# INTERNATIONAL SOCIETY FOR SOIL MECHANICS AND GEOTECHNICAL ENGINEERING

XIII ICSMFE, 1994, New Delhi, India / XIII CIMSTF, 1994, New Delhi, Inde

# FIELD STUDIES OF AN INSTRUMENTED FULL-SCALE PILE DRIVEN IN CLAY

# ETUDES SUR PLACE D'UN PIEU INSTRUMENTE EN GRANDEUR NATURE BATTU EN ARGILE

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## SYNOPSIS

This paper presents the results of measurements of total stress against the shaft and base of a mandrel-driven cast-in-situ steel pile 57 m long, 508/457 mm in diameter, installed in NC clay. The pile was instrumented with 8 dilatometer cells (4 pairs at 6 m depth intervals) along the shaft and one dilatometer cell at the pile tip center. The cells readings (total stress) were taken during installation, subsequent reconsolidation and test loading to failure. During installation, driving was interrupted at various depths in order to take readings immediately after stopping driving and, at three different levels, after complete reconsolidation of the surrounding soil. The "initial" $\sigma_h$ readings and the subsequent decay curves have been compared to the $p_0$ readings and the subsequent decay curves against the side of a "standard" dilatometer blade, in order to explore the possibility of using DMT-$p_0$ to get indications of $\sigma_h$ against the pile at installation stage and subsequently. The total stress measured by the cell installed at pile tip has been compared to CPT-$q_c$. Two loading tests to failure were performed on the pile at two different levels. The first loading test was performed with the pile tip in clay at 50 m depth, with the mandrel inside the steel shell, without concrete. The second one was performed with the pile tip in sand at 57 m depth on the pile filled with concrete.

## INTRODUCTION

The case history presented herein is part of a major research effort coordinated and sponsored by ICELS PALI (Milan), starting in 1990, to investigate the performance of their mandrel-driven cast-in-situ "Multiton" piles. The main purpose of the field studies described in this paper is to investigate the possibility of predicting lateral stresses acting against pile shaft during installation ($\sigma_h$) and at the end of reconsolidation ($\sigma'_{hc}$) from the results of dilatometer test (DMT). The prediction of $\sigma_h$ during installation is useful to select an appropriate shell thickness in order to avoid collapse under soil pressure before pouring the concrete. Moreover $\sigma'_{hc}$ is of primary importance in determining shaft resistance of piles in clay (Azzouz et al. (1990)).

## TEST SITE DESCRIPTION

The test site is located at Mortaiolo, near Livorno (Italy). It was investigated by means of one borehole to 60 m depth, six dilatometer tests (DMT) and one cone penetration test (CPT) to 50÷60 m depth. DMTA dissipation tests were also performed at different levels, according to the procedure indicated by Marchetti et al. (1986). The typical soil profile (Fig.1) consists of:

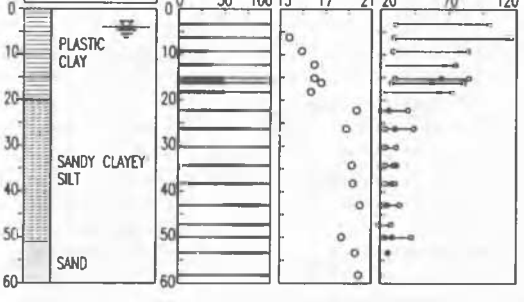

**Fig.1.** Soil profile and physical properties

•0÷20 m: very homogeneous, highly plastic, slightly sensitive soft clay; a thin overconsolidated desiccation crust is present in the upper 4 m;
•20÷51 m: medium plastic, slightly overconsolidated sandy clayey silt;
•51÷60 m: loose uniform silty sand.

The ground water table was about 4 m below ground level. Fig.2 shows the results of a typical DMT (see Fig.5b for CPT test result). Profiles of $c_u$ and $OCR$, evaluated from laboratory tests and from DMT, are shown in Fig.3. Laboratory tests results interpretation, including data from a nearby site (Rocchi et al., 1991), shows that the mechanical behaviour of the upper plastic soft clay layer is significantly influenced by the material "structure": once the microfabric is destroyed, beyond a critical value of applied stress or deformation, the material is in an underconsolidated state. For this material, DMT correlations (originally proposed for uncemented deposits) somewhat overestimate $c_u$ and $OCR$. In the "more normal" lower sandy clayey silt, $c_u$ and $OCR$ from laboratory tests and DMT are very similar.

