INTERNATIONAL SOCIETY FOR SOIL MECHANICS AND GEOTECHNICAL ENGINEERING

Proceedings of the 16th International Conference on Soil Mechanics and Geotechnical Engineering

doi:10.3233/978-1-61499-656-9-1327

Performance prediction of stone-column-supported foundations

Prévision d'exécution des bases pierre-colonne-soutenues

José L.M. Clemente, Harish Senapathy & John R. Davie
Bechtel Corporation, Frederick, MD, USA

ABSTRACT

Three-dimensional numerical analyses, using FLAC-3D (Itasca 2002), were conducted to study the performance prediction of stone-column-supported foundations. The analyses included a combination of different soil profiles, and square foundation sizes. Unimproved soil profiles, and soil profiles including stone columns at different square grid configurations were employed. The effects of stone-column spacing, depth of ground improvement, and the number of stone-columns under the loaded area were identified from the analysis results. Some of the basic assumptions contained in the current design methodology were generally confirmed by the numerical results. However, new features such as the stress-dependent nature of improvement factors were also identified. A reassessment of the currently available stone column ground improvement design methodology is proposed. Additional analyses using a wider range of stone column configurations will be required to broaden the findings of this work.

RÉSUMÉ

Des analyses numériques tridimensionnelles, en utilisant FLAC-3D (Itasca 2002), ont été conduites pour étudier la prévision d'exécution des bases pierre-colonne-soutenues. Les analyses ont inclus une combinaison de différents profils de sol, et des tailles de base de place. Des profils non améliorés de sol, et les profils de sol comprenant les colonnes en pierre à différentes configurations carrées de grille ont été utilisés. Les effets de l'espacement de pierre-colonne, de la profondeur de l'amélioration au sol, et du nombre de pierre-colonnes sous le secteur chargé ont été identifiés des résultats d'analyse. Certaines des prétentions de base contenues dans la méthodologie de conception courante ont été confirmées par les résultats numériques généralement confirmés. Cependant, de nouveaux dispositifs tels que la nature soumettre à une contrainte-dépendante des facteurs d'amélioration ont été également identifiés. On propose une réévaluation de la méthodologie de conception au sol d'amélioration de colonne en pierre actuellement disponible. Des analyses additionnelles employant un éventail de configurations en pierre de colonne seront exigées pour élargir les résultats de ce travail.

1 INTRODUCTION

Stone columns are formed by inserting a vibratory probe into the soil creating a hole that is filled with stone. The procedure when used within the context of this study will refer to applications in soils that contain more than about 20 percent silty fines or almost any amount of clayey fines (Baez 1993). Stone column installation into these soils causes little to no densification of the surrounding soils due to the high fines content. Probe diameters typically range from 0.3 to 0.6 m resulting in stone column diameters of up to 1 m. Typical stone column grid spacings range from 1.5 to 2.5 m.

Application of the stone column technology requires the selection of stone column depth, diameter and grid spacings to meet specified foundation settlement criteria. Currently, a preliminary stone column configuration is selected to yield a target settlement-based improvement factor (IF), which is defined as the ratio of foundation settlement for original ground conditions over the foundation settlement for improved ground conditions.

Once the desired IF value is established, published empirical correlations between the IF and stone column Area Replacement Ratio (ARR), such as the curves proposed by Priebe 1993 and shown on Figure 1, can be used to determine the stone column configuration.

Priebe (1993) also suggests an increase in ARR to account for the relative stiffness of stone column to soil, which is not included on Figure 1.

Drawbacks of the grid configuration selection procedure described above include the following:

- It relies very heavily on empirical correlations that often do not account for the actual mechanical properties of the soils.
- The improvement factor is considered constant for a given ARR. Large-scale field load tests (Clemente et. al. 1997, Clemente & Davie 2000) have shown that IF is also affected by bearing pressure.

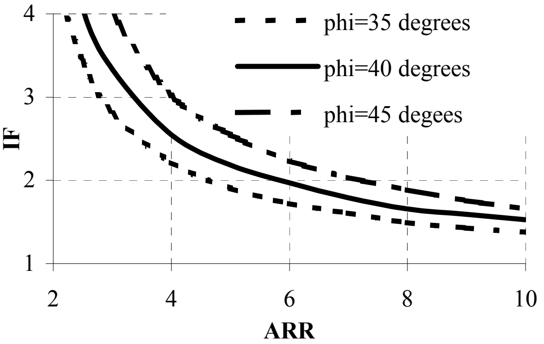

Figure 1 – IF versus ARR by Priebe 1993

The goal of this study was to numerically develop relationships between IF and ARR that take into account the actual subsurface and stone column mechanical properties, as well as the effects of bearing pressure and foundation size. The analyses included stone column geometry (diameter and length), grid configuration (square), foundation geometry (square foundations), and stiffness of the stone column and surrounding soil (E_c, E_s). A homogeneous one-soil layer was used, and no steady-state or transient ground water effects were included.

