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Soil improvement by stone columns-Interpretation of a local database of loading tests and Finite Elements Modelling

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ABSTRACT: The aim of this paper is to present a detailed analysis of a local database of full-scale loading tests of single stone columns built from many projects of soil improvement carried out in Algiers and its neighbourhood. Focus is first made on the interpretation of the loading curves in order to derive the bearing capacity and the critical load and correlate them with the geotechnical properties. Furthermore, numerical modelling using the finite elements method is carried out by calibrating the experimental loading curves, in order to derive the mechanical parameters of the experimental stone column. The synthesis of the results obtained allowed suggesting a simple non-linear analytical approach for the calculation of the settlements of a single stone column.

1 INTRODUCTION

Although the soil improvement by stone columns is commonly practised in the construction projects, it is still a rather laborious research topic partly due to the inherent complexity of the natural soil behaviour in which another material is included, which results in a composite medium. The analysis of a representative number of experimental observations of the behaviour of the stone columns based on the vertical loading tests offers a pragmatic approach to study the key parameters governing such an interaction.

Northern Algeria is characterized by a high seismic activity and contains large areas of very poor soils consisting mainly of loose sands and/or soft soils. The intense activity of construction necessitates the use of sites exhibiting sometimes poor values of bearing capacity, unacceptable settlements or sometimes non-negligible potential of liquefaction in loose saturated sandy layers. Consequently, soil improvement techniques are usually undertaken to improve the bearing capacity of such soils, as well as to reduce foundation settlements, or to minimize the liquefaction risks. The vibro-flotation is the most commonly used improvement technique in Algeria, and hundreds of kilometers of stone columns are already installed within several sites. Monotonic vertical loading tests of stone columns are systematically carried out within the scope of the quality control of the production process of stone columns, which allows an easy empirical analysis of a local database and assessing the key parameters governing the experimental load-settlement response of single stone columns.

This paper aims at describing this database, presenting the methodology of analysis and outlining the main results of analysis of the experimental data as well as the calibration process undertaken by use of a finite elements modelling of the sites studied.

2 DESCRIPTION OF THE DATABASE

2.1 *Geotechnical context*

The database was built on the basis of the information collected from 9 sites located along the northern coast of Algeria as shown in figure 1. Due to their poor mechanical properties these sites were improved by inserting a convenient grid of stone columns by the vibroflotttation technique.

Comprehensive site investigations based on boreholes sounding, conventional laboratory tests as well as in-situ tests namely the pressuremeter test (PMT) and the cone penetration tests (CPT) carried out before the stone columns loading tests showed the sites A, B, F, G and I were composed of deep soft clayey deposits (CL-CH clays), whereas the sites C, D and I are bi-layered soils composed of CL clays underlying or overlying a layer of sand or gravely sand. At last, the site H is a tri-layered soil composed of a sandy deposit thick of 9 m overlying rollers and a deep horizon of marl.

Figure 1. Location of the experimental sites

Figure 2. Typical arrangement of the column loading test

2.2. *Description of the stone columns*

All the sites studied were improved by the vibroflotation technique using either the dry method in urban areas or by the wet method used exclusively outside agglomeration. Stone columns are compacted vibratory driven porous columns constructed by inserting ballasts into boreholes with diameters ranging between 0.6 to 1.1 m and slenderness ratios from 6.3 to 22. Improved area in these sites consists of a convenient grid composed of either squared or triangular elementary shapes.

2.3. *Stone columns loading test*

An overall of 93 full vertical loading tests of columns was carried out in order to check out the in-situ production quality, according to the French standard NF P94-150-1 as well as the French recommendations CFMS (2011).

As illustrated in figure 2, the axial load was provided by a double effect hydraulic jack connected to a piston pump, the counterweight being provided by the weight of the crane and the vibrator. The column head is simply instrumented by 4 dial gauges having a sensitivity of 1/100 mm for the top settlements reading, and a load cell to measure the vertical load.