The bottom silty sand formation is characterized by a relative density $D_r \leq 20\%$ and a peak angle of shear resistance $\varphi'_{peak} \approx \varphi'_{cv} = 32° \div 34°$. More details are illustrated by Rocchi et al. (1991).

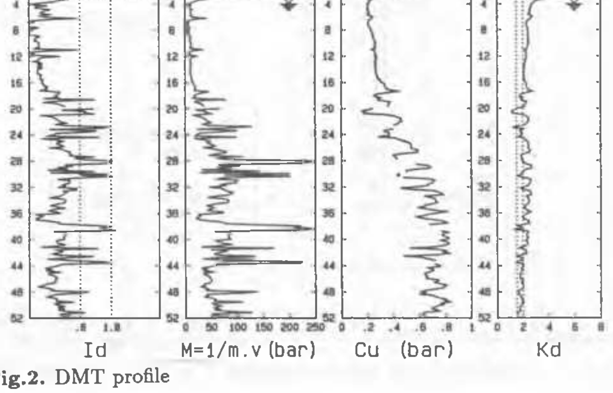

**Fig.2.** DMT profile

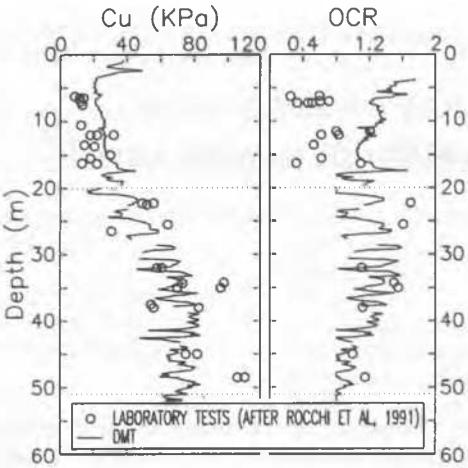

**Fig.3.** $C_u$ and $OCR$ of the clayey layers from DMT and laboratory tests (including data from a nearby site)

## TEST PILE: INSTRUMENTATION AND HISTORY

The test pile was a "Multiton" pile, 57 m long, consisting of two joined steel pipes, mandrel driven, then filled with concrete. The lower steel pipe was 457 mm in diameter, 8 mm thick, 30 m long and closed-ended with a flat base. The upper section was 508 mm in diameter, 6 mm thick, 27 m long. More details are given by Fioruzzi et al. (1991). The pile was instrumented along the lower 20 m of the shaft with 8 dilatometer cells, placed in pairs at 180° at four different levels, at 6 m intervals. The pile tip was instrumented with one dilatometer cell placed at the center. The location of the cells is shown in the detail of Fig.4. The dilatometer cells are total stress cells, working on the same principle as the DMT blade (Marchetti, 1980), except that only the first reading $p_0$ (lift-off) is taken, while the expansion (and $p_1$) is omitted. In practice these cells are the circular plates one would obtain by trimming the central part of a DMT blade. Before installing each cell, a vertical plane stripe (80×1000 mm) was first obtained in the shaft by filing, then a cylindrical recess was made in the wall. The cells were blocked in the recesses with the membrane flush with the surrounding plane.

The history of pile installation and testing (Fig.4) is described here below:

1. First the pile driving was interrupted every 3 m for a few minutes, in order to take readings at the lateral and base cells, until the pile tip reached 29 m of depth. At this level the pile driving was stopped for 5 days, in order to observe the decay of the total stresses against the pile shaft and base with time.
2. Subsequently the pile was driven from 29 m to 50 m with short intermediate stops every 3 m in order to take readings at all the cells. At this level the pile driving was stopped for 19 days. At the end of this interval, complete dissipation of excess pore pressure was achieved.
3. The first vertical loading test was performed with the pile tip in clay at 50 m depth.
4. Subsequently the pile was driven from 50 m to 57 m with intermediate interruptions every 0.20 m, in order to take readings at the tip cell with the same step used for CPT-$q_c$, while the readings at the lateral cells were taken every meter of depth. At the end of the pile driving, with the tip at 57 m, the pipe was filled with concrete.
5. After 28 days, the second vertical loading test was performed on the pile with the tip in sand at 57 m depth.
6. The latest readings at all the cells were taken after 245 days.

