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Stress increase induced by impact precast pile driving

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ABSTRACT: The paper describes the experimental activities carried out in a test site, set up to evaluate the increase of soil stiffness and horizontal effective stress, mainly in saturated low-medium density sandy layers and in silty sand, after the driving of tapered precast piles.

The experimentation consisted in driving some prefabricated tapered piles with different energies and spacing between them in an area where some soil layers had a high liquefaction potential. To evaluate the pile driving effects on the stress state around them, preliminary CPTu and DMT tests were carried out and repeated after the driving activity.

In particular, the change of the CPTu sleeve resistances was compared with that of the DMT K_D data, to evaluate the increase in horizontal stress using different methods, a phenomenon influencing the soil susceptibility to liquefaction and the pile bearing capacity.

1 INTRODUCTION

To reduce the potential liquefaction-induced settlement in cohesionless soils, many ground improvement techniques are adopted, among them vibrocompaction, rammed aggregate piers (RAP), stone columns, drilled displacement piles, driven displacement piles, deep dynamic compaction, and blast densification (Han 2015). In particular numerous studies have been performed to evaluate the ability of deep vibratory compaction (Van Impe et al., 1994, Massarch & Fellenius 2002, Massarch et al. 2020), of drilled displacement piles (Siegel et al. 2007, Siegel et al. 2008) and RAP (Rollins et al. 2021) to mitigate the risk of liquefaction in sandy soils, while few refer to the use of driven precast tapered piles.

With deep vibratory compaction cyclic stresses are generated in the ground, resulting in a denser particle arrangement and changes in effective stresses. The installation process of drilled displacement piles consists in displacement of soil and subsequent placement of fluid cement grout within the dislocated volume. For RAPs, the improvement mechanisms include increased lateral pressure and increased shear stiffness.

These processes can result in a measurable densification and in an increase in lateral stress. Therefore, these methods are used to mitigate cyclic liquefaction in sandy soils.

To recognize the deriving soil improvement, the most widely in situ tests used are the cone penetration test (CPT), CPT with pore water pressure measurement (CPTu), the seismic cone penetration test, the Marchetti flat dilatometer test (DMT) or the seismic dilatometer test (Mayne et al. 2009).

Indeed, the CPT and the DMT measured data enable to detect changes in strength, stiffness, and horizontal stress in the soil.

The installation process of driven precast piles determines, seemingly, similar phenomena, but there is a lack of knowledge about their quantification.

