Sand and stone columns in soft soil at different relative densities

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ABSTRACT

The technique of sand or stone columns is widely used to improve the load carrying capacity and reduce the settlement of soft soils. The technique consists of excavating holes of specific dimensions and arrangement in the soft soil and backfilling them with either stone or sand particles. The efficiency of the technique depends primarily on the type of the backfill material (sand or stone) and gradation as well as the placement relative density.

In the present research, holes 50 mm in diameter and 300 mm in length were excavated in a bed of soft soil, 400 mm in thickness, of undrained shear strength between 16-19 kPa. The holes were backfilled with either sand or stone particles at loose and dense states. Each column was loaded gradually through a circular rigid footing 64.6 mm in diameter up to failure with continuous monitoring of the settlement.

The outcomes of the model tests revealed that for both floating and end bearing types, the sand columns at low relative density exhibited higher bearing improvement ratios and lower settlement reduction ratios compared to stone columns. On the other hand, a reverse behavior was noticed, when the backfill material was placed at "dense state". The results shed the light on the importance of placement relative density of both backfill materials. The results are thoroughly analyzed in terms of the stress concentration ratio and stiffness ratio.

Keywords: sand, stone, columns, ground improvement, soft soil

1. INTRODUCTION

Iraq is among the countries where nearly 30 to 40 % of its area is characterized as soft saturated silty clay. This soft soil exists along the alluvial plain, beginning from north of Baghdad and extending south to the Arabian Gulf. The area is expected to experience rapid development in its infrastructure and hence ground improvement becomes a major task for construction industry.

Stone or sand columns have been widely applied internationally as a successful, sustainable and efficient technique for improving the load carrying capacity and controlling the settlement of soft soils and in many cases considered as an economic alternative to deep foundation.

The selection of the type and properties of the backfill material plays a major role in the design process as it is directly related to the cost and the feasible benefits to be achieved. Crushed stone has been widely used in many projects and proved to be efficient in providing satisfactory improvements when used in reinforcing soft deposits. Up to present there are no clear specifications or limitations of the properties of the backfill granular material regarding size of particles, type of gradation and placement density. From practical point of view, the optimum size of the particles must be selected to attain easy filling of the column and satisfactory degree of compaction. After thorough review of many articles “Barksdale and Bachus (1983)”, “Andreou et al. (2008)”, “Samenta et al.(2010)” and “Karun and Nigee (2013)” and many others, it is found that there is a wide range of average stone particle diameter to column diameter (1/10, 1/12, 1/13, 1/20, 1/30,1/64).

In cases where there is lack of proper crushed stone or the location of the quarry is far from the required site, sand will be the alternative source. This is currently the case in the southern parts of Iraq where one stone quarry exists, located in Najaf city south west of Baghdad. Sand is available in abundant quantities in many areas of Iraq and often considerably cheaper and faster in construction than stone. One of the disadvantages of sand is its lower angle of internal friction and lower stiffness compared to stone. “Barksdale and Bachus (1983)”, “Al- Zuhairi (2000)”, “Al- Gharbawi (2013)” and “Rajab (2013)”. The present work is an attempt to evaluate the influence of placement relative density on the improvements in bearing capacity and settlement of soft soil reinforced with sand and stone columns.
2 LABORATORY MODEL TESTS

2.1 Material properties

Soil used
The soil used was brought from Al-Nahrawan city. The soil consists of 16% sand, 34% silt and 50% clay. Atterberg limits revealed LL = 44 and PI = 25. According to the Unified Soil Classification System (USCS), the soil is classified as CL (clay of low plasticity). The soil was prepared at undrained shear strength between 16 - 18 kN/m².

Sand used
The sand used has a specific gravity 2.65. The grain size distribution consists of 10% gravel, 89% sand and 1% fines with D₁₀, D₃₀ and D₆₀ are 0.28 mm, 0.79 mm and 2 mm respectively, revealing coefficient of uniformity 7.14 and coefficient of curvature 1.11, accordingly classified as well graded sand. The dry unit weights used in the construction of sand columns are 17 and 19.1 kN/m³ corresponding to relative densities 15% (loose state) and 70% (dense state).

Stone used
The crushed stone was brought from a crushing stone factory, white in color and angular in shape with specific gravity 2.61. D₁₀, D₃₀ and D₆₀ are 4.9, 5, and 5.2 mm respectively revealing coefficient of uniformity 1.06 and coefficient of curvature 0.98, classified as poorly graded. The dry unit weights used in the construction of stone columns are 13 and 15.7 kN/m³ corresponding to relative densities 23% (loose state) and 71% (dense state).