Testing procedure comprised a series of 6 increments $Q_N/4$, $Q_N/2$, $3Q_N/4$, Q_N, $5Q_N/4$ and $3Q_N/2$, Q_N being the nominal vertical load corresponding to the column SLS (Serviceability Limit State). The increments were applied on an 80-100 cm diameter steel plate in contact with the column top, each load increment was usually maintained 1 hour or when the rate of top settlement v_0 is less than 2 mm/min (Kerkar, 2015).

2.4. *Presentation of results*

Among the diversity of criteria defining the limit vertical load or the axial bearing capacity from a simply instrumented column loading test, the hyperbolic criterion was chosen. In fact, for most of test columns the load-top settlement curve Q-v_0 was found hyperbolic shaped and fits well the following hyperbolic function with a least-squares regression coefficient R> 85% in most of cases:

$$\frac{v_0}{Q} = \frac{1}{K_{v0}} + \frac{v_0}{Q_l} \qquad (1)$$

where K_{v0} is the initial slope of the Q-v_0 curve and corresponds to the initial vertical column stiffness, and Q_l is the column bearing capacity. Figure 3 shows a typical hyperbolic fitting of the load-settlement curve to derive K_{v0} and Q_l.

The critical load Q_c (or creep load), defined as the threshold of instability of the column settlements with time, corresponds, as depicted in figure 4, to the intercept of the two straight lines describing the evolution of the rate of settlement, say b, with the applied load Q.

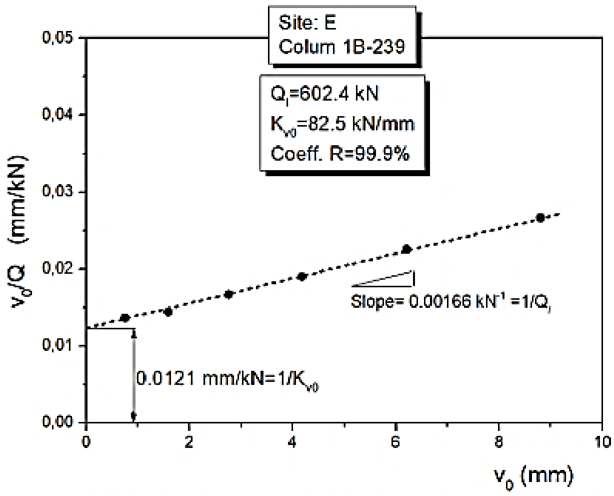

Figure 3. Hyperbolic fitting of a typical Q-v_0 curve

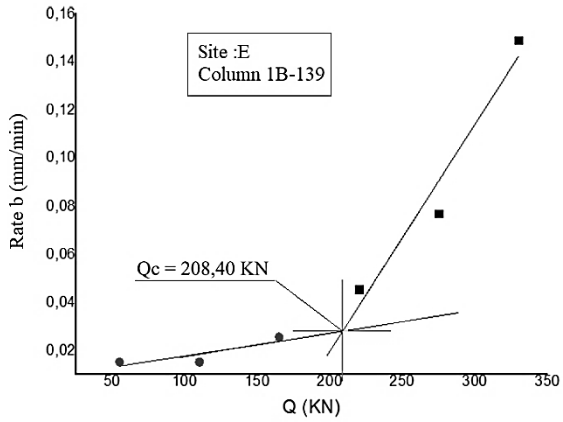

Figure 4. Geometrical determination of critical load Q_c

Test columns were subjected to large margins of maximum applied loads Q_{max} varying from 10 to 87.5% of the vertical bearing capacity Q_l derived from the hyperbolic criterion, which induced maximum values of the relative settlement v_0/B ranging from 0.4 to 8.8%, B being the column diameter. The response of the experimental columns was therefore well analysed up to large deformations highlighting inherent non-linear behaviour of the columns (Djerrouni & Touazi, 2018).

3 INTERPRETATION OF RESULTS

3.1 *Bearing capacity of the stone columns*

According to the literature, three possible failure mechanisms of the soil surrounding a single column, notably the punching shear (Fig. 5-a), the generalized shear (Fig. 5-b) and the lateral expansion or bulging failure (Fig. 5-c), the last one being the most adopted mechanism which the current design methods are based on to describe the failure due to long end bearing or floating columns (Barksdale & Bachus, 1983).