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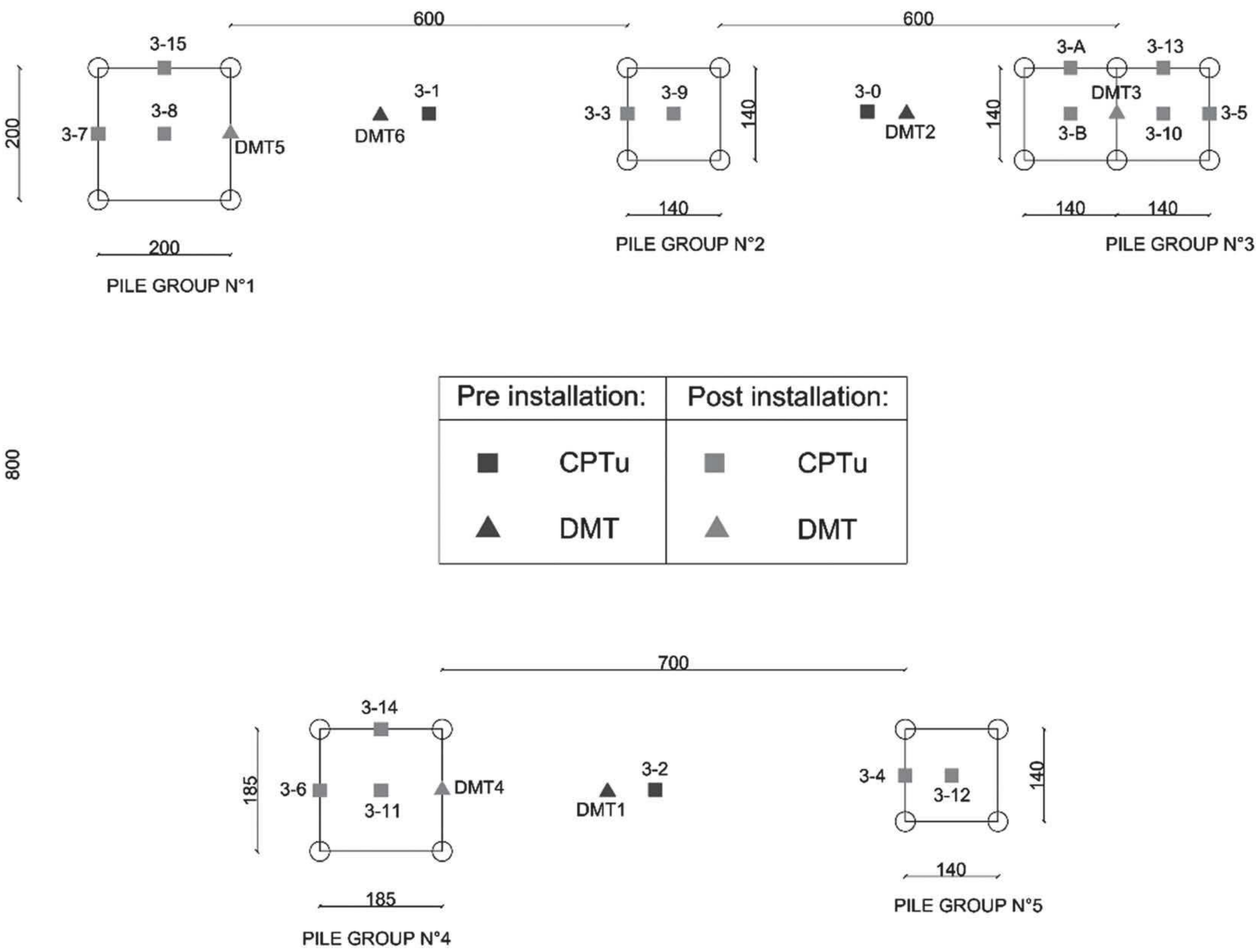

Figure 1. Configurations of pile groups and relative CPTu/DMT locations.

Therefore, the paper describes the experimental activities carried out in a test site, set up to evaluate the possible existence of these phenomena after the driving of precast piles.

The experimentation consisted in driving some prefabricated tapered piles with different energies and spacing between them in an area where some soil layers had a high liquefaction potential. To evaluate the pile driving effects on the stress state around them, preliminary CPTu and DMT tests were carried out and repeated at various distances from the pile center, after the driving activity.

One of the most interesting objectives of the research is to evaluate the change of the tip (q_t) and sleeve resistances (f_s) of the CPTu tests, comparing them with the data acquired using the DMT tests, principally the horizontal stress index K_D. Among the various phenomena influencing the soil susceptibility to liquefaction and the pile bearing capacity, the increase of horizontal stresses, resulting in a preloading effect, and its accurate determination could allow economic benefits in the design of deep foundations.

2 TEST SITE CHARACTERIZATION

2.1 *Soil characteristics*

The experimental site was chosen in the area where the most significant and widespread liquefaction phenomena had occurred during the 2012 Emilia seismic sequence. In particular the choice fell on an area near the Mirabello village (Ferrara, Italy), previously investigated in detail since a blast test was carried out in a neighboring area, in 2016 (Amoroso et al. 2017). During these previous studies, geological, geotechnical and geophysical characterization was carried out in proximity to the observed liquefaction evidence. The geotechnical in situ investigation included several standard penetration tests, piezocone tests, seismic dilatometer tests and deep boreholes, while geophysical tests included down-hole tests (DH1), MASWs (Multichannel Analysis of Surface Waves), P-wave and S-wave topographies, electrical resistivity topographies (Amoroso et al. 2017). During the new research further CPTu and DMT tests were carried out in the specific test area (CPTu 3-0, 3-1 and 3-2 and DMT 1, 2 and 3 – Figure 1) to confirm the main soil characteristic recognized previously.

