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# Improvement of loose sandy soil foundation by compaction piles

## Amélioration de sols de fondation sablonneux lâches pour pieux de compactage

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ABSTRACT
Some case histories of densification of loose sandy soils using compaction piles are analyzed. The sandy soils were improved to support heavily loaded structures using shallow foundations. The technique used was deep compaction through compaction piles driven with Franki -type equipment, and shallow compaction by vibratory plate. Results of the compaction processes are presented and discussed to provide guidance for future projects. The analysis includes distance from the compaction pile, initial relative density, time delay for results verification after compaction, and depth. The results of densification demonstrate the method is efficient.

RÉSUMÉ
Quelques cas historiques de densification de sols sablonneux lâches sont analisés. Les sols sablonneux était amélioré pour supporter des strutures lourdement chargée avec semelles. La technique usée était compactage profonde avec pieux de compactage battus avec équipement employé pour battage de pieu Franki et compactage superficiel avec plaque vibratoire. Les resultats des conduites de compactage sont presentés et examinés pour directionment de oeuvres futures. L'analyse considére distance du pieu de compactage, compacité initial, le temp aprés compatage pour verification de resultats, et hauteur. Les resultats de densification confirme la efficacité de la méthode.

## 1 INTRODUCTION

Soil improvement techniques are extensively used to avoid many of the settlement and stability problems associated with loose sand deposits. Over the past 10 years, considerable experience and data have been obtained in the state of Espirito Santo, ES, Southeastern region of Brazil, on the use of compaction piles with the purpose of densifying loose marine sandy soils (Bicalho et al., 2002; Bicalho et al., 2004a; Bicalho et al., 2004b; Bicalho and Castello, 2004). This paper analyses a database of some of these case histories. The technique used was deep compaction through compaction piles driven with Franki-type equipment, and shallow compaction by vibratory plate. The case histories were selected to verify the validity of the proposed loose sandy soil improvement technique. The use of the compaction piles resulted in schedule and cost savings in the projects. Today the improved sandy soils support heavily loaded structures using shallow foundations.

Compaction piles are not a deep foundation as a typical pile. They are made of sand or sand-stone mixture columns (the stone is used for drivability reasons only) driven in the loose sands. Compaction piles densify primarily by displacement, principally for the initial piles when the deposit is still loose, and in design this is the only factor taken in account. But at the final stages when the sand deposit is already partially compacted, vibration occurring during driving may be an additional important factor in improvement. Prevision of the effectiveness of compacting piling was made from the volume of sand (or sand/stone mixture) added (displacement of soil only), and its measurement was made with a comparison between the Standard Penetration tests (yielding the $N_{SPT}$ value), and/or Dynamic Penetrometer tests (yielding a dynamic point resistance, $q_d$) which also measure the vibration effect, before and after the soil densification. The analysis includes distance from the compaction pile, initial relative density, time delay for results verification after compaction, and depth.

## 2 DESCRIPTION OF SITE CONDITIONS

The cases studies have several common findings starting with the same geological origin. All sites are located on the coast of the state of Espirito Santo, Brazil. The region presents subsoil composed of marine sediments from the Holocene (quaternary age). The sand layers are generally heterogeneous with regard to relative densities, probably due to stratifications occurring during rise and fall of ocean level (transgression and regression) through the ages (Castello and Polido, 1988). The soil treated in the cases histories presented is a fine to medium quartz clean sand (less than 5% passing U.S. Standard sieve No. 200). The index properties for the treated sub angular sand are specific gravity 2.65, uniformity coefficient 1.65-3.35, curvature coefficient 0.3-0.7, dry unit weight 16.6 kN/m$^3$ (densest condition) and 13.6 kN/m$^3$ (loosest condition), Cordeiro (2004).

### 2.1 *Case Study No. 1*

This case is a 10-story apartment building located in the city of Vila Velha, Brazil. The soil profile primarily consist of 1 m of sand fill overlying 4 m of clean sand ranging in density from very loose to very dense. At the base of these superficial layers there is approximately 0.5 m of very loose organic clayey sand and then other layers that don't matter to this analysis. The ground water level at the test site, GWL, was encountered at a depth of about 1.3 m.