## STRESS MEASUREMENTS DURING INSTALLATION

The readings of total horizontal stress taken during pile installation at both lateral pressure cells located 2 m above pile tip (lowest cells) are plotted versus depth in Fig.5a, compared to the corresponding DMT-$p_0$ readings. The figure shows that the initial undrained $p_0$ readings taken at the lowest cells, which are practically unaffected by possible dissipation during installation, are nearly coincident with DMT-$p_0$ values. This was observed in both the cohesive layers forming the deposit, despite

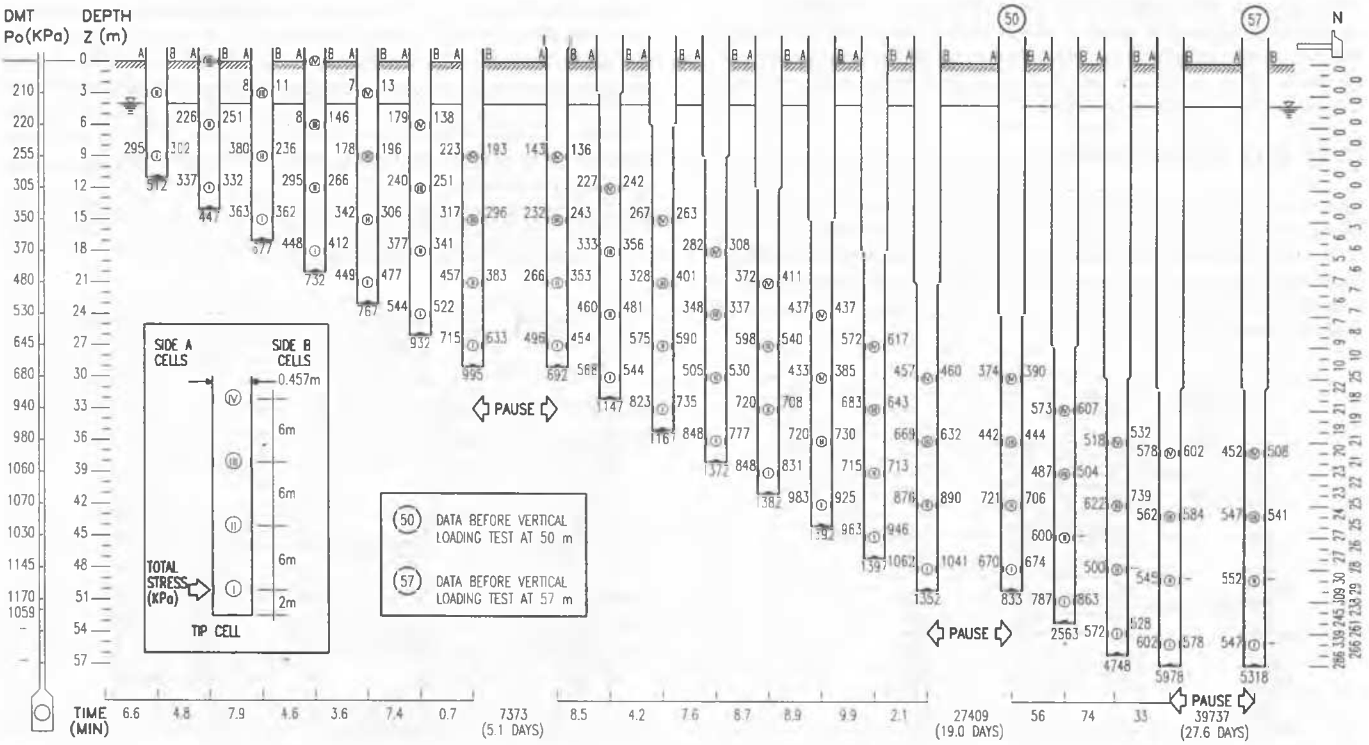

**Fig.4.** Pile history (total stresses taken in 1 ÷ 3 minutes after pile installation and, at 29, 50 and 57 m depth, also after pauses)