On the basis of the abovementioned investigations, a geotechnical model was determined and the layers with the higher probability of liquefaction were identified. In the following, the main units are listed with their Unified Soil Classification System (USCS) descriptors according to ASTM D2487-11 (2011):

- Topsoil from 0 to 1 m bgl (CH) – Layer 1;
- Silty clay from 1 to 4 m bgl (CH) – Layer 2;
- Clayey silt with sand from 4 to 6 m bgl (CL-CH) – Layer 3;
- Silty sand and sandy silt (fluvial Apenninic deposits) from 6 to 8 m bgl (ML-SM) – Layer 4;
- Silty sand (paleochannel of the Po River) from 8 to 17 m bgl (SM) – Layer 5;
- Silty sand (Syn-Glacial braided Po River deposits) from 17 to 20 m bgl (SM) – Layer 6.

2.2 *Pile test characteristics*

In the area four pile groups, with four piles each, and one with six piles were set up (Figure 1).

The piles were tapered precast piles 16.0 m long, with a tip/head diameter of 260/500 mm and a taper of 15 mm/m.

In pile groups N.2, N.3 and N.4 piles have the same center distance, but were driven with different energies, measured using a Pile Driving Analyser (PDA).

Chosen the installation energy of the piles ensuring the best performance as regards the improvement of the soil characteristics, two other pile groups were set up with a larger center distance than the previous ones. In Table 1 are shown the data regarding center distance and installation energy.

Table 1. Installation energy and center distance of piles in the pile groups.

	Energy (E)	Center distance (i)
Pile group	kNm	m
1	43.0	2.00
2	25.0	1.40
3	43.0	1.40
4	43.0	1.85
5	20.0	1.40

3 RESULTS

3.1 *Analysis of q_t data*

Figures 2, 3 and 4 show the profiles of the corrected cone tip resistance (q_t) acquired before and after the pile driving activity for the different pile groups. The CPTu tests after the pile driving were performed

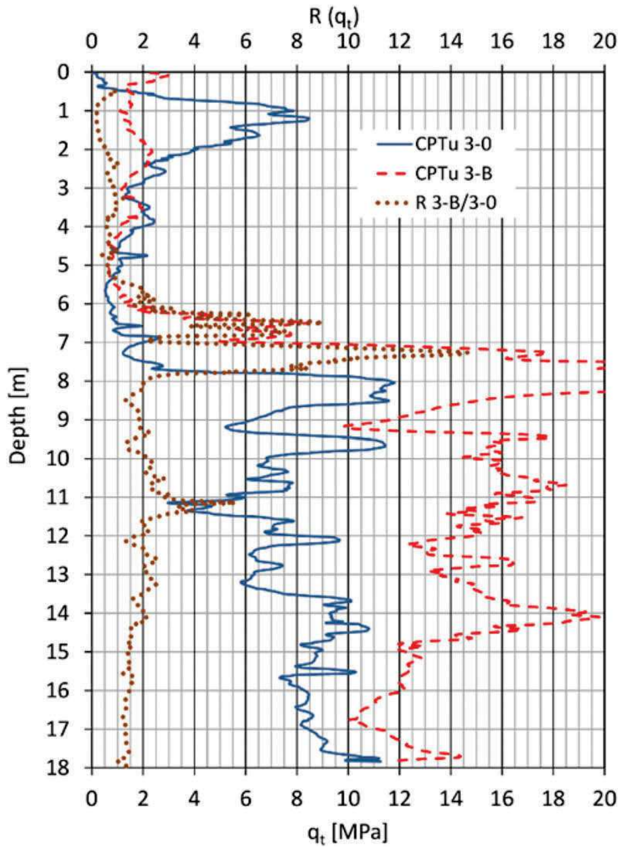

Figure 2. Corrected cone tip resistance q_t before and after pile driving and R(q_t) value - Pile group N.3 [E=43 kNm, i=1.4 m].

both in the center position of the group and along one or two sides of the group of piles.

