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Comparison between theoretical procedures and field test results for the evaluation of installation effects of vibro-stone columns

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ABSTRACT: Several theoretical procedures to estimate the soil improvement produced by installation of vibro-stone columns are described. Particularly, finite element model and analytical solutions of a cylindrical cavity expansion were compared with results from an actual field test which was performed in silty sand and clayey soil treated with a column group. The results show that after dissipation of pore pressure the installation effects produce considerable improvement due to a large increase of the horizontal effective stress and due to densification process of sand. The load settlements response of the tested column group has been analyzed and compared with theoretical estimation of the improvement with and without installation effects, and with Priebe's analytical solution. It is observed that the column group installation effects have an important influence that should be evaluated with more advanced modelling or directly with in situ testing.

1 INTRODUCTION

1.1 *Soil grain size influence to installation effects*

The main goal of the vibrocompaction method is to increase the relative density of soil (Dr). To allow the adequate transmission of vibratory energy it is necessary to fluidise the soil by controlled water jets that increase pore water pressure and reduce the frictional contact between the soil particles. Then, after a certain time of compaction, rearrangement of the particles in a denser state will be achieved. More details related to vibrocompaction and its effects can be seen in the literature (Sonderman & Wehr 2004, Greenwood & Kirsch 1984, Slocombe et al. 2000, Kirsch & Kirsch 2010). Fines content superior of 10% (Degen 1997) and permeability lower than 10^{-3} cm/s (Greenwood & Kirsch 1984) are considered as limits for the soil densification by the application of vibrocompaction due to strong damping of vibratory energy and time effects. Vibro-stone columns represent a technological development that has allowed extension of the application limits of deep vibration techniques to fine cohesive soils (Kirsch & Kirsch 2010).

1.2 *Assessment of installation effects*

Standard procedures for the stone column design commonly do not take into account any installation effects. However, densification effect could be considered in design on the bases of the properly evaluated final soil characteristics (Priebe 1995). Experience has demonstrated that evaluation of the compaction suitability is not related only with the grain size distribution but also with an indicator of soil strength, e.g. cone penetration resistance. In this sense, Figure 1 shows empirical relation proposed by Massarsch (1991), for the evaluation of compaction suitability as a function of friction ratio and cone penetration tip resistance.

Several authors analyzed effects of the vibro-stone column installation based on field

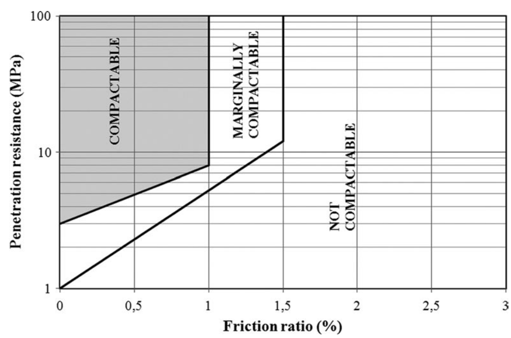

Figure 1. Compactability of soils. Massarsch (1991).

measurements (Watts et al. 2001, Kirsch 2006, Gäb et al. 2007, Castro 2008), but related to a specific cases and hence, cannot be generalized in a straightforward manner (Castro & Karstunen 2010).

On the other hand, there have been various attempts with different approaches for theoretical modelling (Kirsch 2006, Elshazly et al. 2007, Guetif et al. 2007, Castro & Karstunen 2010, Castro et al. 2012). In general, field measurements and theoretical approaches have shown that installation of vibro-stone columns in saturated clays cause its remolding close to the vibrator, and produces increase of pore pressures and horizontal stress. However, after a relative short period of consolidation, soil tends to gain in stiffness and additionally provides greater confinement to the columns. Whereas, in intermediate soils with variable fines content such as silty sands or sandy silts, vibrations could also achieve a significant densification by rearranging the soil particles into a denser state. This would be the case of soils in the marginal compactable zone shown in Figure 1.

In the following sections some theoretical procedures that are considered as reasonably suitable to evaluate the vibro-stone column installation effects will be described and compared with field trial measurements obtained during column installation.