The density of the 4 m of clean superficial sand varies considerably with both depth and lateral extent. Therefore, the test program site was divided into two distinct test sites, a very loose to very dense sandy site (area 1), and a very loose to loose sandy site (area 2). The $N_{SPT}$ values (efficiency of about 70% accordingly to most measurements in Brazil) within the area 1 ranged from 4 to 10 blows per 300 mm for the first 2 m and below the 3 m depth, the $N_{SPT}$ values ranged from 13 to 80 and the $N_{SPT}$ values within the area 2 ranged from 4 to 10. An additional subsurface exploration program was done using dynamic penetrometer tests, PD. The results of SPT and PD established that the locally sand required in-place densification to minimize differential settlements of shallow foundations. The densification technique was required to improve the soil to support conventional footings for a design bearing pressure of 0.3 MPa. The footings have a minimum width (B) of 1 m and placed at an approximated depth of 1.3 m below the original ground surface.

### 2.2 *Case Study No. 2*

This project is an analysis of the foundation of a 6-story apartment building located in the city of Vitoria, capital of the state Espirito Santo, Brazil. Several soil borings were performed to determine the subsurface conditions. It was found that the soil profile consisted of 13 m of sand layers of different compactness degrees. The uppermost layer, approximately 6 m thick, consisted of loose to medium dense sand. Underlying the sand layer there is approximately 7 m of a medium to very dense sand. The GWL was encountered at a depth of about 3 m. A loose sandy soil improvement technique was required to support conventional footing for a design bearing pressure of approximately 0.3 MPa. The footings (B=1 m) were placed at an approximated depth of 3 m below the ground surface.

### 2.3 *Case Study No. 3*

The 59,550 $m^2$ shopping center is located in Vila Velha, Brazil. Highly variable subsurface conditions were encountered at the project site. Therefore, fifteen independent but associate facilities, 3970 $m^2$ of area each, were proposed for 59,550 $m^2$ property and various independent foundations systems were selected for each building area. The area of concern has an uppermost layer, approximately 0.5 m of debris, refuse, and fill. A layer of approximately 3 to 4 m of loose, saturated sand underlies this material. Underlying the sand layer there is approximately 11 m of a medium to very dense sand. The GWL was encountered at a depth of about 0.5 m. Improvement of the top 4 m of loose sand was required to minimize differential settlements between shallow foundations. The soil improvement technique was required to support conventional footing for a design bearing pressure of approximately 0.25 MPa. The footings (B= 1 m) were placed at an approximated depth of 2.5 m below the ground surface.

### 2.4 *Case Study No. 4*

These are two 10-story apartment buildings located in Vitoria, Brazil. The soil profile primarily consisted of 1.5 m of sandy clay fill overlying 8 m of clean sand ranging in density from loose to very dense and then other layers that don't matter to this analysis. The GWL was encountered at a depth of about 1.5 m during the investigation. The results of SPT and PD established that the locally clean sand required in-place densification to minimize differential settlements of shallow foundations. The densification technique was required to improve the sandy soil to support conventional footings for a design bearing pressure of 0.25 MPa. The footings (B= 1 m) were placed at an approximated depth of 1.5 m below the ground surface.

Evaluation of alternative foundation systems, which included deep foundations (which would pose problematic driving), led to the conclusion that the densification of loose marine sandy soils offered significant economic advantages. The technique used was deep compaction through compaction piles driven with Franki- type equipment, and shallow compaction by vibratory plate. Compaction piles are not a deep foundation as in a typical pile. They are made of sand or sand/quarried stone mixture columns (the quarried stone is used for drivability reasons only) dynamically densifying the loose adjacent sands. Compression is produced principally by sidewise displacement of the soil during ramming, but vibration also is produced and helps densification. The case histories were selected to verify the validity of the proposed loose sandy soil improvement technique.

## 3 COMPACTION PILE EXECUTION

The piles were made by using the usual Franki pile process (Tomlinson, 1995). This method of compaction by piles is to drive a withdrawable tube with 20 kN hammer having a 6 meters drop within the tube. The pipe casing of 400 mm diameter is longer than the length of the compaction pile so it protrudes from the ground. The blows of the hammer strike a plug formed of sand and gravel or quarried stone at the base of the tube, which is open at both ends. The plug under the blows of the ram pulls the tube down by friction. After the desired depth has been reached, the plug is forced out by holding the tube while tamping. More sand (or sand/quarried stone mixture) is fed into the tube and tamped while the tube is withdrawn gradually. This forms a dense sand (or mixture of sand and quarried stone) pile with enlarged base and, if desired, enlarged shaft at chosen depths. Soil compression is produced during ramming by vibration, but principally by sidewise displacement of the soil.