The influence of tapered precast pile installation may be represented by the ratio post-installation q_t to the pre-installation q_t, the Improvement Ratio [R(q_t)], defined by the following expression:

$$R(q_t) = \frac{q_{t\ post\ installation}}{q_{t\ pre\ installation}} \quad (1)$$

It was computed at each measurement depth and shown in the same figures.

The $q_{tpost\ installation}$ data differ considerably depending on both the pile driving energy used and the distance between the piles.

Considering the group of piles N.3, there is a high increase in q_t along the entire profile of the pile, after the installation, in the layers 4 and 5 (Figure 2) with an average increase of about 2 in soils with initial q_t greater than 6 MPa and even greater with values lower than 6 MPa. In the latter case, some of the very high values could depend on the comparison of different type soils (clay in the pre-installation CPTu and sand in the post-installation CPTu).

In the case of pile group N.2, the piles have the same center distance as group N.3, but they have been driven with lower energy (Table 1). R(q_t) is

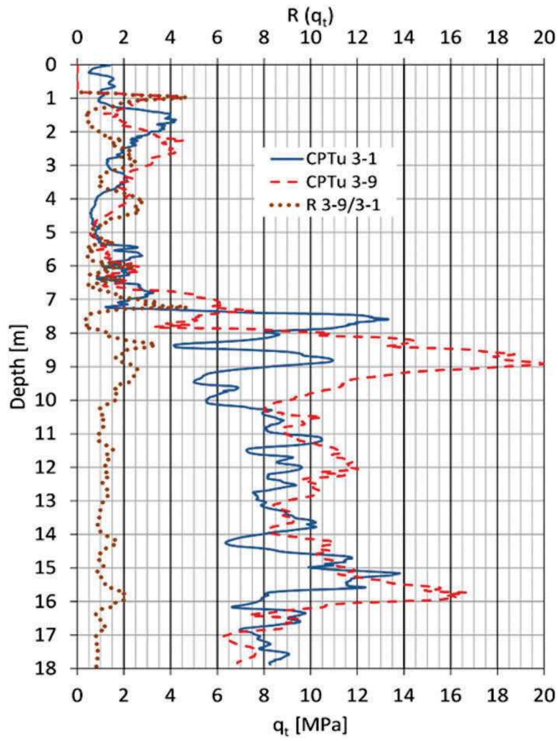

Figure 3. Corrected cone tip resistance q_t before and after pile driving and $R(q_t)$ value - Pile group N.2 [E=25 kNm, i=1.4 m].

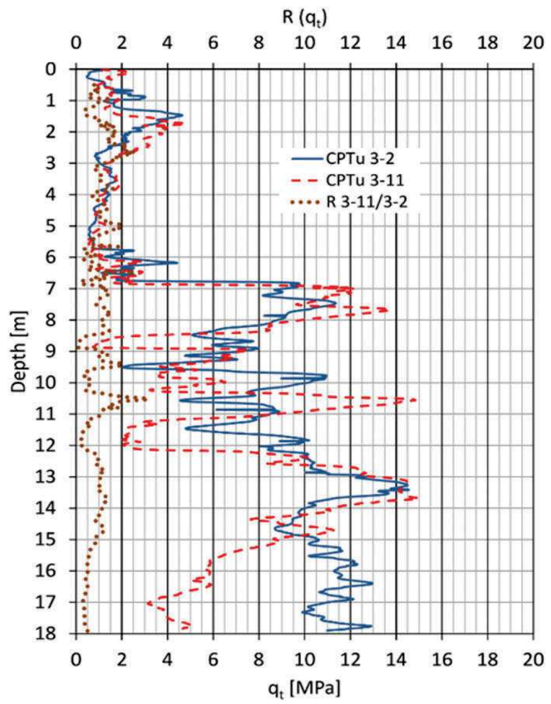

Figure 4. Corrected cone tip resistance q_t before and after pile driving and $R(q_t)$ value -Pile group N.4 [E=43 kNm, i=1.85 m].

significantly greater than 1 (Figure 3) only in the first meters of the silty sand layer 5, probably due to the effect of the lateral spreading of the soil, and is equal to 1 at deeper levels.