2 MODELLING PROCEDURES

2.1 *Vibro equipment performance*

A rigorous modelling should consider both, the actions imposed by vibrator and the soil response. Due to the complex interaction between vibrator and soil, densification modelling of vibro techniques is still a challenging task (Heibrock et al. 2006). However, various attempts have been done by several authors (Fellin 2000, Cudmani et al. 2003, Arnold et al. 2008, Arnold & Herle 2009). Ranges of vibrators basic characteristics, and ones usually considered for modelling are shown in Table 1.

Usually dynamic effects are substituted by quasi-static loading; one-dimensional or 2D axisymmetric conditions are often adopted, as well as dashpot-spring and full numerical models. Furthermore, in order to simulate the shear waves induced into the ground, hypoplastic model has been considered, which fits more accurately to the sand behavior. In general, results obtained from models are in accordance with the experiences on actual observed effects, stressing the importance of the compaction time in order to achieve better results. It is also important to emphasize the model performed by Arnold et al. (2008), which confirms the existence of a zone with low densification immediately close to the vibrator, a second zone with best compaction results that extends from approximately 0.5 to 3 m from vibrator axis, and a third zone at a distance of more than 3 m, where soil compaction does not occur.

Mentioned findings are directly related to the vibrocompaction process and may be applied to vibro-stone column installation for predominantly sandy soils. For predominantly fine grained soils the densification is insignificant using vibrocompaction.

Table 1. Vibrator characteristics.

Freq. (Hz)	Amplitude (mm)	Horiz. force (kN)	Weight (kN)	Diameter (mm)
25–60	6–50	150–700	15–45	300–500

2.2 *Cavity expansion*

As it was mentioned, if vibro-stone column are performed in saturated clays the vibration energy has a negligible effects on the densification of the natural soil, but effects caused by the displacement of soil due to column installation can be evaluated by the application of the cylindrical cavity expansion theory. As columns are built in a reasonably short period the undrained conditions should be considered even for relatively permeable soils, e.g. sandy silts. Due to difficulties to determine the internal cavity pressure produced by the vibrator it is more suitable to simulate column expansion considering radial displacement (Castro 2008).

Egan et al. (2008) summarized the development of analytical solutions in three aspects: soil behavior, consideration of finite or infinite medium, and the influence of the initial radius of the cavity. Concerning mentioned aspects there are several solutions proposed by various authors, e.g. Vesić (1972), Carter et al. (1986), Yu & Houlsby (1991), Yu (2000). For complex soil models numerical analysis very frequently is selected as more adequate, but large strains should be enabled and considered.

In any case the key aspects to be analyzed will be the rapid increase of pore pressure and its dissipation over time, as well as the final stress state in terms of the at rest earth coefficient K_0.

2.3 *Stress distribution*

When the cavity expands to a certain radius, a plastic boundary is obtained at a radial distance ρ_F measured from the cavity axis. Stresses between cavity and ρ_F are in the plastic zone, whereas beyond ρ_F stresses are in the elastic zone. Thus, the radial stress p_F at point ρ_F represents fully plastic behavior. For a cohesive and frictional soil in an infinite medium and assuming Mohr Coulomb behavior Baguelin et al. (1978) proposed the expression (1) to determine the plastic radius, based on the assumption of no volume change and considering total stresses:

$$\frac{a^2}{\rho_F^2} = \frac{(p_0 + c \cdot \cot\phi) \cdot \sin\phi}{G} \tag{1}$$

where a is the radius of the cavity; p_0 is the pre-existence horizontal stress; and G is the shear modulus. ϕ is the angle of internal friction and c the cohesion of the soil.

It can be noted that for purely cohesive clay with $\phi = 0$, or adopting Tresca criterion, equation 1 turns into equation 2, which is the same solution developed by Randolph & Wroth (1979) for large strains:

$$\frac{a^2}{\rho_F^2} = \frac{c_u}{G} \tag{2}$$

where c_u is the undrained shear strength.