2 NUMERICAL MODEL

The numerical model used employs a finite difference method to solve equilibrium equations for a 3-dimensional continuum, which in the present case consists of the soil and stone columns. The 3-dimensional continuum is discretized by grids consisting of polyhedral elements.

Upon application of gravity loads and foundation loads, the grid deforms and reaches equilibrium configurations under the applied loads. Displacements of each grid node are calculated using the appropriate constitutive model for each material (soil and column). Since the improvement factor of only the zone with the stone columns is of concern, the 3-D models extend only to the bottom of the stone columns. Four vertical boundaries were defined at a horizontal distance from the edge of the stone columns equal to twice the stone column center-to-center spacing, and a horizontal boundary was defined at the bottom of the soil profile. Grid points along the 4 vertical boundaries of the model were prevented from moving horizontally, thus providing rigid vertical boundaries. Grid points at the bottom horizontal boundary were prevented from moving both horizontally and vertically, thus providing a rigid bottom boundary. While in most stone column applications the boundary at the bottom of the treated layer is not rigid, either stiffer soils or rock are present, and thus the assumption used in the model is reasonable.

Both the soil and stone columns are modeled as Mohr-Coulomb materials. The soil profile consisted of clay ($c>0$ and $\phi=0^o$), and the column consisted of a slightly cohesive, granular material ($c>0$ and $\phi>0^o$), where c is the undrained shear strength of the material, and ϕ is the angle of shearing resistance.

To minimize the effects of the computer generated mesh sizes and configurations on the estimated settlement and the corresponding improvement factors, the following steps were adopted:

- For every case analyzed, the settlement of the “soil-only” and the “soil with stone column” cases were analyzed simultaneously.
- To keep the grid dimensions and configurations identical, the two models were created using the same steps. To generate the “soil with stone-columns” model the surrounding soil is first created with cylindrical holes. Stone-column elements are then inserted into the cylindrical holes and connected to the surrounding soil using interfaces. The “soil-only” model was generated using the same procedure. The only difference in the two models is that the stone-column elements in the “soil with stone-columns” model has the properties of a stone-column where the stone-column elements in the “soil-model” model has the properties of the surrounding soil.

Since the improvement factor is a ratio of the improved and unimproved settlements, the above procedure would minimize the effects of computer modeling.

The foundation loading was applied over a square area that remained flat as the load increased, thus resulting in uniform vertical displacements beneath the footprint of the foundation. The bearing pressure beneath the footprint of the foundation was thus different from grid point to grid point, and the foundation pressure was calculated by averaging the bearing pressure values at each grid point beneath the foundation.

No ground water effects were included in the analyses, i.e., it was assumed that the ground water table was located below the bottom horizontal boundary of the model.

3 NUMERICAL STUDIES

3.1 *Stone Column and Finite Difference Grid Configurations*

Four basic types of stone column configurations were used, as described below:

- Square stone column configuration consisting of soil and 9 stone columns. This will be referred to as the 1x1 configuration. The stone column diameter (D) is 1 m, and stone column center-to-center spacing (S) is 1.5 m (S/D=1.5), 2 m (S/D=2) and 3 m (S/D=3). The soil profile thickness (L) is 3 m (L/D=3), 6 m (L/D=6) and 9 m (L/D=9). The horizontal distance from the edge of the stone columns to the vertical boundaries (S_B) is $S_B/S=2$. Equal vertical displacements were applied to the FLAC 3D grid points beneath the footprint of a square footing with width (B_F) such that $B_F/S=1$, i.e., $B_F=1.5$ m (for S/D=1.5), 2 m (for S/D=2) and 3 m (for S/D=3). The footing was centered on the internal column.
- Square stone column configuration consisting of soil and 16 stone columns. This will be referred to as the 2x2 configuration. The stone column diameter (D) is 1 m, and the same S/D, L/D and S_B/S values used for the 1x1 column configuration are used here. The square footing width (B_F) is such that $B_F/S=2$, i.e., $B_F=3$ m (for S/D=1.5), 4 m (for S/D=2) and 6 m (for S/D=3), and the footing was centered on the 4 internal columns. The typical FLAC3D grid for this configuration is illustrated in Figure 2.