The volume of loose sand replaced by injected sand (or sand/quarried stone mixture) is one of the most important factors in improving weak ground using compaction piles. A check of the depth of base construction is done with the dynamic formula proposed Nordlund (1982) for conventional Franki-type piles. Based on Nordlund (1982), the number of blows needed to ram out the last unit volume of sand into the base, N, is:

$$N = \left( \frac{R_{ad} V_c K_s}{W H V_t^{2/3}} \right) \tag{1}$$

where Rad = the allowable working load on the base (50 KN for 400 mm diameter tube); Factor of safety = 3), $V_c$ = the unit volume of sand added into the base ($m^3$), $V_t$ = the total volume of sand in base ($m^3$), H = the height trough which drop hammer falls (6 m), W = the weight of drop hammer (20KN) and Ks = the soil empirical constant (Fine to medium sand, $K_s$ = 14).

Barksdale and Takefumi (1991) suggest that a column spacing s for a square sand compaction pile grid can be expressed by:

$$s = \sqrt{\frac{1+e_0}{e_0 - e_f} \frac{\pi D^2}{2}} \tag{2}$$

where $e_0$ = the initial void ratio of loose sand before improvement, $e_f$ = the final void ratio of loose sand after improvement and D = the sand pile diameter. They assume that the loose sand is densified due only to lateral displacement equal to the volume of sand pile added. It is neglected any increase in relative density of the loose sand due to vibration from pile construction, therefore, the value of s predicted by Eq. (2) is conservative. For central piles multiply Eq. (2) by 1.08.

In order to estimate the minimum value of s for the studied cases the values $e_{min}$ = 0.53 (densest condition) and $e_{max}$ = 0.86 (loosest condition) were applied to Eq. (2) leading to a value circa of 2.1 pile diameters for a square compaction grid (2.3 for central piles). Then, a sand column spacing (center to center) of about 2.5 pile diameters was originally proposed. However, the several assumptions used in Eq. (2) required that the final spacing between compaction piles be determined by a field testing program performed at the site. For better field control of effectiveness, piles were driven first at the corners of the rectangular footing foundation, then at the center. Prior to driving additional compaction piles at intermediate points on each side, the increase in sand density was evaluated through penetration resistance of soundings after densification. The most common performance criterion is the comparison between the initial and final penetration resistance. Measurement of the effectiveness of compacting piling was made from the volume of sand added, together with Standard Penetration tests, SPT, and Dynamic

Penetrometer tests, PD. The PD tests were performed in accordance with ISSMFE (1989).There are more than one procedure considered and accepted in the report, but in this specific case a hammer of 64 kg mass and a height of fall of 0.45 m is used to drive a pointed probe (cone) (Castello et al., 2001). The hammer strikes an anvil that is rigidly attached to extension rods. The penetration resistance is defined as the number of blows per 0.20 m penetration. The penetrometer shall be continuously driven into the subsoil. The number of blows should be recorded every 0.2 m. The test provides a dynamic resistance parameter, $q_d$, defined as:

$$q_d = \left( \frac{M}{M + M'} \frac{MgH}{Ae} \right) \tag{3}$$

where M = mass of the hammer (64 kg), M' = total mass of the extension rods, the anvil and the guiding rods (2.96 kg/m + 0.43 kg), H = height of fall (0.45 m), e = average penetration per blow, A = area at base of the cone (Cone diameter = 51 mm), and g = acceleration of gravity.

Since the $q_d$ value is not as well known as $N_{SPT}$ a correlation between both was developed. The data in Fig. 1 indicate no effect of the depth on the ratio $q_d$ (MPa)/ $N_{SPT}$ for the studied case. It can be seen that the average ratio of $q_d$ (MPa)/ $N_{SPT}$ for the studied sand lies between 0.5 and 1.0 (Fig. 1). An approximate correlation between the Dutch cone penetration resistance, $q_c$, and the dynamic resistance parameter, $q_d$, proposed by Waschkowski (1983) for loose to medium dense sands, is:

$$\left( \frac{q_d}{q_c} \right) \cong 1 \tag{4}$$

Assuming that Eq. (4) is applied to the studied sands, the data presented in Fig. 1 confirm previous studies addressing empirical correlations between the $q_C$ and $N_{SPT}$ (Danziger, 1982).

## 4 RESULTS AND ANALYSIS

Four case histories of densification of loose marine sandy soils using compaction piles of 400 mm diameter in the state of Espirito Santo, Brazil, are summarized in Table 2.

