for the stone column are given in Table 2. The angle of internal friction has been determined using a 300 mm $\times$ 300 mm $\times$ 100 mm direct shear box. The stones were compacted to a density of 16.62 kN/m$^3$ which could be achieved while constructing the stone columns for the experiments and sheared at a constant rate of 1.25 mm/min under normal pressures of 75, 100, and 125 kPa. The dilation angle is arrived at as per method suggested by Atkinson (1993). Modulus of elasticity of the stones reported is the constrained modulus obtained by loading the stones in a cylindrical mold of 150 mm diameter and 180 mm height at an initial density of 16.62 kN/m$^3$. The Poisson’s ratio used is as per typical values suggested by Bowles (1988).

Sand
The sand used is clean river sand of a size less than 4.75 mm. The angle of internal friction and the dilation angle reported in Table 2 are based on direct shear test. Poisson’s ratio reported is as per Bowles (1988).
Preparation of Soft Clay Bed

Tests have been conducted in a clay bed prepared at three different shear strengths of 30, 14, and 7 kPa. To determine the moisture content corresponding to the required shear strength values, unconfined compression tests were carried out on a cylindrical specimen of 38 mm diameter and 76 mm height. Based on the results (Fig. 4), three different water contents of 25, 30, and 35% were selected for the required shear strength of 30, 14, and 7 kPa, respectively. For preparation of each test bed, the pulverized clay was air dried and checked for initial moisture content. The additional water quantity required to achieve the desired moisture content was added and thoroughly mixed to form a uniform paste. A thin coat of grease was applied along the inner surface of tank wall to reduce friction between clay and tank wall. Clay was filled in the tank in layers with measured quantity by weight. The surface of each layer was provided with uniform compaction with a tamper to achieve a 50 mm height and uniform density as per requirement. Care was taken to ensure that no significant air voids were left out in the test bed. After the tank was filled to 450 mm height, vane shear test was carried out at the center of the tank at a depth of 50 mm below surface with a blade of 50 mm diameter and 100 mm height to verify the shear strength of the clay bed. The shear strengths obtained are reported in Table 1.

Construction of Stone Column

All stone columns were constructed by a replacement method. A thin open-ended seamless steel pipe of 100 mm outer diameter and wall thickness 2 mm was pushed into the clay at the center of the tank up to the bottom. Slight grease was applied on both inner and outer surface of the pipe for easy penetration and withdrawal without any significant disturbance to the surrounding soil. The clay within the pipe was scooped out using a helical auger of 90 mm diameter. A maximum height of 50 mm was removed at a time to ensure no suction effect. Stones were charged into the hole in layers with a measured quantity of 0.65 kg to achieve a compacted height of 50 mm. The pipe was then raised in stages ensuring a minimum of 5 mm penetration below the top level of the placed gravel. To achieve a uniform density, compaction was given with a 2 kg circular steel tamper with 10 blows of 100 mm drop to each layer. This light compaction effort was adopted to ensure that there is no significant lateral bulging of the column creating disturbance to the surrounding soft clay. The corresponding density was found to be 16.62 kN/m³. The procedure was repeated until the column is completed to the full height.

Test Procedure

After preparing the stone column, the load deformation behavior of the column/treated soil was studied by applying vertical load in a loading frame. To load the stone column area alone, a loading plate of 100 mm in diameter was placed over the stone column. The load was applied through a proving ring with a constant displacement rate of 0.0625 mm/min. Load was observed for equal intervals of settlements up to failure. In the case of the entire area loading, a 30 mm thick sand layer was placed over the entire surface. A steel plate of 12 mm in thickness and a diameter of 10 mm less than the inside diameter of the test tank was placed over the sand blanket. The loading was applied in a similar way up to an axial stress of 100–150 kPa until the settlement exceeded 10 mm. After completion of each test, the shape of the stone column was established by carefully removing the stones and filling the hole with a paste of plaster of paris. This material is in powder form and when mixed with water, it reforms into a thick paste which cannot penetrate in clay due to high viscosity and eventually gets hardened into a solid within a day. After the paste gets hardened, the surrounding clay was removed and a typical shape obtained is shown in Fig. 5. The bulging mode of failure is clearly seen in the case of tests where the column alone