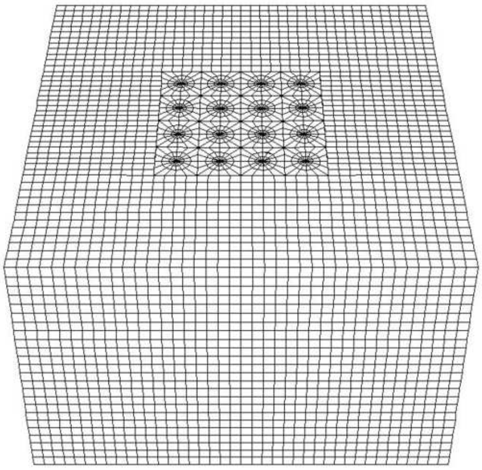

Figure 2 – Numerical model for the 2x2 configuration

- Square stone column configuration consisting of soil and 49 stone columns. This will be referred to as the 5x5 configuration. The stone column diameter (D) is 1 m, and the same S/D, L/D and S_B/S values used for the previous configurations are used here. The square footing width (B_F) is such that $B_F/S=5$, i.e., $B_F=7.5$ m (for S/D=1.5), 10 m (for S/D=2) and 15 m (for S/D=3), and the footing was centered on the 25 internal columns.
- Model consisting only of soil. This is necessary to obtain foundation settlements on the untreated soil, which is required to establish IF values. The same footing sizes and footing locations with respect to the horizontal boundaries used for the soil/column cases were used for the soil only cases. Thus, for each soil/column case described in the previous bullets, a soil-only case was also analyzed.

The stone column diameter used in the analyses (D=1 m) is representative of stone columns installed by the wet method (vibroreplacement). Stone columns installed by the dry method (vibrodisplacement) generally have smaller diameters. All columns extended from the ground surface to the bottom of the soil profile, so as to eliminate the possible contribution to footing settlement from layers below the bottom of stone columns, and thus allow for estimates of improvement in the layer where the columns are installed. The soil profile thickness (L=3 m, 6 m and 9 m) is representative of profile thickness where stone columns are effective.

3.2 *Engineering Parameters*

Engineering parameters for the soil were selected to reflect subsurface conditions typical of those where ground improvement by stone columns could be employed. Thus, a cohesive profile was used consisting of a medium stiff clay. Engineering properties for the clay are c=28.8 kPa (600 psf) and $\phi=0^o$. The stone columns were modeled as a granular material with a small amount of cohesion. Engineering properties for the stone columns are c= 4.8 kPa (100 psf) and $\phi=40^0$. The small cohesion can be justified by the fact that some amount of fines migration is expected from the clay into the columns. These and other engineering properties used in the analyses are summarized in Table 1.

Table 1 – Engineering properties used in the analyses

Property	Soil	Stone Column
Cohesion, c (kPa)	28.8	4.8
Angle of Shearing Resistance, ϕ	0	40^o
Unit Weight, γ (kN/m^3)	19.6	17.7
Elastic Modulus, E (MPa)	17.3	119.9
Poisson's Ratio, ν	0.4	0.3

A parametric study was performed to check the effects of Poisson's ratio and angle of shearing resistance of the stone columns on the calculated IFs. The parametric study also served to develop comparisons to the IFs obtained from Priebe 1993. The parametric study was performed using the 1x1 stone column configuration with S/D=1.5 and L/D=3. Thus, the FLAC analyses made use of a square footing with $B_f = 1.5$ m.

The results suggest that ν has only a very minor influence, and ϕ_c has a relatively minor influence on the results. Based on these results, a value of $\nu=0.3$ and $\phi_c=40^o$ was used for the stone columns in all subsequent analyses. Note that $E_c/E_s=6.9$.

3.3 *Numerical Results for Various Stone Column Configurations*

Bearing pressure versus settlement results from FLAC 3D were obtained for all configurations for both the soil-only condition, and the soil with stone columns conditions. IF values were then calculated for different bearing pressure values by dividing the settlement of the stone column and soil supported footing by the settlement of the soil supported footing. The results are shown in Figures 3, 4 and 5.

It can be shown that ARR is 2.9, 5.1 and 11.5 for the different square footing sizes used in the three different configurations. The Priebe curve on Figures 3, 4 and 5 has been adjusted to account for $E_c/E_s=6.9$. Priebe's curve shown on Figures 3, 4 and 5 is thus "lower" than the Priebe curve shown on Figure 1.

The trend of increasing FLAC-generated IF-values with increasing bearing pressures is observed on Figures 3, 4 and 5.