Finally, in the case of pile group N.4 (Figure 4), in which the piles have been driven with the same energy used for those of stand 3, but with a center distance of approximately 1.85 m, R has a value substantially equal to one, showing a lack of improvement, albeit a probable stratigraphic variability is present at various depths. A similar situation is also observed in the case of pile group N.1.

3.2 *Analysis of K_D and f_s data*

A similar comparison was carried out taking into account the horizontal stress index K_D of DMT tests carried out before and after the installation of the piles. The DMTs after installation were performed along one side of the pile groups.

K_D of the dilatometric test is strictly correlated with the horizontal earth stress coefficient and therefore with the horizontal stresses.

As in the previous cases, the influence of tapered precast pile installation may be represented by the ratio post-installation K_D to the pre-installation K_D, the Improvement Ratio [$R(K_D)$], defined by the following expression:

$$R(K_D) = \frac{K_{D\ post\ installation}}{K_{D\ pre\ installation}} \tag{2}$$

Also in this case for pile group N.3 (Figure 5) there is a marked increase in post-installation K_D with $R(K_D)$ values between 3 and 5, while for pile group N.4 the improvement is absent, except in the loose silty sand and sandy silt layer between 6.0 and 8.0 m, having a low cone resistance (Figure 6).

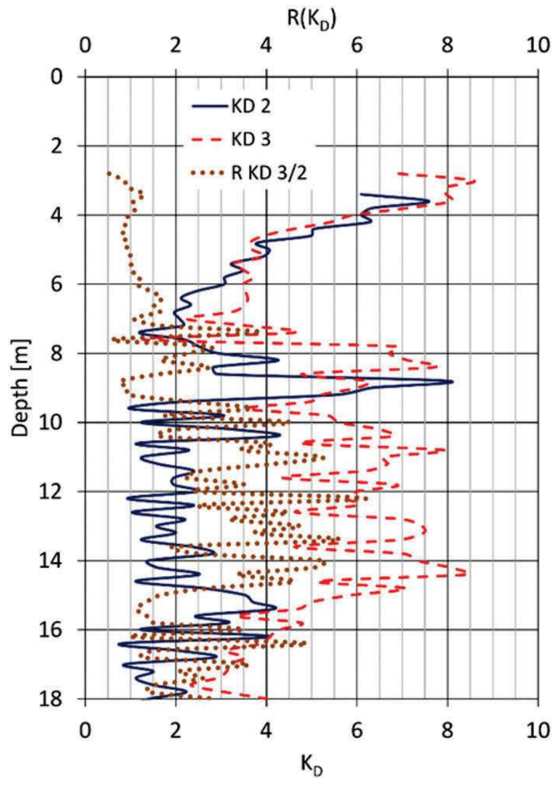

Figure 5. K_D before and after pile driving and $R(K_D)$ value - Pile group N.3 [E=43 kNm, i=1.4 m].