Since plastic radius is known, and combining the equilibrium equation of cylindrical cavity expansion with Mohr Coulomb failure criterion, the solution for distribution of major principal stress σ_r in the radial direction within the plastic zone can be obtained for purely cohesive soil with equation 3, and for cohesive-frictional soils with equation 4, as a function of radial distance ρ.

$$\sigma_r = p_F + c_u \cdot \ln\frac{\rho_F}{\rho} \tag{3}$$

where $p_F = p_0 + c_u$

$$\sigma_r = (p_F + c \cdot \cot\phi) \cdot \left(\frac{\rho_F^2}{\rho^2}\right)^{\frac{1-K_a}{2}} - c \cdot \cot\phi \tag{4}$$

where $p_F = p_0 \cdot (1 + \sin\phi) + c \cdot \cot\phi$; and $K_a = \tan^2(45\text{-}\phi/2)$.

Whereas in the elastic zone the well-known solution can be used to determine radial stress distribution:

$$\sigma_r = p_0 + (p_F - p_0) \cdot \frac{\rho_F^2}{\rho^2} \tag{5}$$

2.3.1 *Pore pressure*

Considering that excess pore pressures are caused only by the variation of mean total stresses, all pore pressure increments take place only in the plastic zone. In the same way, if we take into account the equilibrium equation of cylindrical cavity expansion and adopting zero pore pressure in the elastic zone as a boundary condition, it can be obtained the equation 6 to determine de excess pore pressure distribution as function of ρ.

$$\Delta u = 2c_u \ln \cdot \left(\frac{\rho_F}{\rho}\right) \tag{6}$$

2.3.2 *Influence of soil constitutive model*

In the previous sections the elastic perfectly-plastic models have been applied in order to highlight the possibilities of available and relatively simple analytical solutions to reach a simple approach of most important effects of vibro-stone columns installation. In fact, saturated clays with low sensitivity may be well modelled with these approaches.

With hardening plastic behavior a first enhancement of the modelling could be made considering the stress state modification due to plastic strains and stress dependency of stiffness. This model is suitable for the application of both, sandy and clayey soils subjected to a cavity expansion, although calculations usually have to be done with numerical model.

On the other hand, Castro & Karstunen (2010) presented a numerical modelling of Bothkennar clay based on S-CLAY1 and S-CLAY1S, which are Cam clay-type models that consider anisotropy and destructuration. Results agree with practical experience, showing a great remolded zone close to vibrator. Authors recommend for practical purposes a reduction of 15%–20% of initial undrained strength for standard columns grid.

2.4 *Back-calculation procedures*

As it was stated by contractors experience and several field measurements, during vibro-stone columns installation a certain heave on the surface could appear (Egan et al. 2008), and is more important for closer distances between columns. It confirms that soil stress state varies, and from the heave measurement data it is possible to perform back-calculation of final soil characteristics.

The real scale load test on treated soil might be very useful for back analysis regarding the additional stiffness of the soil. This procedure can be used to estimate the installation effects of column groups, e.g. Kirsch (2006) proposed the evaluation of an enhancement zone placed around column group by means of stiffness variation and its comparison with measured settlements.

3 FIELD TEST

3.1 *Field measurements*

In order to evaluate the installation effects of vibro-stone columns executed with dry bottom feed method in a profile of intermediate soil i.e. silty sand and clayey silt, field test consisting of a group of 13 vibro-stone columns, 20 m to 25 m long and with diameter between 0.90 and 1.00 m was carried out. Instrumentation campaign was composed by 12 piezometers distributed along 4 lines and located at depths of 6 m, 10 m and 16 m each. To control vertical displacements and stresses 1 extensometer was extended up to depth of 40 m. Figure 2b shows a section and plan view of the field test. In order to compare measurements with theoretical procedures, the pore pressure was monitored during the installation of one column (see column A, Fig. 2b).

Piezometer radial distances from the axis of the column A can be seen in Figure 2b (distance of 1.25 m for piezometer Pz3, 1.8 m for piezometer Pz4, whereas both piezometer Pz1 and Pz2 were set at radial distance of 5 m).