Within the scope of the rigid-plastic behaviour of the soil, adopting the bulging failure mechanism leads to the formulation of the vertical bearing capacity Q_l as function of the following parameters:

$$f(Q_l, R^*, B, D, A_c)=0 \quad (2)$$

where R^* denotes a net equivalent soil resistance having the dimension of a pressure and measured by an in-situ test. B, D and A_c are respectively the diameter, the embedded length and the cross-sectional area of the column (usually $A_c=\pi B^2/4$).

Dimensional analysis by using the Vashy-Buckingham theorem led to a simpler formulation using non-dimensional variables as follows:

$$g(K_r, D/B)=0 \quad (3)$$

K_r is a bearing capacity factor given by the following Equation, and D/B denotes the slenderness ratio of the column:

$$K_r=Q_l/(A_c.R^*) \quad (4)$$

According to Greenwood (1970), K_r is expressed as follows:

$$K_r=tg^2(\pi/4+\varphi_c/2) \quad (5)$$

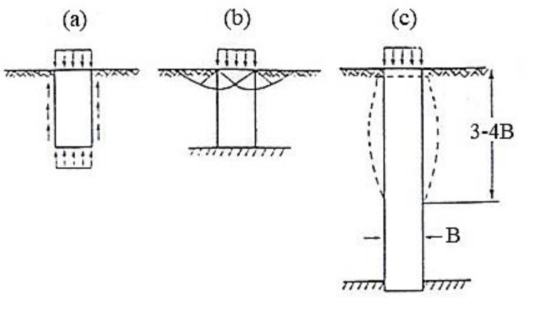

Figure 5. Usual failure mechanisms around a stone column

φ_c is the internal friction angle of the stone column taking typical values ranging from 38 to 45° depending on the degree of compactness. K_r may therefore be bounded to 5.8.

The pressuremeter test PMT and the cone penetration test CPT are usually modelled within the scope of the cavity expansion theory as stress paths involving local lateral expansion at the depth of test (Djoneidi & Frank, 1983) which exhibits some similarity to the mechanism of deformation by bulging aforementioned. The net resistance R^* may therefore be taken as an equivalent net limit pressure P_{le}^* derived from the PMT data.

Due to the local aspect of failure of the soil around the column, it is suggested to take the minimum value of P_{le}^* of N slices thick 2B along the column (CFMS, 2011). If z denotes the depth of the mid-height of the slice k, $P_{le}^*(k)$ is then calculated as a weighted average value:

$$P_{le}^*(k) = \frac{1}{2B}\int_{z-B}^{z+B} P_l^*(t)dt \quad (6)$$

$$P_{le}^* = min\{P_{le}^*(k), k = 1, N\} \quad (7)$$

P_l^* is the net limit pressure at the depth t equal to the following difference:

$$P_l^*(t)=P_l(t)-P_0(t) \quad (8)$$

$P_l(t)$ and $P_0(t)$ are respectively the measured limit pressure and the horizontal initial stress. In fine saturated soils, $P_0(t)$ is evaluated as follows:

$$P_0(t) = u(t) + K_0\, \sigma_{v0}'(t) \quad (9)$$

u, K_0 and σ_{v0}' are respectively the pore water pressure, the coefficient of pressures at rest and the effective overburden vertical stress.

When the CPT data is available, R^* is an equivalent net cone resistance q_{ce}^* which is obtained as the minimum value of q_{ce}^* of N slices thick 2B along the column:

$$q_{ce}^*(k) = \frac{1}{2B}\int_{z-B}^{z+B} q_c^*(t)dt \quad (10)$$

$$q_{ce}^*=\min\{q_{ce}^*(k), k=1,N\} \quad (11)$$

q_c^* is the net cone resistance at the depth t evaluated as follows:

$$q_c^*(t)=q_c(t)-\sigma_{v0}(t) \quad (12)$$

$q_c(t)$ is the measured cone resistance and $\sigma_{v0}(t)$ is the vertical overburden stress at the depth t.