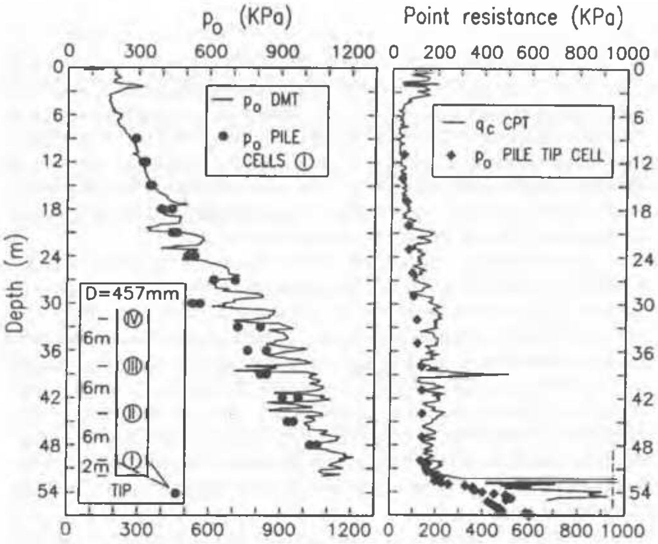

**Fig.5.** (a) $p_0$ at lowest pile cells during penetration compared to $p_0$ DMT. (b) $p_0$ at pile tip cell during penetration compared to $q_c$ CPT

considerable differences in plasticity and structure. Such coincidence also points towards insensitivity of the undrained $\sigma_h$ to shape and dimensions of the penetrating object. This result suggests that DMT-$p_0$ readings could be taken as $\sigma_h$ against the pile, useful to select shell thickness to prevent collapse. Fig.5a also shows the quasi-coincidence of $p_0$ measured by opposite (independent) cells, confirming the reliability and proper functioning of the cells; the close agreement can also be seen in Fig.4.

Fig.5b shows the profile of the readings taken at the base cell during pile driving versus depth, superimposed to CPT-$q_c$ profile. The figure shows that the pile tip cell provides a "penetrometric" profile very similar to the CPT profile in the cohesive layers (particularly in the upper homogeneous plastic clay layer), while in the bottom cohesionless layer the tip reaction is about 50% less than CPT-$q_c$. It should be noted, however, that it is unknown to what extent the tip cell pressure, located at pile tip center, represents the average pile base reaction.

## STRESS MEASUREMENTS AFTER RECONSOLIDATION

Horizontal effective stresses against pile shaft at the end of reconsolidation ($\sigma'_{hcp}$) have been evaluated from final (stabilized) total horizontal stresses measured by the lateral cells at different levels, by subtracting the corresponding equilibrium water pressure $u_0$. Total lateral stress versus time decay curves were observed at four different levels along pile shaft during driving interruptions, when the pile tip was at 29 m, 50 m and 57 m respectively, thus obtaining 12 $\sigma'_{hcp}$ data points. Two representative $\sigma_h$ versus *log t* decay curves, obtained from readings at the lowest lateral cells at 27 and 48 m depth, are shown in Figs.6b and 6e. For comparison, Figs.6a and 6d show the corresponding decay curves obtained by DMTA dissipation tests performed about at the same levels. Decay curves of total stress measured by the pile tip cell at 29 and 50 m depth are also shown in Figs.6c and 6f. It should be noted that the time required to reach full dissipation of excess pore pressure measured by the pile cells was approximately 10 days, i.e. about 10 times longer than dissipation time measured by the dilatometer blade. This indicates that reconsolidation around the bottom part of the pile occurred ≈ 10 times faster than inferred from DMTA tests scaled according to the squared ratio of the radii (expected time ratio ≈ 100), assuming for the DMT blade an equivalent radius of 20.57 mm (Robertson et al., 1988).

Effective horizontal stress profiles at the end of reconsolidation obtained by the cells located along pile shaft ($\sigma'_{hcp}$) and by DMTA dissipation tests ($\sigma'_{hcd}$) are shown in Fig.7a. Fig.7b shows the corresponding profiles of lateral earth pressure coefficients at the end of consolidation, $K_{cp} = \sigma'_{hcp}/\sigma'_{v0}$ (pile) and $K_{cd} = \sigma'_{hcd}/\sigma'_{v0}$ (DMT), superimposed to the profiles