The first evaluation step of the effects of columns installation can be seen in Figure 2a. Typical cone penetration resistance between columns increase 8 to 14 times comparing with tip resistance before the treatment. From the combination of friction ratio FR% and tip resistance it can be observed that soil most probably experienced a certain improvement due to densification according to the marginal compactable zone shown in Figure 1.

Regarding excess pore pressure, in Figure 3 are presented both, the peak values measured by piezometer and its comparison with the theoretical predictions that are exposed in next chapter.

Maximum values of 65 kPa and 110 kPa were measured at depths of 10 m and 16 m respectively, and at radial distance of 1.8 m. At a radial distance of 5 m were observed peaks values of 50 kPa and 68 kPa, probably related to the presence of sand layer that is able to transmit the generated vibrations.

Figure 4 shows that after column installation the excess pore pressure is rapidly dissipated and after 5 hours (300 minutes) almost no excess pore pressure was detected. Moreover, the same Figure shows that consolidation rate estimated with finite element method reasonably agrees with the measured consolidation rates.

3.2 *Modelling of field measurements*

Finite element modelling with Plaxis v8 code was performed and results were compared with analytical solutions presented in chapter 2.2. Modelling comprises an axisymetric model of 4 soil layers with Mohr-Coulomb behavior and extrafine mesh of 15-noded elements close to the column axis. The whole geometry is indicated in Figure 2c.

Column expansion was modeled by a prescribed displacement. According to Carter et al. (1979) the expansion from a finite radius can be related to the idealized expansion on an infinite medium where radius starts from 0, by the relationship $r^2_{f,fin} - r^2_{0,fin} = r^2_c - 0$. Therefore, adopting initial radius $r_{0,fin}$ of 0.1 m and actual column radius r_c of 0.55 m, the magnitude of expansion to be considered is $r_{f,fin} - r_{0,fin} = 0.46$ m.

To enable the soil free movements roller boundaries were assumed on all sides. Soil properties are

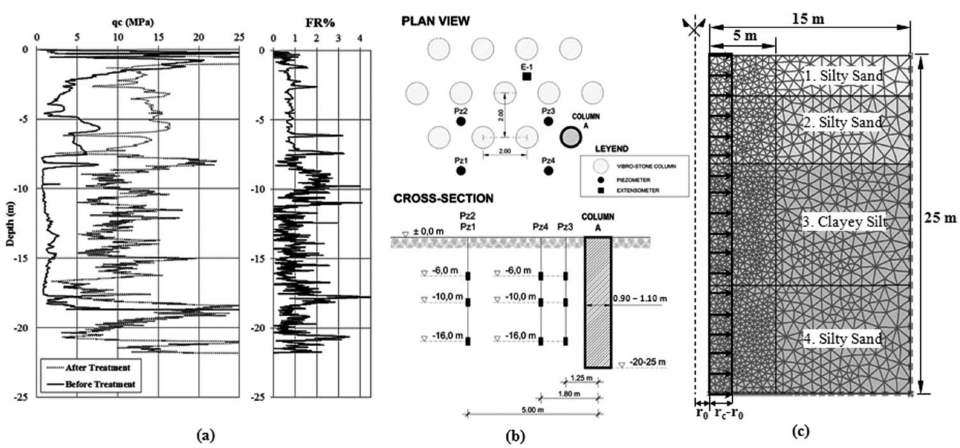

Figure 2. Field test characteristics, (a) cone penetration resistance after and before the treatment, (b) plan view and section of field test and (c) geometry of finite element modelling.

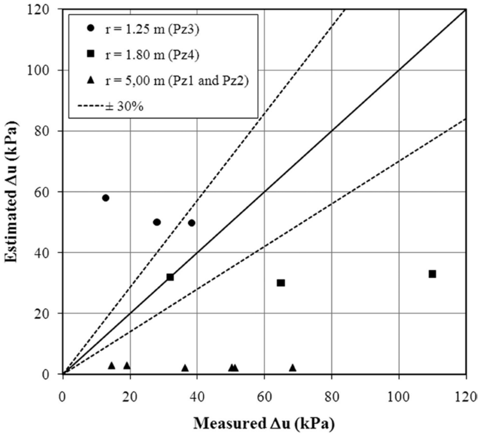

Figure 3. Predicted and measured peaks of excess pore pressures.