For each loading test, the bearing capacity factor K_r was obtained from Equation (4) by dividing the bearing capacity Q_l, derived from the experimental load-settlement curve, by the resistance R^* and A_c, where R^* is $\{P_{le}^*\}_{min}$ or $\{q_{ce}^*\}_{min}$ depending on which in-situ test is available. Figure 6 and 7 illustrate the experimental values of K_r as function of D/B based on PMT test or CPT test, K_r being therefore respectively denoted K_p or K_c.

As shown in figure 6, experimental values of K_p linearly vary as function of D/B, but exceed the limit value given by Greenwood, which led to suggest a simplified chart where K_p is bounded to 5.8 for D/B greater than 5.6, corresponding to long columns. Such a conservative estimate of K_p would lead to an evaluation of Q_l on the safe side.

3.2 *Vertical stiffness of the stone columns*

Within the domain of small settlements v_0 of a stone column subjected to a vertical load Q, the vertical stiffness K_{v0}, which is hence equal to Q/v_0, may be formulated as function of the following parameters:

$$f(Q, v_0, E_M^e, B, D)=0 \tag{13}$$

where E_M^e denotes the equivalent pressuremeter modulus along the stone column.

According to the theorem of Vashy-Buckingham, this Equation may be simplified through a dimensional analysis involving the following non-dimensional variables:

$$g(K_s, D/B)=0 \tag{14}$$

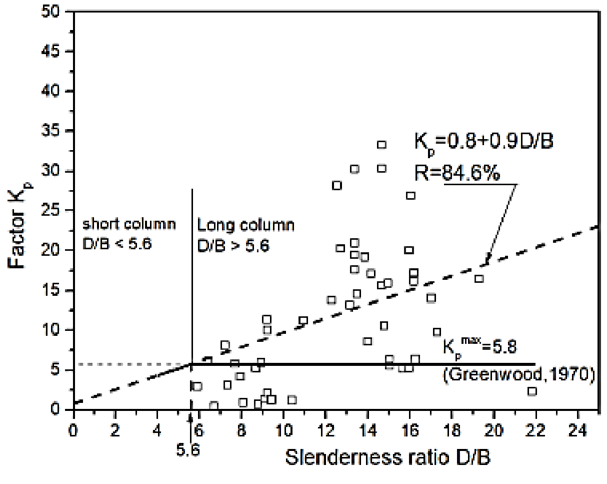

Figure 6. Bearing capacity factor K_p versus D/B

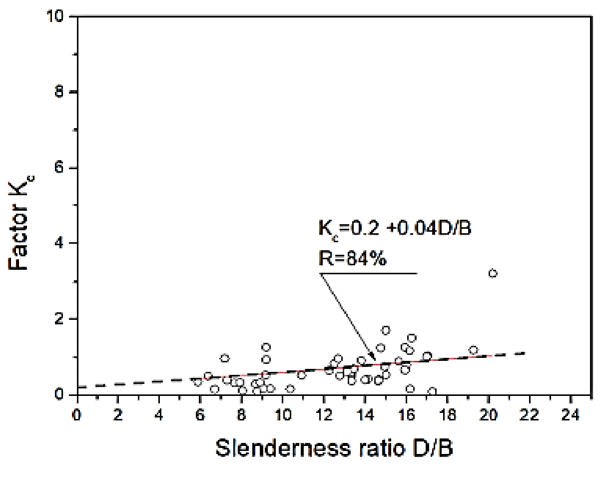

Figure 7. Bearing capacity factor K_c versus D/B

K_s is a settlement factor given by the following Equation:

$$K_s=K_{v0}/(E_M^e.B) \tag{15}$$

The equivalent pressuremeter modulus is defined as a weighted average along the stone column:

$$E_M^e = \frac{1}{D}\int_0^D E_M(t)dt \tag{16}$$

As illustrated in figure 8, no obvious correlation is noticeable between K_s and D/B. However, histogram of K_s shows a Gaussian distribution with a characteristic value of K_s equal to 10.9. The initial stone column stiffness may therefore be formulated as:

$$K_{v0}=10.9E_M^eB \tag{17}$$

Regarding the CPT test and following the same methodology of analysis, it was found the stiffness K_{v0} linearly increases with the equivalent net cone resistance q_{ce}* along the stone column. The equivalent cone resistance is defined as a weighted average value along the stone column:

$$q_{ce}^* = \frac{1}{D}\int_0^D q_c^*(t)dt \tag{18}$$

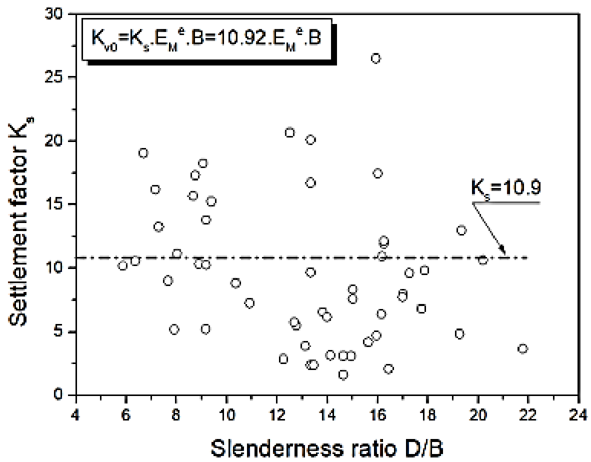

Figure 8. Settlement factor K_s versus D/B

3.3 *Critical load (or creep load) Q_c*

As shown in Figure 9, it was found the critical load Q_c is proportional to the bearing capacity Q_l, exhibiting a simple linear relation as follows:

$$Q_c=0.384Q_l \tag{19}$$

Since Q_c is sometimes interpreted as the threshold of the linear portion of the load-settlement curve, the ratio Q_l/Q_c is hence equivalent to an overall safety factor approximately equal to 3. Moreover, compared to that of displacement piles, the ratio Q_c/Q_l of a stone column is 0.38 whereas that of pile is 0.70 according to the French code, say about a half one.

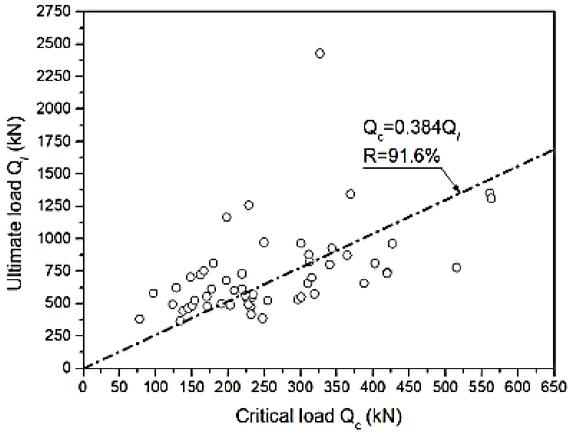

Figure 9. Chart of Q_c versus Q_l

4 PROPOSAL OF A NON-LINEAR ANALYTICAL APPROACH

4.1 *Methodology of calculation*

The suggested values of the factors K_r and K_s allow using the Equations (4), and (15) to compute le top settlement v_0 of a single stone column subjected to a vertical load Q based on a simple non-linear analytical approach, able to simulate the whole load-settlement curve. The methodology of calculation is outlined as follows:

1. Evaluate the equivalent net resistance R* using the PMT or CPT whichever is available, according to Equations (6) through (9) or (10) through (12),

2. Determine the bearing capacity factor K_r either from Figure 6 or 7 and compute the bearing capacity Q_l from Equation 4,

3. Compute the initial stiffness K_{v0} from Equation (17),

4. Compute the top settlement v_0 from Equation (1), this latter being transformed into the following Equation:

$$v_0 = \frac{\frac{Q}{K_{v0}}}{1-\frac{Q}{Q_l}} \qquad (20)$$

4.2 *Internal validation of the approach*

In order to demonstrate the capabilities of the suggested approach, process of internal validation was undertaken by predicting the load-settlement curves of the experimental stone columns used in the interpreted database, and comparing them with the experimental curves.

In Figure 10 showing a comparative example, very good agreement between the simulated curve and the experimental curve can be noticed, demonstrating this simple analytical approach may be a practical tool for estimating the stone settlements under working loads.