2.2 Model preparation and testing

Beds of fully saturated soil were prepared inside steel containers 600 mm * 600 mm * 500 mm in depth. The dry soil was mixed thoroughly with the required amount of water to obtain soil of undrained shear strength between 16 – 18 kN/m². The lumps of soil were placed in layers inside the container and each layer was tamped gently to remove any entrapped air. The process continues till the thickness reaches 400 mm for floating models and 300 mm for end bearing models. After completion of the final layer, the top surface was scraped and levelled to get approximately a flat surface. Each bed of soil was divided into 4 zones 300 mm * 300 mm and the sand or stone column was constructed at the center of each zone. A PVC tube 50 mm in diameter was inserted vertically to the required depth and the inside soil was removed completely. The sand or stone was carefully charged into the hole in three layers to achieve the loose state densities of 17.0 kN/m³ for sand and 13 kN/m³ for stone. In case of the dense state, the sand or stone was charged in five layers with gentle tamping revealing a dry unit weight of 19.1kN/m³ for sand and 15.7 kN/m³ for stone. After completion of the construction of the columns, a steel circular rigid footing 64.6 mm was placed on each column and loaded gradually up to failure. The loading assembly is shown in “Fig.1”.

Fig.1. Loading Assembly

3 ANALYSIS AND DISCUSSION OF RESULTS

The following terms are used in the evaluation of improvements achieved by sand or stone columns. The bearing ratio q/cᵤ represents the ratio of the applied stress to the undrained shear strength. The bearing improvement ratio q /cᵤ t /q /cᵤ u represents the ratio of the bearing ratio of the treated soil to the bearing ratio of the untreated soil. The settlement reduction ratio S t/Su represents the ratio of the settlement of the treated soil to the settlement of the untreated soil.

3.1 Floating type columns
“Figs.2 and 3” demonstrate the variation of the bearing ratio and bearing improvement ratio versus settlement ratio respectively. “Fig.2” shows that when sand or stone is placed at loose state, the sand exhibits a stiffer behaviour while the reverse is obtained in the dense state. In “Fig.3”, the bearing improvement ratios achieved at failure (S/D = 10 %) are 1.85 and 1.70 for the dense stone and dense sand respectively. On the other hand, the bearing improvement ratios of the stone and sand columns at loose state are 1.2 and 1.42 respectively.

“Fig.4” shows the bearing ratio q/cᵤ versus the settlement reduction ratio S t/Su. The general trend of sand indicates a steep reduction in S t/Su up to q/cᵤ = 3 then levelled off gradually up to q/cᵤ = 5, revealing a final settlement reduction ratio 0.58 and 0.33 for the loose and dense states respectively. On the other hand the stone demonstrated a slight decrease in S t/Su up to
$q/c_u = 2$ for both loose and dense states, followed by a steep drop for the dense state that levelled off gradually at $q/c_u = 5$ providing a settlement reduction ratio 0.3. The stone in the loose showed similar trend exhibiting a settlement reduction ratio of 0.75 which is considered as marginal compared to the dense state.

### 3.2 End bearing type columns

The end bearing columns were constructed to rest on the base of the container, simulating the case in the field, where the columns extend through the soft layer and rest on a firm layer. Similar to the floating type, “Figs. 5 and 6” demonstrate the bearing ratio and bearing improvement ratio versus the settlement ratio respectively. “Fig. 5” demonstrates that both materials demonstrate an increase in stiffness as a result of changing their states from loose to dense.

This change in behavior is more pronounced in stone columns compared to the sand columns. The dense stone column exhibits approximately a linear behavior up to $S/D = 3\%$, followed by a rapid increase in the rate of settlement. In such behavior, $S/D = 3\%$, indicates the start of the yielding stage followed by the gradual development of the bulging mode up to failure. Such behavior was not clearly observed in the dense sand columns.

“Fig. 6” illustrates the relationship between the bearing improvement ratio and the settlement ratio. The dense stone column demonstrated peak improvement ratio of 2.75 at settlement ratio 3 % followed by a steady drop up to failure, revealing an improvement ratio 2.4 at failure ($S/D = 10\%$), while...
loose stone column provided improvement ratios of 1.1 at S/D =3% and 1.3 at failure. On the other hand, the dense and loose sand columns provided improvement ratios of 2 and 1.5 at failure.