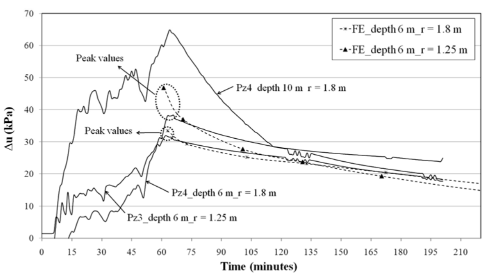

Figure 4. Measurements of excess pore pressure dissipation.

listed in the Table 2. The water table was situated at 1.20 m depth. Because the modelling is aimed to study the effects on the soil, the column was considered only as a void with infinite permeability. It was adopted Plaxis K_0-procedure corresponding to normally consolidated soil.

Calculations were performed in two phases, with undrained conditions for the first phase and consolidation analysis for the second phase. The "up-dated" option of Plaxis software was activated to take into account large strains.

Figure 5 shows the distribution of initial excess pore pressures Δu after cavity expansion, obtained at the same depths of piezometers. It is observed that results from finite elements modelling and the analytical solution (equation 6) are quite similar. The undrained shear strength, adopted to estimate the plastic radius ρ_F, was determined combining equations (1) and (2) and considering effective stresses and effective soil parameters (c', ϕ'). Results show that the plastic radius ρ_F coincides with the influence zone of Δu, and its values decrease with depth, reaching approximate radial distances of 4 m, 3 m and 2.5 m at 6 m, 10 m and 16 m depths respectively. It is also in accordance with the increase of undrained shear strength c_u, while the maximum values of Δu increase with depth. At a radial distance close to 1.75 m the pore pressures are almost equalized at the three analyzed depths (6 m, 10 m and 16 m), and the Δu values that fits best with theoretical procedures are close to this zone, near normalized radial distance of 4 column diameters.

On the other side, Figure 6 shows the results from finite element method for the distribution of normalized effective mean stress p and normalized coefficient of earth pressure K at the end of consolidation. Therefore, it is supposed that the stress state associated to increased K and p values will be suitable to estimate the stiffness generated due to column installation. Although the increase of the stiffness is dominated by the great increase of the radial stress, the reduction of the circumferential stress in the plastic zone has to be taken into account.

Consequently, for the estimation of the enhanced modulus of soil, instead of the coefficient of lateral earth pressure K, the mean effective stress should be considered. In general, a power law in the form $E = E_0\,(p'/p'_0)^m$ might be used. However, K is still the best indicator to assess the extent to which the soil stiffness increases due to columns installation.

Figure 6 also shows that the zone influenced by $K/K_0 = 1.5$–2 could be very suitable for the estimation of the stiffness increase. This zone is in between 4 and 6 column radii at 16 m depth, while at 6 m depth is between 10 and 14 column radii.

It is important to emphasize that in the analyzed case less difference between K/K_0 and p/p_o was observed with the increase of the depth.

3.3 *Field load testing*

An equivalent load of 75 kPa was placed above the field-test site shown in Figure 1b. Loading was made of 4 m height backfill material, widely larger than the testing zone. In Figure 7 the load settlements behavior of a group composed of 13 columns are indicated.

Several theoretical estimations are compared with the actual measurements, taking into account the situation without improvement and situations considering the improvement with and without any installation effects.

Thus, a settlements estimation with coefficient of lateral earth pressure at rest K_0 is compared with the case of improved lateral earth pressure $K^* = 1.75$ according to the modelling results shown in Figure 6, and with the analytical solution proposed by Priebe (1995) which represents the case of $K = 1$.

Table 2. Soil properties.

Layer	Depth (m)	γ' (kN/m³)	c' (kPa)	ϕ' (°)	E_{oed}* (kPa)	k_h (m/s)
1. Silty sand	0–3	8.50	1	33	20,000	$1.15 \cdot 10^{-8}$
2. Silty sand	3–8	8.50	5	25	4500	$1.15 \cdot 10^{-8}$
3. Clayey silt	8–17	8.50	10	17	3600	$2.30 \cdot 10^{-9}$
4. Silty sand	17–25	8.50	5	28	7200	$1.15 \cdot 10^{-8}$

*confined modulus.