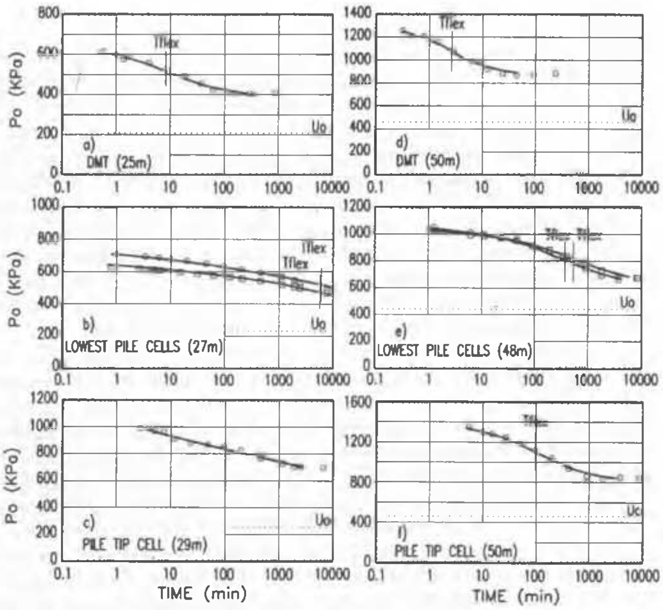

**Fig.6.** Sample decay curves

of initial lateral earth pressure coefficient $K_0$ estimated from laboratory tests (Rocchi et al., 1991) and from DMT. The most remarkable result shown in Fig.7a is that, while in the upper 25 m $\sigma'_{hcp}$ and $\sigma'_{hcd}$ values are nearly coincident, below this level $\sigma'_{hcp}$ is significantly lower (about 50%) than the corresponding $\sigma'_{hcd}$. This situation, relative to drained conditions, is quite different from the one observed during installation (undrained conditions), where $p_0$ values measured by pile cells and DMT blade were nearly equal for the whole depth. Some of the possible factors expected to cause such difference in behaviour are the following:

- geometry (shape and dimensions) of the penetrating object;
- length/depth effects in the pile (not affecting DMT response);
- soil arching (probably higher for the cylindrical than for the flat shape);
- soil properties (plasticity, sensitivity).

At the present state of the art, however, it is not easy to quantify the individual responsibility of the various factors. This leaves open the problem of predicting lateral effective stresses against pile shaft at the end of reconsolidation in a straightforward manner from DMTA dissipation tests.

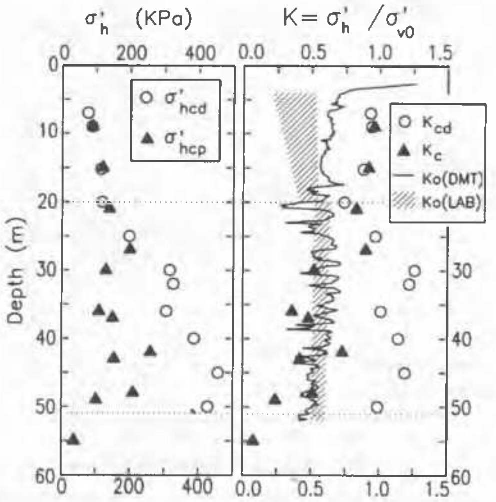

**Fig.7.** (a) End of reconsolidation effective horizontal stresses determined by DMT ($\sigma'_{hcd}$) and pile cells ($\sigma'_{hcp}$). (b) Horizontal pressure coefficents ($K_{cd} = \sigma'_{hcd}/\sigma'_{v0}$, $K_{cp} = \sigma'_{hcp}/\sigma'_{v0}$)

## LOADING TESTS

Two vertical loading tests to failure were performed on the same pile at different depths. The first one was a "rapid" constant rate of penetration test (0.25-0.50 mm/min) and was carried out with the mandrel inside the steel cell, without concrete, when the pile tip was at 50 m depth (clay). Failure occurred under an axial load of about 2.4 MN. The second test was performed by incremental loading, with increments of 0.3 MN up to a failure load of 4.2 MN, on the pile filled with concrete, when the pile tip was at 57 m depth (sand). Both loading tests were performed about 3÷4 weeks after stopping driving, when the dilatometer cells measurements indicated the full dissipation of excess pore pressure. More details about testing procedures, instrumentation, results and interpretation may be found in the papers by Rocchi et al. (1991) and Totani (1991).