### Table 2. Properties of Materials Used

<table>
<thead>
<tr>
<th>Material</th>
<th>$w$ (%)</th>
<th>$c_u$ (mm²/min)</th>
<th>$E$ (kPa)</th>
<th>$\mu$</th>
<th>$c_u$ (kPa)</th>
<th>$\phi$ (deg)</th>
<th>$\phi$ (deg)</th>
<th>$\gamma_{dry}$ (kN/m³)</th>
<th>$\gamma_{bulk}$ (kN/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay</td>
<td>25</td>
<td>0.63</td>
<td>5,500</td>
<td>0.42</td>
<td>30</td>
<td>—</td>
<td>—</td>
<td>15.56</td>
<td>19.45</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>0.99</td>
<td>3,100</td>
<td>0.45</td>
<td>14</td>
<td>—</td>
<td>—</td>
<td>14.60</td>
<td>18.98</td>
</tr>
<tr>
<td></td>
<td>35</td>
<td>1.34</td>
<td>2,150</td>
<td>0.47</td>
<td>7</td>
<td>—</td>
<td>—</td>
<td>13.60</td>
<td>18.38</td>
</tr>
<tr>
<td>Stones</td>
<td>—</td>
<td>—</td>
<td>55,000</td>
<td>0.30</td>
<td>—</td>
<td>10⁰</td>
<td>43⁰</td>
<td>16.62</td>
<td>—</td>
</tr>
<tr>
<td>Sand</td>
<td>—</td>
<td>—</td>
<td>20,000</td>
<td>0.30</td>
<td>—</td>
<td>4⁰</td>
<td>30⁰</td>
<td>15.50</td>
<td>—</td>
</tr>
</tbody>
</table>

Fig. 4. Variation of shear strength with moisture content

Fig. 5. Shape of stone column after loading
is loaded with maximum bulging at a depth of 0.5 times the diameter of the column from the top. No bulging is noticed when the entire tank area is loaded.

**Finite-Element Analysis**

The analysis was carried out using an available package—PLAXIS, to compare the load-settlement behavior with the model test and for the parametric study. The package was validated by analyzing the load settlement behavior of a single stone column by Narasimha Rao et al. (1992). The test tank used in their experiment is 650 mm diameter and height of clay bed is 350 mm. A stone column of diameter 25 mm and height 225 mm was made at the center of the clay bed and loaded with a plate of diameter equal to two times the diameter of the stone column. Properties of clay and stones are shown in Table 3. An axisymmetric analysis was carried out using Mohr-Coulomb’s criterion for clay and stones. The finite-element discretization using 15-noded triangular elements with boundary conditions is shown in Fig. 6. Fig. 7 compares the results obtained from the model test and based on PLAXIS analysis, which matches well.

**Analysis of Stone Column**

Axisymmetric analyses were carried out using Mohr-Coulomb’s criterion considering elastoplastic behavior for soft clay, stones, and sand. A drained behavior is assumed for all the materials. The analyses consider that sufficient time has lapsed after the application of the load and the stress concentration as well as the settlement have stabilized. The initial vertical stress due to gravity load has been considered in the analysis. However, the stress due to column installation depends on the method of construction and the same has not been considered in the analyses. The input parameters ($E$, $\mu$, $c_u$, $\phi$, $\gamma_{dry}$) are given in Table 2. The basic axisymmetric finite-element mesh and boundary conditions used to represent the stone column, surrounding clay and sand pad is shown in Fig. 8. Fifteen-noded triangular elements were used for meshing. Along the periphery of the tank (interface between the soft clay and the cylindrical surface of the unit cell), radial deformation is restricted but settlement is allowed. Along the bottom of the tank both radial deformation and settlement are restricted. At the interface between the stone column and soft clay, no interface elements have been used as the deformation of the column is mainly by radial bulging and no significant shear is possible. Also the interface between a stone column and clay is a mixed zone where the shear strength properties can vary depending on the method of installation. As this is not precisely known, an interface element is not used. Mitchell and Huber (1985), Saha et al. (2000), etc., also carried out a similar finite-element analysis of a stone column without an interface element.