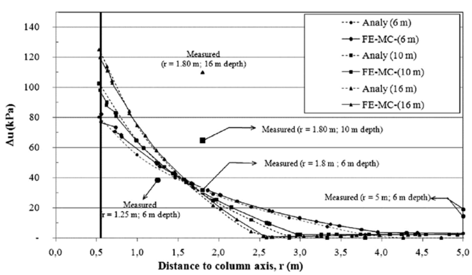

Figure 5. Estimation of excess pore pressures due to vibro-stone column installation.

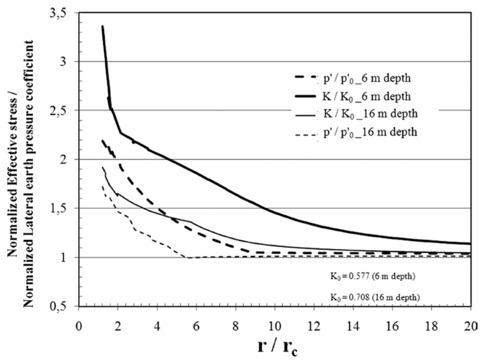

Figure 6. Distribution of effective mean stress and coefficient of lateral earth preasure.

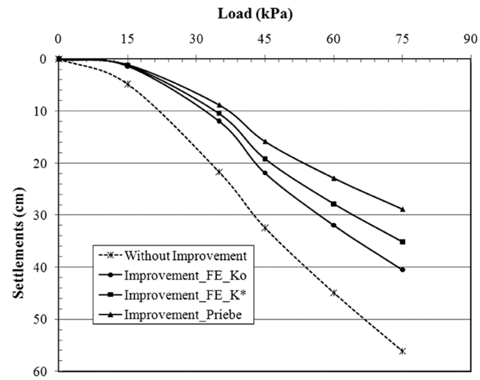

Figure 7. Load settlements results of field test.

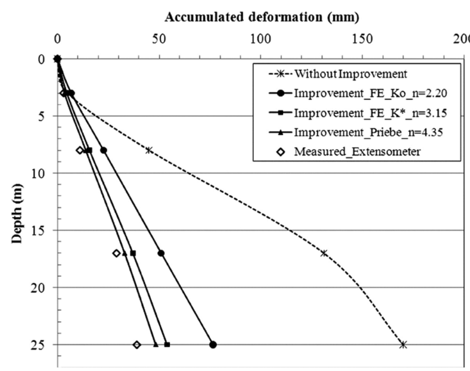

Figure 8. Accumulated compressibility results of field test.

Figure 8 shows the accumulated compressibility monitored by the incremental extensometer installed in the middle of field test, together with the theoretical estimation of improvement considering the same cases described in Figure 7. During the load testing the extensometer only provided data up to a maximum foundation load of 35 kPa.

The reduction of settlements obtained with the consideration of K* improved by cavity expansion is about 20–30% greater than the case of improvement without any installation effects (K_0). The results closer to the actual measurements are those estimated according to Priebe (1995).

Furthermore, as have been stated by Kirsch (2006), the settlement reduction in terms of improvement factor "n" depends on the load level, which can be noted comparing results from Figures 7 and 8. It has to be stressed that the difference between actual measurements and theoretical estimation could be attributed to the global installation effects of the column group, possible densification effects and the global increase of horizontal stresses. The evaluation of these global effects should be made with more advanced modelling or directly by means of in situ testing.

4 CONCLUSIONS

Installation effects should be considered in the design of vibro-stone column treatments, and can be estimated according to cavity expansion theory with reasonable accuracy. The improvement induced by individual column installation can be expressed by the increase of coefficient of earth pressure. For the evaluation of global effects induced by a column group installation, advanced modelling should be performed to consider more realistic characteristics of vibro-stone column execution and its effects.

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