In Fig.8 the axial load at pile head, measured by a loading cell, and the total pressure measured by the pile tip cell are plotted versus pile head settlement, measured by three displacement transducers. A very unexpected pile tip behaviour was observed during the second loading test (pile tip in sand), as shown in Fig.8b: in the early stages of the test, up to about 3.6 MN, the cell did not nearly feel the load, while a sharp drop in base reaction (some 40%) was observed approaching the failure load. This behaviour is in contrast to the "common sense" notion that tip reaction should increase with load (as observed in the first loading test with pile tip in clay, Fig.8a). The writers consider highly unlikely that this behaviour is due to cell malfunctioning, because the cell, of mechanical balance-of-zero type, provided perfectly consistent results throughout the 9 months experimentation, including the heavy hammering at installation stage. Possible reasons for this behaviour are probably related to the sudden release of residual driving stresses and/or getting over peak strength. Furthermore it is unknown to what extent the pressure measured by the cell at pile tip center represents the average pile base reaction.

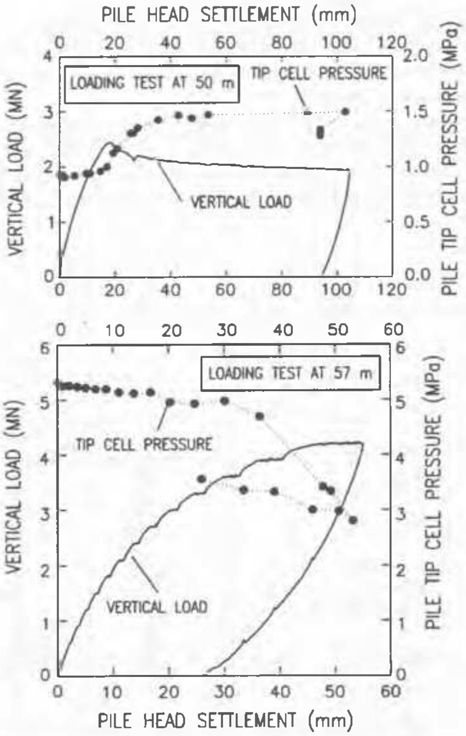

**Fig.8.** Vertical loading tests results. (a) Test at 50 m (constant settlement rate: 0.5 mm/m). (b) Test at 57 m (incremental loading: 300 KN steps)

## CONCLUSIONS

1. In the cohesive deposit studied the DMT-$p_0$ measurement is nearly coincident with $\sigma_h$ (total, undrained) against the pile during penetration. This was observed in both the cohesive layers forming the deposit, despite considerable differences in plasticity and structure. This suggests that DMT-$p_0$ could be taken (once confirmed by additional comparisons) as $\sigma_h$ against the pile, useful to select shell thickness to prevent collapse. Such coincidence also points towards insensitivity of the undrained $\sigma_h$ to shape and dimensions of the penetrating object.

2. A quite different situation is found for stresses at the end of the reconsolidation (drained conditions). The final horizontal effective stresses for the pile ($\sigma'_{hcp}$) and for the blade ($\sigma'_{hcd}$) are nearly equal in the upper layer, but significantly different ($\sigma'_{hcd} \approx 2\sigma'_{hcp}$) in the lower layer. This leaves open the problem if and how $\sigma'_{hcd}$ can be used to predict $\sigma'_{hcp}$. Solving such problem entails the ability to quantify the individual responsibility of various factors causing such difference. Among the expected ones: shape and dimensions of the penetrating object, length/depth effects in the pile, arching (probably higher for the cylindrical shape than for the flat shape), soil sensitivity. Such ability is probably beyond the present state of the art.

3. Dissipation around the bottom part of the pile occurs ≈ 10 times faster than dissipation inferred from DMTA tests scaled according to the squared ratio of the radii.

4. A very unexpected pile tip behaviour was observed during the second load test (pile tip in sand). In the early stages of the loading the pile tip cell did not nearly feel the load. Approaching the failure load, the pile tip cell indicated a marked drop (some 40%) in base reaction. This drop (perhaps related to the sudden release of residual driving stresses and/or getting over peak strength) is in sharp contrast to the common sense notion that tip reaction should increase with load.

5. The pile tip cell provided a "penetrometric" profile very similar to the CPT profile, except in the final cohesionless layer, where tip reaction was about 50% less than CPT tip reaction.

## ACKNOWLEDGEMENTS

ICELS PALI active collaboration and support are gratefully acknowledged. The writers also thank L. Fatigati for his technical assistance during tests execution.

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