Fig. 9(a) shows a typical deformed mesh at the time of failure in the case of column alone loaded for $s/d=3$ and $c_u=30$ kPa. Failure is by bulging of the column at a depth of 0.5 times diameter of the column. For all the cases where an entire tank area was loaded, an equal settlement analysis was carried out assuming rigid behavior of the loading plate such that the settlement at the top of the sand pad is equal over the loading area. The analysis was carried out for an axial stress corresponding to the model test for the same shear strength and spacing. Fig. 9(b) shows a typical deformed mesh when the entire area is loaded for $s/d=3$ and $c_u=30$ kPa. No bulging of the column is seen as in the case of

---

**Table 3. Properties of Materials Used for Validation of PLAXIS**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Clay</th>
<th>Stones</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus of elasticity (kPa)</td>
<td>4,000</td>
<td>45,000</td>
</tr>
<tr>
<td>Poisson's ratio ($\mu$)</td>
<td>0.45</td>
<td>0.3</td>
</tr>
<tr>
<td>Shear strength, $c_u$ (kPa)</td>
<td>20</td>
<td>0</td>
</tr>
<tr>
<td>Angle of internal friction ($\phi$)</td>
<td>0</td>
<td>38°</td>
</tr>
</tbody>
</table>

---

**Fig. 6.** Finite-element discretization of model test by Narasimha Rao et al. (1992)

**Fig. 7.** Validation of PLAXIS

**Fig. 8.** Finite-element discretization for single column
model test. Analysis for a group of seven columns was also carried out as shown in Fig. 10 using an axisymmetric model with surrounding six columns replaced by a ring of stones having equivalent thickness and material properties of stone as adopted by Mitchell and Huber (1985). The group analyses were carried out by varying $s/d$ from 1.5 to 4 and $c_u$ from 7 to 30 kPa.

### Comparison of Laboratory Tests and FEM Analysis

#### Single Column

**Column Area Alone Loaded**

Fig. 11 shows a typical relationship between axial stress and settlement for different shear strength values of a single column with $s/d=2$. Similar behavior has been observed for other $s/d$ values also. Fig. 12 shows a typical axial stress versus settlement behavior for different $s/d$ for $c_u=30$ kPa. Similar behavior is observed for $c_u=14$ and 7 kPa. As the $s/d$ increases, limiting axial stress of the column decreases up to $s/d=3$ and beyond in which the reduction in axial stress is negligible. Both experimental and finite-element results reveal comparable behavior regarding the ultimate load as well as load-deformation relationship. The limiting axial stress obtained from experiment and finite-element analyses are listed in Table 1. The ratio of limiting axial stress to corresponding shear strength is shown in Fig. 13 which is found to be constant for a given $s/d$ as well as the given angle of internal friction of stones and it is independent of the shear strength of the surrounding clay.

The analysis is extended to study the effect of the angle of internal friction of stones by varying the value of $\phi$ as 35, 40, 43, and 45° for different $s/d$'s of 1.5–4. Based on the results, limiting axial stress/shear strength ($r_{su}/c_u$) versus $s/d$ ratio for different angles of internal friction of stones is presented in Fig. 14. This relationship is valid for any shear strength value of surrounding soil. The results of experimental findings are also shown in Fig. 14 for comparison.