The charts on Figures 3, 4 and 5 also indicate an increase in IF as the number of stone columns increases. This is important from a full-scale load test interpretation standpoint. Generally field load tests are conducted on 1x1 or 2x2 configurations of stone columns to minimize test footing size, whereas the number of stone columns under a large foundation is considerably more than 5x5. The charts on Figures 3, 4 and 5 indicate that the IF value obtained from the load test can be increased by at least 10% when considering the same stone column configuration for a large foundation. Even if the increase is only about 10%, this could lead to substantial savings in cost and schedule when considering large-scale projects.

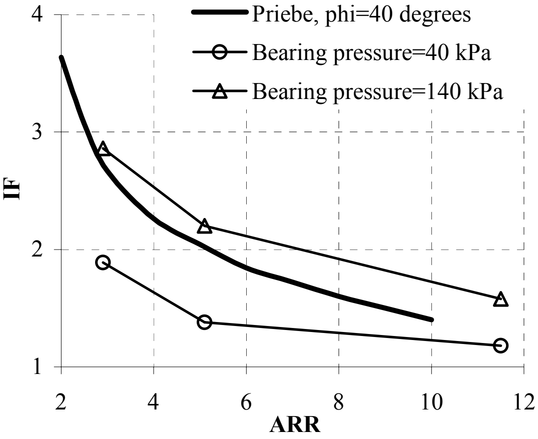

Figure 3 – Comparison of Priebe 1993 and FLAC IF versus ARR for the 1x1 Configuration

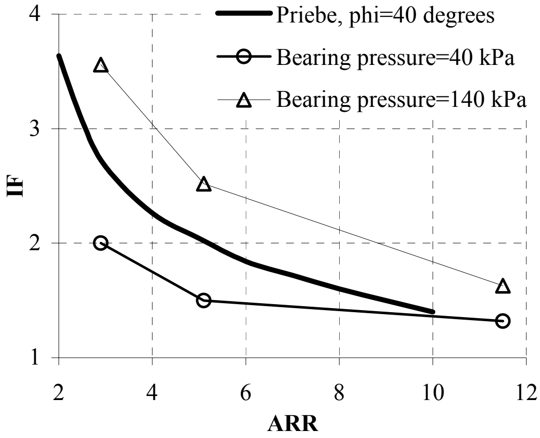

Figure 4 – Comparison of Priebe 1993 and FLAC IF versus ARR for the 2x2 Configuration

4 EQUATION TO ESTIMATE IF

Based on the numerical results, an equation was developed to estimate IF taking into account the bearing pressure and ARR, and ignoring the depth of improvement. The following assumptions were made to generate this equation:

- IF will always be equal to or greater than 1.
- To simplify the equation, a range of bearing pressures was selected. The minimum bearing pressure in the range was set just above the allowable bearing pressure of the surrounding soil. Generally, ground improvement would not be required if the actual bearing pressure is less than the allowable bearing pressure, which is about 2 times the undrained shear strength. The maximum bearing pressure in the range was set at the ultimate bearing pressure of the

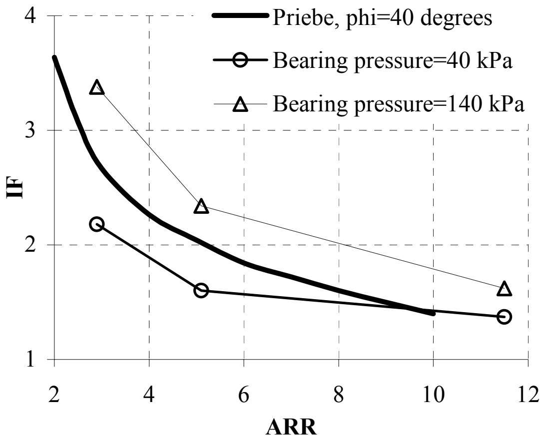

Figure 5 – Comparison of Priebe 1993 and FLAC IF versus ARR for the 5x5 Configuration

surrounding soil, which is about 6 times the undrained shear strength.

- The equation was developed so that the estimated IF values are slightly lower than the FLAC3D values for the 1x1 configuration. The equation generally underestimates IF for the 2x2 and the 5x5 configuration.

$$IF = 1+ \{(-0.078 \cdot \log(ARR)+0.109) \cdot p^3 + (2.377 * \log(ARR)-6.523) \cdot p^2 + (-338.875 * \log(ARR)+728.618) \cdot p\}/100,000$$

where: p = bearing pressure in kPa

5 BEARING PRESSURE BASED IF

The results summarized on Figures 3, 4 and 5 highlight the shortcoming of defining IF solely as a settlement ratio. It is quite clear that IF is also bearing pressure dependent. Also it cannot be defined once the bearing pressure reaches values where large rates of settlement start to develop.