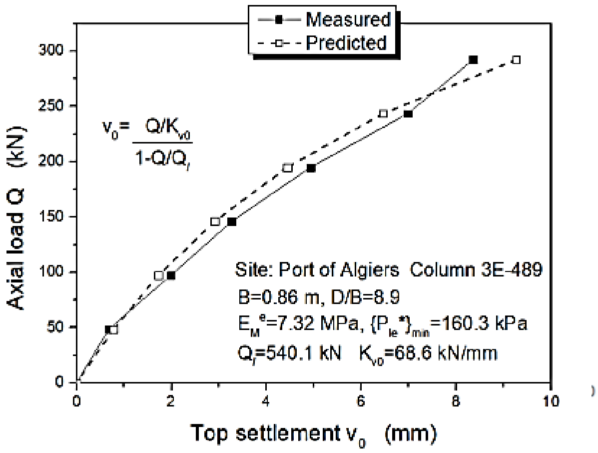

Figure 10. Comparison of measured and predicted settlements

5 FINITE ELEMENTS MODELLING

5.1 *Description of the FEM model*

The FEM software Plaxis v8.2 was used to simulate the response of a single stone column under an increasing axial load by assuming the stone column as well as the surrounding soil as elastic plastic materials obeying the Mohr-Coulomb's failure criterion. As shown in Figure 11, slip elastic-plastic elements were included at the stone/soil interface to describe a more realistic sliding of the stone column elements with respect of those of the soil.

Calibration of the load-settlement curve of all the experimental stone columns was undertaken based on this FEM model in order to derive the values of the mechanical properties of the stone columns, namely the elastic modulus E_c and the friction angle φ_c. E_c was varied to simulate the small strains domain, whereas φ_c was varied to study the response at large strains (Belkahla & Benkreira, 2017).

It was found these two parameters have significant effects on the load-settlement curve with relatively large margins, namely 35-110 MPa for E_c and 14-50° for φ_c, most likely depending on the degree of compactness of the stone column, which is in contrast with typical values recommended in the literature like those of CFMES (2011): 60 MPa for E_c and 38-41° for φ_c (Dhouib & Blondeau, 2005).

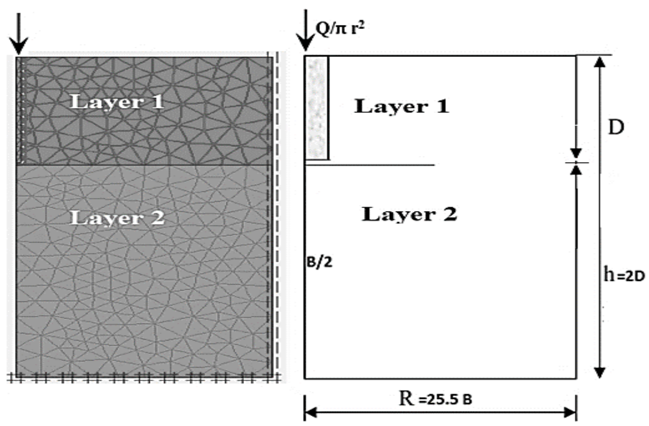

Figure 11. Typical FEM mesh for a bi-layered soil

6 CONCLUSIONS

This paper focuses on the single stone columns behaviour under axial loads by analysing a local database built on the basis of the information collected from 9 sites located along the northern coast of Algeria. After a brief description of this database as well as the methodology of analysis, the experimental results were interpreted and a simple non-linear analytical approach characterized by factors of bearing capacity and settlement, in correlation with the PMT or CPT data, was suggested.

Process of internal validation was used by comparing the experimental load-settlement curves to the ones obtained during the axial full-scale test. Good agreement was found but further improvements of the coefficients K_r and K_s followed by an external validation, by testing the predictive capability of such an approach on other stone column out of the database, will allow to recommend it for practical use.

Calibration process of the load-settlement curve of a stone column was undertaken by using an axisymmetric FEM model for all the stone columns tested, and showed a non-negligible effect of the mechanical parameters of the stone column, namely the elastic modulus and the internal friction angle, as well as with relatively large margins, namely 35-110 MPa for E_c and 14-50° for φ_c, most likely depending on the degree of compactness of the stone column.

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