In most of the actual field conditions, about 10–30% of the applied load is shared by the clay surrounding the column as a surcharge, which improves the capacity of the column. To study the effect of a surcharge on limiting axial stress, finite-element analysis is extended for surcharge intensity $q$ varying from 0, 20, 40, and 60 kPa over the surrounding clay as the column is gradually loaded to failure. Fig. 15 shows a typical finite-element deformed mesh at the time of failure for the $s/d=3$ and $c_u=30$ kPa. Typical axial stress versus settlement behavior for $s/d$ of 2 and $c_u=14$ kPa under different surcharge is shown in Fig. 16. Typical variation of limiting axial stress with surcharge for the case of $s/d=2$, angle of internal friction of stones is 43°, and various shear strength of clay is shown in Fig. 17(a). A similar variation is observed for other $s/d$ ratios of 3 and 4 and other $\phi$ values. Fig. 17(b) shows the variation of limiting axial stress with a surcharge and an angle of internal friction of stones for $s/d=2$ and $c_u=30$ kPa.

### Table 1

<table>
<thead>
<tr>
<th>$s/d$</th>
<th>$c_u$ (kPa)</th>
<th>$r_{su}/c_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5</td>
<td>7</td>
<td>0.4</td>
</tr>
<tr>
<td>1.5</td>
<td>14</td>
<td>0.3</td>
</tr>
<tr>
<td>2</td>
<td>14</td>
<td>0.2</td>
</tr>
<tr>
<td>3</td>
<td>14</td>
<td>0.1</td>
</tr>
</tbody>
</table>

**Fig. 9.** Typical deformed mesh for $s/d=3$: (a) column alone loaded; (b) entire area loaded

**Fig. 10.** Finite-element discretization for group test

**Fig. 11.** Effect of shear strength on behavior of stone column

**Fig. 12.** Effect of $s/d$ on behavior of stone column
tween limiting axial stress with a surcharge on surrounding clay ($\sigma_{auq}$) and limiting axial stress without surcharge ($\sigma_{au}$):

$$\sigma_{auq} = \sigma_{au} + (0.0088\phi^2 - 0.5067\phi + 10.86)q$$

(1)

The previous relationship is obtained based on FEM considering $\phi$ varying from 35 to 45°. The experimental results without surcharge and for $\phi=43^\circ$ compare well with the $\sigma_{au}$ from FEM. However, further experimental work is required to verify the previous relationship with stones having different $\phi$ values. In a field situation, the size of the stone is large and it is unlikely that the $\phi$ value is less than 43° used in the experimental works.

**Entire Area Loaded**

This analysis aims at evaluating the improvement of the stiffness of the treated ground. The loading of both the stone column and the surrounding equivalent area with confinement of the tank wall represents an actual field condition for the interior columns of a large group of stone columns. Fig. 18 shows typical axial stress versus settlement behavior for improved and unimproved grounds based on model tests as well as finite-element analysis for different shear strengths and for $s/d$ of 2. When the entire area is loaded, because of the confining effect from the boundary of the unit cell, failure does not take place even for a settlement as high as 10 mm. Fig. 19 shows typical axial stress versus settlement behavior from model test and finite-element analysis for different $s/d$ and for a shear strength of 30 kPa. As the axial stress versus settlement relation is almost linear, stiffness (Young’s Modulus) of the improved ground can be obtained from average slope of the plot and is listed in Table 1. The stiffness improvement factor ($\beta$) is derived as stiffness of treated ground divided by stiffness of untreated ground. The stiffness of untreated ground is based on experimental results on a test tank filled with only the clay of corresponding shear strength and entire loaded area. The stiffness (see Fig. 18) of untreated soil is found to be 5,600, 3,250, and 2,260 kPa for the shear strength of 30, 14, and 7 kPa, respectively. As can be seen from Fig. 20, the stiffness improvement factor is found to be independent of the shear strength of the surrounding clay and its value for various $s/d$’s and angle of internal friction $\phi$ of stone is shown in Fig. 21 along with the experimental result. It is clear from Fig. 21 that for $s/d$ ratio beyond 3, there is no significant improvement in the stiffness.