Because of the large number of parameters involved, it is not possible to present a thorough parametric study of the response of loose sand to the densification process. Consequently, to illustrate some of the features of compaction piles, comparisons between the $N_{SPT}$ and/or $q_d$ before and after the soil densification have been obtained for the studies cases. The analysis includes distance from the compaction pile, initial relative density, time delay for results verification after compaction, and depth.

PD test results before (initial PD tests, $q_{di}$) and after (final PD tests, $q_{df}$) densification by compaction piles are shown in Fig. 2, where $q_{di}$ is the initial dynamic resistance parameter for penetrometer tests, PD. Test results indicate that compaction piles increased the penetration resistance of loose cohesionless soil in general. The wide scatter of the data, however, precluded fitting any meaningful curve through the points. A clear trend how ever is displayed: the penetration resistance increased with the depth.

Fig. 3 presents the relationship between the original penetration resistance, $q_{di}$, and the penetration resistance increase, $K_m = q_{df} / q_{di}$, in points (Case Study No. 1). It can be seen that compaction piles tend to compact the loosest soils the most. The results tend to produce a more homogeneous density, reducing future differential settlement. On the other hand, one can imagine that compaction piles are useless and maybe detrimental in sandy soils above a certain penetration resistance (or relative density), which would vary with depth (local or general shear).

Table 2: Examples of compaction piles application

| Case study | s / D | $K_m$ | Reference |
|---|---|---|---|
| 01 (area 1) | 5-8 | 2-4 | Bicalho et. al. (2002) |
| 01 (area 2) | 3.5-4 | 2-10 | |
| 02 | 3.5-4 | 2-6 | Bicalho et al. (2004a) |
| 03 | 3.5-7 | 2-10 | Bicalho and Castello (2004) |
| 04 | 3.5-4 | 2-6 | Bicalho et al. (2004b) |

s = pile spacing (center to center); D = pile diameter; $K_m = q_{df} / q_{di}$

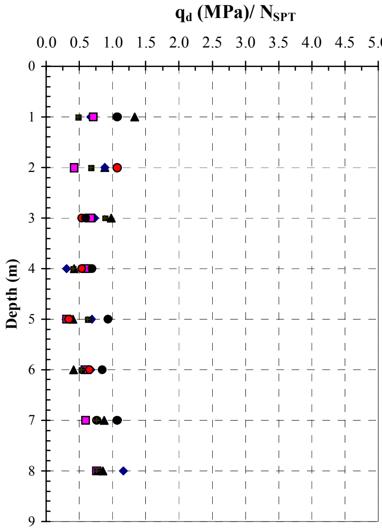


Figure 1. Variation of $q_d$ (MPa)/ $N_{SPT}$ ratio with depth Case Study No. 4 (Moraes et al., 2004).

Most tests were carried out during the period 1-10 days after densification, but the PD 1A test was carried at 40 days after densification. The test data clearly showed the penetration resistance increased with time after densification (Fig. 2). Penetration resistance, strength, and stiffness of sands increase with time after densification. The mechanisms responsible for time effects are not known (Mitchell and Solymar, 1984). It is recommended, therefore, to further investigate the effect of time on compaction piles.

The PD 2A, PD 2B and PD 2C tests were carried out at points located at central area of the test site, area 2, where better results were expected. The test data, as shown in Fig. 3, clearly confirmed this expectation. So most of the verification soundings were made at the periphery of the clusters of compaction piles. If the results were good there, they would be better at central points. Furthermore, the test data clearly showed that the 0.5 meter very loose organic clayey sand had been virtually fully compressed by dynamic replacement and mixed with sand (as in a hydraulic fracture) after the densification process. The method of dynamic replacement and mixing of organic soils with sand was investigated by Lo et al. (1990).

The effect of both depth and the original penetration resistance on the penetration resistance increase in points is shown in Fig. 4. The results show that the compaction piles are useless at shallow depths. The intend of pile densification was focused on depths of 2.5 m or more. Castello and Polido (1982) show that superficial cohesionless soils have low confinement, so when a pile is driven it displaces the soil to the ground instead of compacting it. For the upper meter of sand the improvement was obtained with vibratory plates with mass up to 400 kg. Results of the PD tests after shallow compaction are not presented in this paper since the purpose herein is to discuss the compaction pile technique only. Anyway the final PD tests results were $q_{df} > 15$ MPa. Fig. 4 shows also that the compaction piles tend to compact the loosest soils the most and the PDV 82, PDV 92 and PDV 93 tests located at central area of the test site presented better results.