“Fig.7” demonstrates the relationship between the bearing ratio versus settlement reduction ratio. Since the dense stone column exhibits the maximum stiffness, the settlement reduction ratio drops to 0.15 at q / c_u = 2 and remains decreasing steadily to a final value of 0.11. The dense sand exhibits similar behavior to the dense stone, an initial sharp drop to a value 0.21 at q/c_u =3, then remains steady up to failure. The loose stone and sand columns showed close behavior up to q/c_u =3, then the loose stone gradually dropped to 0.59 at q/c_u=5, and the loose sand dropped to 0.44 at q/c_u=5. The bearing ratio at failure, bearing improvement ratios and settlement reduction ratios are summarized in “Table 1”

3.3 Stress concentration ratio and stiffness

“Aboshi et al. (1979)” stated that when a uniform load is applied to composite soil reinforced with stone or sand columns, the stress concentrates on the columns due to the difference in the deformation characteristics or stiffness between the columns and the surrounding soil. The stress concentration factor for stone or sand columns in a cohesive soil matrix is defined as follows

\[ n = \frac{q_t}{q_i} \]  
\[ q = q_s + q_c(1 - a_s) \]  
\[ q_c = q/[1 + (n - 1)a_s] = \mu_c * q \]  
\[ q_s = nq/[1 + (n - 1)a_s] = \mu_s * q \]
Where

$q$ is the average applied stress
$q_c$ is the applied stress on the column
$q_s$ is the applied stress on the clay
$\alpha_s$ is the area replacement ratio
$\mu_c$ is the stress concentration factor
$n$ is the stress concentration factor

According to the above equations, the stress at failure of the composite soil defined as the stress ratio corresponding to settlement ratio $S/D = 0\%$, is deducted from “Figs. 2 and 5” for each model test. The stress concentration factor $n$ and the stress ratios $\mu_c$ are calculated and presented in “Table 2”, based on the assumption that full mobilization is achieved at $S/D=10\%$.

To estimate the ratio of the stiffness of the column to the stiffness of the cohesive soil, “Watts et al. (2000)” proposed equation

$$E_{eq} = \frac{(E_s \cdot a_s + E_c \cdot a_c)}{(A_s + A_c)} \quad (5)$$

Where

$E_{eq}$ is Young’s modules of the treated soil
$E_s$ is Young’s modules of the stone or sand column
$E_c$ is Young’s modules of the clay soil

Equation 5 can be rewritten as

$$E_{eq} = E_s \left[ a_s + \left( \frac{E_c}{E_s} \right) (1 - a_s) \right] \quad (6)$$

Since the settlement reduction ratio at any stress level is proportional to the Young’s modulus, the values of $E_{eq}/E_c$ are the reciprocal of $St/St_m$, deducted from “Figs. 2 and 5” and then substituted in equation 6 to determine the ratio of $E_s/E_c$. The ratio $E_s/E_c$ is a good indicator of the improvements achieved in terms of settlement reduction ratio and bearing improvement ratio. “Table 3” illustrates the values of $E_s/E_c$, $S_t/S_{tn}$ and $n$. It is clear that $E_s/E_c$ value is directly related to both $n$ and $S_t/S_{tn}$ values. The table also demonstrates that floating type columns exhibit lower stiffness ratio compared to end bearing columns as floating columns can sink into the soft soil and thus exhibit less resistance to the applied stress.

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<th>Table 2. Failure stresses and stress concentration factors</th>
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<td>Floating Type column</td>
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<th>Table 3. Stiffness ratio and settlement reduction ratio</th>
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4 CONCLUSIONS

The article sheds the light on the importance of placement relative density and type of the backfill material used in the construction of granular columns in soft soil. Currently no guidelines are yet available in the literature regarding the type, gradation and other geotechnical properties of the backfill material. The common practice is to use crushed stone as a backfill material due to its satisfactory stiffness. However in some cases, sand is considered as an accepted alternative due to economic consideration.

In both floating and end bearing types, the dense stone columns exhibit higher bearing improvement ratio and lower settlement reduction ratio compared to the sand columns. Apart from the type of material, it is important to adopt proper and effective compaction technique and equipment that match with the selected granular backfill material to achieve the highest relative density and thus optimum improvements.

REFERENCES