An alternative way of defining the improvement factor is in terms of bearing pressure ratios for an allowable settlement. For instance, an allowable settlement of ΔH=25 mm, which is widely considered adequate for spread footings (Peck et al 1974), can be used. IF in this case can be defined as the ratio of the bearing pressure corresponding to ΔH=25 mm for the soil and stone column case over the the bearing pressure corresponding to ΔH=25 mm for the soil only case.

Lower bound and upper bound curves for IF defined as a bearing pressure ratio for ΔH=25 mm are shown graphically on Figure 6. The Priebe curve is also shown on Figure 6 for reference purposes only. Direct comparisons between the Priebe settlement based IF curve and the bearing pressure based IF curves are not appropriate because of the different ways in which IF is defined for each case. However, Figure 6 shows that the bearing pressure based IF curves follow a trend similar to that of the Priebe curve, i.e., IF values decrease with increasing ARR values.

It is expected that curves similar to those shown on Figure 6, developed for ΔH > 25 mm, say, for ΔH = 50 mm, would show higher IF values, everything else being the same.

A new design methodology that makes use of curves such as those shown on Figure 6 would consist of evaluating the bearing pressure corresponding to ΔH = 25 mm for the soil only case, and applying the corresponding bearing pressure based IF value to calculate a higher bearing pressure for the soil and stone column case for ΔH = 25 mm. Such bearing pressure based design methodology would be more rational than that currently used, but would require considerable additional work to generate bearing pressure based IF curves for different soil types and stone column grid configurations.

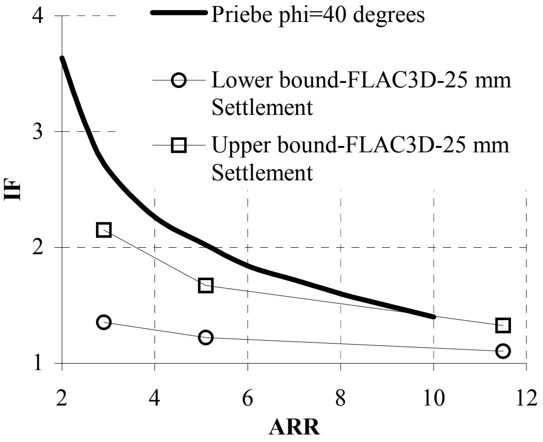

Figure 6 – Range of IF defined as a bearing pressure ratio for ΔH=25 mm

6 CONCLUSIONS

Several conclusions were identified in the study, and the following are directly related to the results presented here:

- Improvement Factor (IF) defined as a settlement ratio is dependent on bearing pressure and tends to increase with increasing bearing pressure.
- An equation was developed based on FLAC results to calculate IF taking into account not only the area replacment ratio (ARR), but also the bearing pressure. This is a new contribution to existing published procedures that can be further improved to also account for soil strength.
- Defining bearing pressure based IFs as pressure ratios for a fixed, tolerable settlement (say ΔH=25 mm) has positive implications in permanent foundation design on stone column improved soils.

ACKNOWLEDGEMENT

The work described here was sponsored by a 2003 Bechtel Technical Grant.

REFERENCES

Baez, JI (1993). “Advances in the Design of Vibro Systems for the Improvement of Liquefaction Resistance”, *Proceedings of the Symposium on Ground Improvement*, Vancouver Geotechnical Society, Vancouver, BC, Canada.

Clemente, JLM, M Vogelfanger, LW Young, Jr. and D Heere (1997). “Ground Improvement for a Large Power Plant”, *Proceedings of the XV International Conference on Soil Mechanics and Foundation Engineering*, Hamburg, Germany, pp. 1677-1680.

Clemente, JLM, and J Davie (2000). "Stone Columns for Settlement Reduction", *Proceedings of GeoEng2000*, Melbourne, Australia.

Itasca Consulting Group, Inc. (2002). *FLAC-3D User’s Guide*, Minneapolis, MN.

Peck, RB, WE Hanson and TH Thornburn (1974). *Foundation Engineering, 2nd Ed.*, John Wiley & Sons, Inc., New York, NY.

Priebe, HJ (1993). “Design Criteria for Ground Improvement by Stone Columns”, *Proceedings of the 4th National Conference on Ground Improvement*, Lahore, Pakistan.