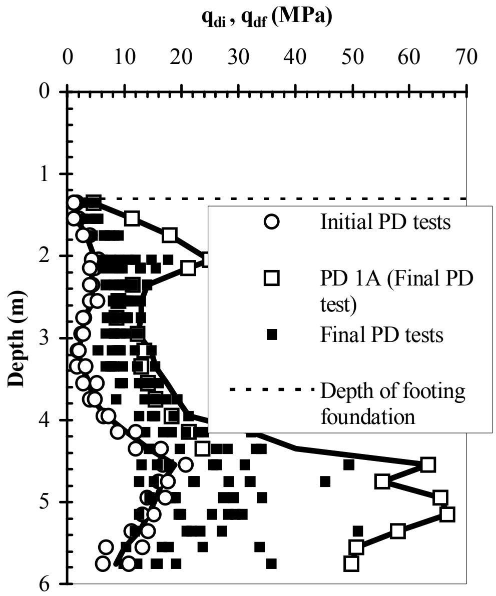


Figure 2. Penetrometer test results in area 1 before and after densification by compaction piles (Case study No. 1)

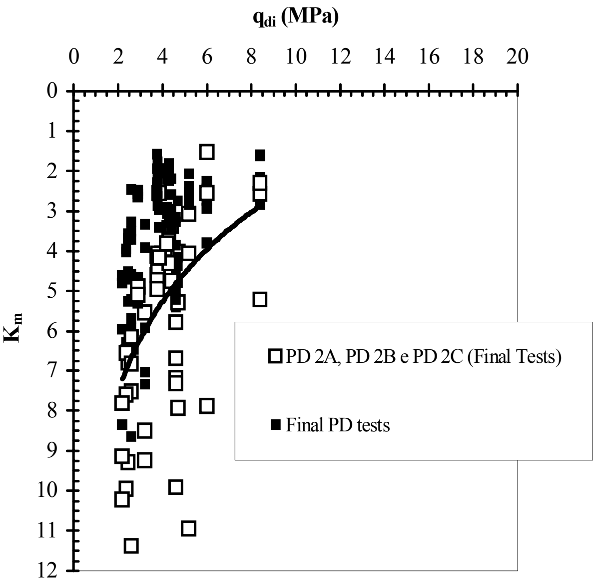


Figure 3. Variation of $q_{df}$ /$q_{di}$ ratio with initial Penetrometer tests (area 2) (Case study No. 1)

## 5 SUMMARY AND CONCLUSIONS

This study has shown the feasibility of using compaction piles executed with the Franki Process for the purpose of densifying loose cohesionless soil. Test results indicate that in general compaction piles of 400 mm diameter casings spaced on 1.4-3.2 m centers (3.5 to 8 pile diameters) can increase the penetration resistance of loose cohesionless soils by 2-10 times the initial penetration resistance. Loose cohesionless soils can be densified effectively using compaction piles.

It is shown that compaction piles tend to compact the loosest soils the most. The results tend to produce a more homogeneous density, reducing future differenttial settlement. As the sand density increases the effectiveness of the method decreases. The test data clearly showed the penetration resistance, which serves as a measure of the amount of densification obtained, increased with time after densification. If the penetration resistance is used as a basis for quality control of densification, then values measured at early times will be conservative.

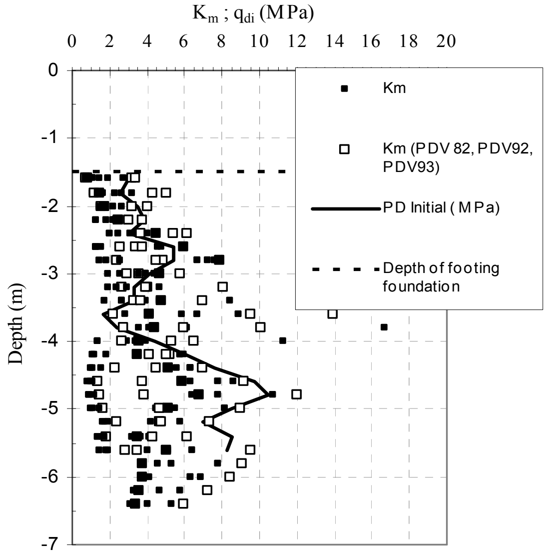


Figure 4. Variation of $q_{df}$/$q_{di}$ ratio with depth (Case Study No. 4)

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