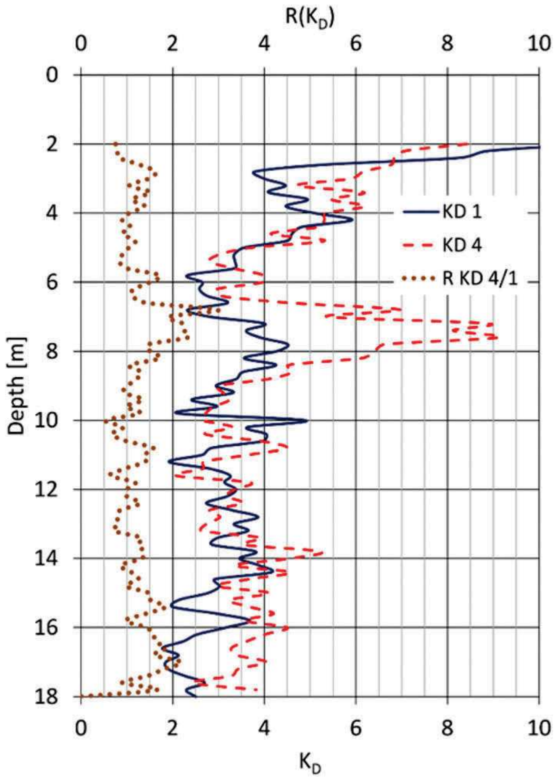

Figure 6. K_D before and after pile driving and $R(K_D)$ value - Pile group N.4 [E=43 kNm, i=1.85 m].

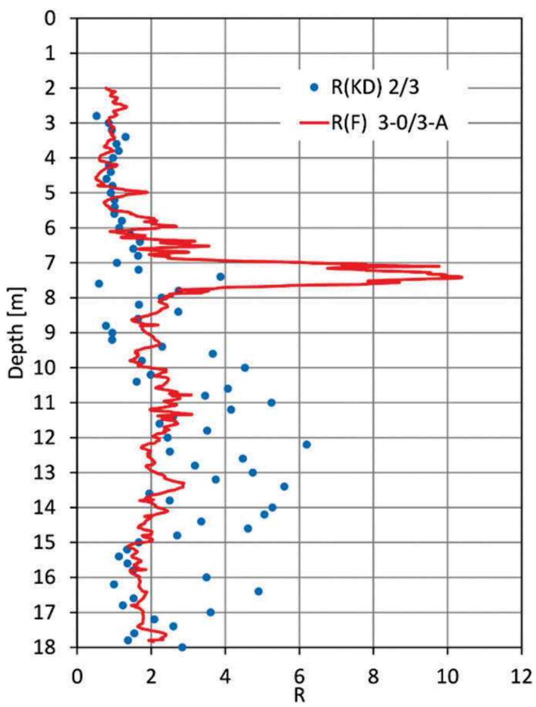

Figure 7. $R(K_D)$ and R(F) - Pile group N.3 [E=43 kNm, i=1.40 m].

Robertson (2016) highlighted that the normalized CPT sleeve resistance ($F = f_s/\sigma'_{vo}$) can be used to estimate the DMT K_D; therefore, the trend along the depth between R (K_D) and R (F) of various pile groups was compared, where R (F) is the following expression

$$R(F) = \frac{F_{post\ installation}}{F_{pre\ installation}} \tag{3}$$

Both ratios show the same phenomena, improvement of soil characteristics in correspondence of pile group N.3 (Figure 7) and modest if not zero improvement in pile group N.4 (Figure 8). Therefore, both, again, highlight the decrease of the improvement as the distance between the piles increases. The $R(K_D)$ ratio also is more sensitive than R(F) to the effects of the pile driving.

3.3 *Cumulative settlement prediction*

Adopting the Zhang et al. (2002) method to estimate liquefaction-induced ground settlements using CPT data, the effectiveness of the driving of piles on the decrease of the potential liquefaction effects was determined.

The Zhang approach combining the CPT estimate liquefaction resistance with laboratory test results on clean sand evaluates the liquefaction-induced volumetric strains for sandy and silty soils.

Zhang et al. (2002) assume that little or no lateral displacement occurs after the earthquake, such that the volumetric strain will be equal or close to the vertical strain. Integrating with depth the vertical strain in each soil layer, the potential liquefaction-induced ground settlement due to an earthquake is estimated by Equation 4,