**Group of Columns**

Fig. 22 shows a comparison of axial stress versus settlement behavior of a group of seven columns and of a single column when entire area is loaded based on both finite-element analysis as well as experimental results for $s/d=3$. As can be seen, behavior of a single column and a group of columns is almost comparable. Hence the single column behavior with a unit cell concept can simulate the field behavior for an interior column when a large number of columns is simultaneously loaded. When an entire unit cell area is loaded, the average stress on column and soil is obtained from the finite-element analysis for a group test and a single column test. Based on this, a parameter called stress concentration ratio ($n$) is determined as average stress on stone column divided by average stress on clay. The stress concentration ratio versus $s/d$ is shown for different shear strengths of clay in Fig. 23. FEM is also carried out for comparison with a higher consistency of clay having $E_s/E_c=5 \ (c_u=60 \text{ kPa})$. The corresponding curves are also included in Fig. 23. As the shear strength of the clay decreases, there is more stress concentration on the
stone column. As can be seen, the FEM results of stress concentration for both a single column and a group of seven columns are comparable and hence the unit cell concept is valid even for other $s/d$ ratios varying from 1.5 to 4. However, further experimental study is required to verify the unit cell concept for a closer spacing.

Comparison with Existing Theories

From the present work, the relationship between the friction angle for the granular material, shear strength of surrounding clay, and the limiting axial stress of single stone column without considering the effect of surcharge is compared in Fig. 24 with existing theories by Greenwood (1970), Hughes and Withers (1974), and Hughes et al. (1976). The existing theories predict the capacity of a single stone column in infinite soil mass, which does not consider the effect of spacing and surcharge. The present study incorporates the effect of spacing between columns based on unit cell concept. When the surcharge load on the surrounding clay is also considered, there is a considerable increase in load-carrying capacity. Fig. 25 compares the stiffness improvement factor obtained from the present work with the existing theories for different area ratio (area of unit cell/area of stone column) and angle of internal friction of stones. The present work predicts a slightly higher stiffness improvement factor for an area ratio more than 4 and a lower value for an area ratio less than 4 compared to Priebe (1995).

Design Example

Figs. 14, 21, and 23, can be used for the design of stone columns. The design can be carried out for a factor of safety of 1.5 against the limiting axial capacity of stone column. Such a low factor of safety is preferable to ensure a higher settlement and lateral bulging of the column so that more of the applied load is transferred to the surrounding clay. Thus, the available strength of the clay is utilized to make the design economical (Datye 1982). The method proposed for the design, using the results of the present analysis, is explained for the design of a foundation for a 30 m diameter steel storage tank with loading intensity of 120 kPa and supported
on a 12 m thick soft clay layer. The average properties of soft clay are: Shear strength=20 kPa and modulus of elasticity =4,000 kPa. The stones used for columns has modulus of elasticity=55,000 kPa and angle of internal friction=43°.

1. Depending on availability of construction equipments, choose stone column diameter between 500 and 1,000 mm, let us adopt 800 mm diameter.

2. Choose triangular pattern with a center to center spacing in the range of 1.5–3 times the diameter. Let us adopt a spacing of 1.6 m. This spacing can be changed if the stresses on stone/clay are high/low or if the settlement exceeds the permissible value.

3. For the stiffness ratio \( \frac{E_s}{E_c} \) of 13.75 and \( \frac{s}{d} \) ratio of 2, get the stress concentration ratio \( \frac{s}{d} \) from Fig. 23 as 4.8.

4. From the above-presented stress concentration ratio, find the average stress on clay and stone column based on the following relationship:

\[
\sigma_s A_s + \sigma_c A_c = \rho q A
\]

where \( \sigma_c = 4.8 \sigma_s \); \( A_s = 0.503 \text{ m}^2 \); \( A_c = A - A_s = 1.714 \text{ m}^2 \); and \( \sigma_c = 64.45 \text{ kPa} \). Solving \( \sigma_s = 309.4 \text{ kPa} \) and \( \sigma_c = 64.45 \text{ kPa} \).