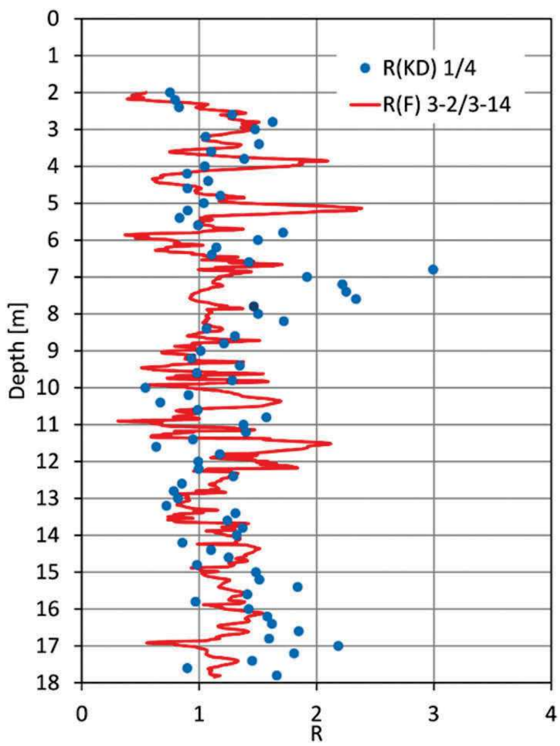

Figure 8. $R(K_D)$ and R(F) - Pile group N.4 [E=43 kNm, i=1.85 m].

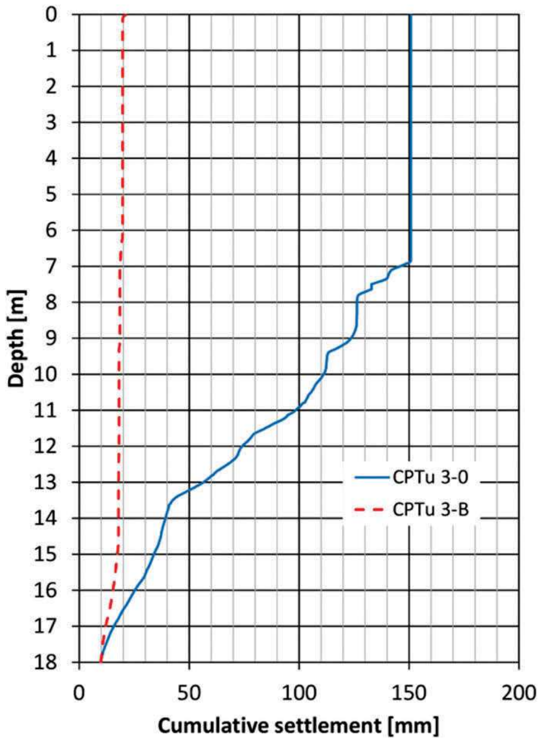

Figure 9. Post liquefaction settlement using Zhang et al. method (2002) - Pile group N.3 [E=43 kNm, i=1.4 m].

$$w = \sum_{i=1}^{j} \varepsilon_{vi}\ \Delta z_i \qquad (4)$$

where w is the calculated liquefaction-induced ground settlement at the CPT location; ε_{vi} is the postliquefaction volumetric strain for the soil sublayer i; Δz_i is the thickness of the sublayer i; and j is the number of soil sublayers.

The analysis of data related to the pile group N.3 (Figure 9) shows a great decrease of potential post liquefaction settlement and consequently that the improvement is large. Therefore, with groups of more piles than in this case, significant improvements can be achieved from the point of view of a potential soil liquefaction settlement lower than the virgin soil.

4 CONCLUSIONS

The CPTu and DMT in situ tests were used to evaluate the improvement of the soil characteristics due to the driving of precast tapered piles.

The data acquired using the CPT and DMT tests show that the installation of prefabricated tapered piles determines an increase of the soil resistance, pointed out by the increase in q_t, as well as a change in effective lateral stress highlighted by both the increase of F and K_D.

The phenomenon is clearly remarkable up to a distance of about 4-5 pile diameters; beyond this distance the phenomenon tends to run out.

In the present case, the driving energy played a non-negligible role in the amount of the improvement.

Referring to the potential post-liquefaction settlement due to an earthquake and determined with the Zhang et al. (2002) calculation method, the improvement of the soil behavior achievable through the installation of driving piles was clearly highlighted.

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