5. For \( s/d = 2 \), \( c_u = 20 \text{ kPa} \), and an angle of internal friction of 43° limiting axial stress without a surcharge on the surrounding clay, \( \sigma_{su} \) is obtained from Fig. 14 as 23.5. Hence \( \sigma_{su} = 23.5 \times 20 = 470 \text{ kPa} \). The surcharge on the surrounding clay, \( q = \text{average stress shared by the clay} = \sigma_s = 64.45 \text{ kPa} \).

Using Eq. (1), limiting the axial stress on a column with a surcharge of 64.45 kPa over the clay can be obtained as follows:

\[
\sigma_{suq} = \sigma_{su} + (0.0088 \phi^2 - 0.5067 \phi + 10.86)q
\]

\[= 470 + (0.0088 \times 43^2 - 0.5067 \times 43 + 10.86) \times 64.45\]

\[= 814 \text{ kPa}\]

6. Find the factor of safety available = \( \frac{\sigma_{suq}}{\sigma_s} = 2.63 > 1.5 \) and hence safe.

7. Ultimate bearing capacity of soil \( \left( q_{ult} \right) = 5.14 \times 102.8 \text{ kPa} \). A factor of safety available against a bearing capacity of soil \( q_{ult} / \sigma_s = 1.59 \) (1.5 is adequate). If the factor of safety available for limiting stress on a column or on clay is less than 1.5 in Step 4 or 5, the design has to be revised by either increasing the diameter of the column or by reducing the spacing between the columns.

8. Find the settlement of untreated deposit as...
9. Find the stiffness improvement factor from Fig. 21 for s/d = 2 and $\phi = 43^\circ$ as 2.6. Settlement of tank after treatment = 331.2/2.6 = 127.4 mm. Compare the treated settlement with permissible value and revise the design, if required.

**Conclusions**

The present work describes experimental and finite-element analyses carried out to study the effect of shear strength of soil, angle of internal friction of stones, and spacing between the stone columns on the behavior of stone columns. Experiments are carried out to study the effect of shear strength of soil, angle of internal friction of stones and spacing between the stone columns. Analyses carried out to study the effect of shear strength of surrounding clay and depends mainly on column spacing and on the angle of internal friction of the stones.

6. Stiffness improvement factor is found to be independent of shear strength of surrounding clay and depends mainly on column spacing and on the angle of internal friction of the stones.

**Notation**

The following symbols are used in this paper:

- $A$: area of a unit cell;
- $A_c$: area of clay in the unit cell;
- $A_s$: area of stone column;
- $c_u$: undrained shear strength of clay;
- $c_v$: coefficient of consolidation;
- $d$: diameter of stone column;
- $E$: Young’s modulus of elasticity;
- $E_s$: Young’s modulus of elasticity of compacted stones;
- $H$: thickness of clay layer;
- $I_f$: influence factor as per Boussinesq’s theory;
- $L$: length of stone column;
- $m_v$: coefficient of volume compressibility of clay;
- $n$: stress concentration ratio, $\sigma / \sigma_v$;
- $p_0$: intensity of loading;
- $q_{ult}$: ultimate bearing capacity of clay;
- $s$: center to center spacing between stone columns;
- $v$: water content;
- $\beta$: stiffness improvement factor;
- $\gamma_{dry}$: dry unit weight;
- $\gamma_{max}$: maximum unit weight;
- $\gamma_{min}$: minimum unit weight;
- $\gamma_{wet}$: bulk unit weight;
- $\Delta \sigma_z$: increase in vertical pressure at depth $z$ due to the applied pressure $p_0$ at the surface;
- $\mu$: Poisson’s ratio;
- $\sigma_c$: average stress over stone column;
- $\sigma_v$: average stress over clay;
- $\sigma_{saq}$: limiting axial stress of stone column with no surcharge over the surrounding clay;
- $\sigma_{saq}$: limiting axial stress of stone column with surcharge $q$ over the surrounding clay;
- $\phi$: angle of internal friction; and
- $\psi$: dilation angle.

**References**


