A case study of non-displacement piles in Pliocene sands
Une étude de cas sur des pieux sans déplacement dans sables du Pliocène

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ABSTRACT
The paper focuses on the foundation piles of a very large industrial plant located in the peninsula of Setúbal, Portugal. The ground is composed by a thick deposit of Pliocene sands and silty sands, with the water table close to the surface. Bored piles drilled under bentonite slurry of diameters from 0.5 m to 1.0 m were adopted with lengths up to 22 m. The site was carefully characterized. A number of static and dynamic axial load tests on prototype piles were carried out, providing the evaluation of the base and the shaft resistance. The results of these tests are discussed and compared with the ones obtained from a semi-empirical method based on in situ ground tests.

RÉSUMÉ
Le papier focalise les pieux de fondation d'une usine industrielle de grande dimension située dans la péninsule de Setúbal, Portugal. Le terrain se compose par un dépôt épais de sables limoneux du Pliocène, avec la nappe phréatique près de la surface. Des pieux forés à la boue de diamètres de 0.5 m à 1.0 m ont été adoptés avec des longueurs jusqu'à 22 m. L'emplacement a été soigneusement caractérisé. Un certain nombre d'essais axiaux statiques et dynamiques de chargement sur des pieux de prototype ont été effectués, fournissant l'évaluation de la résistance de pointe et de frottement latéral. Les résultats de ces essais sont discutés et comparés à ceux obtenus à partir d'une méthode semi-empirique basée sur les essais au sol in situ.

Keywords: Pile, static load test, dynamic load test, installation effects

1 INTRODUCTION
Design methods, observation of pile performance during installation and loading, and soil-pile interaction analysis are key aspects for a complete understanding of pile behaviour. Despite recent advances on the comprehension of the changes that occur in the soil around axially loaded piles, common design calculations heavily rely on empirical correlations (D’Aguiar, 2008). The use of the CPT cone resistance provides a good basis for these calculations but care must be taken in data extrapolation from empirical correlations or even from pile load tests (static or dynamic) to pile geometries and soil conditions outside the current database, in order to ensure the correct consistency with the actual conditions.

The weak reliability of the prediction methods (Viana da Fonseca and Santos, 2008) and the clear influence of installation techniques on the performance are closely related. In fact, the limitations in characterizing the installation method prevent the improvement of prediction methods and, in addition, disable the optimization of some of the installation techniques. Therefore load tests on instrumented piles can play a major role to check bearing capacity and settlement calculations.

2 SITE DESCRIPTION
The paper describes the design and the static and dynamic tests of bored piles used for founding large industrial buildings. The site is located in Setúbal, 50 km to the south of Lisbon, between the valleys of the Tagus and Sado rivers, not far from the Atlantic coast. In the geological chart of Portugal the site corresponds to Pliocene formations, consisting of sands, silty sands, clayey sands, with thin lenses of sandy and silty clays.

The field geotechnical characterization consisted of: i) 91 conventional boreholes with SPTs and (disturbed) sampling; ii) 4 groups of cross-hole seismic tests, close to 4 boreholes; iii) 12 CPTs. In general, the depth reached by the CPT was small, ranging from 1.6 m to 9.4 m (average 5 m). Grain size distribution and Atterberg limits were evaluated in the lab together with some triaxial compression tests on reconstituted samples.

The interpretation of the results provided by that characterization permitted the division of the ground in four Geotechnical Horizons, from GH4 (the shallowest) to GH1 (the deepest), as shown in Table 1.

<table>
<thead>
<tr>
<th>Horizon</th>
<th>Description</th>
<th>N&lt;sub&gt;SPT&lt;/sub&gt; = N&lt;sub&gt;10&lt;/sub&gt;</th>
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<tbody>
<tr>
<td>GH4</td>
<td>Organic soils, man-made fill</td>
<td>≤ 10</td>
</tr>
<tr>
<td>GH3</td>
<td>Sands, silty and clayey sands, interspersed with thin layers and lenses of sandy and silty clays (Pliocene)</td>
<td>11 - 29</td>
</tr>
<tr>
<td>GH2</td>
<td>Sands, silty and clayey sands and, sparsely, sandy or silty clays (Pliocene)</td>
<td>30 - 60</td>
</tr>
<tr>
<td>GH1</td>
<td>Sands, silty and clayey sands and, sparsely, sandy clays (Pliocene)</td>
<td>&gt; 60</td>
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The thickness of GH4 is generally small. GH1 forms the substratum and is normally found at 12-15 m depth. Between these two extreme horizons, the disposition of the other is rather complex: GH2 may occur on GH3 and, even, GH1 (then called GH1A) may be found on or in between GH2 and GH3.

Figure 1 presents results of field tests at one of the locations where pile load tests have been carried out.
3 COMPRESSIVE RESISTANCE FROM GROUND TEST RESULTS

In view of the variability of the geotechnical characteristics, both in depth and in plan, the site was divided in distinct Design Zones, corresponding to boreholes whose results could be considered reasonably alike. The intention was to achieve a common solution for each zone concerning the pile foundations.

The evaluation of the bearing capacity of the piles was done on the basis of the method developed by Bustamante and Gianeselli (1983). Since the CPT did not reach the relevant horizons controlling the pile behaviour, for applying the method mentioned above the values of $q_{cm}$ have been evaluated through $N_{SPT}$, applying the correlation proposed by Robertson and Campanella (1983).

The adopted procedure involved the following steps:

a) Definition of the Design Zones taking into account, particularly, the depth of the substratum, the distribution of the Geotechnical Horizons defined in Table 1 and the proximity of the pile foundations.

b) For the boreholes of each Design Zone and for each Geotechnical Horizon, calculation of the mean value of the ratio $D/N$.

c) For the boreholes of each Design Zone and for each Geotechnical Horizon, calculation of the mean value of the ratio $q/N_{SPT}$.

d) For each borehole, calculation of $q_{cm}$, on the basis of $q_{SPT}$ and taking the coefficient $k_c$ (Bustamante and Gianeselli, 1983), equal to 0.3.

e) For each borehole, calculation of the characteristic values $q_{cm}$, on the basis of $q_{SPT}$ and $N_{SPT}$.

The application of the design procedure described above to the borehole close to the pile load tests ES1 and EI1 (borehole BH24, Figure 1) and the pile load test EI2 (boreholes BH57 and BH58), led to the results summarized in Table 3. The actual pile lengths (about 16 m) and pile diameter (800 mm) were considered for a proper comparison. As indicated in the tables, borehole BH24 was included in the Design Zone 1, borehole BH57 was included in the Design Zone 4 and borehole BH58 belongs to Design Zone 2.

<table>
<thead>
<tr>
<th>Geotech. Hor.</th>
<th>$L_1$ [m]</th>
<th>$q_{cm}$ [kPa]</th>
<th>$q_{cm}$ [MPa]</th>
<th>$q_{cm}$ [kPa]</th>
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<tr>
<td>1</td>
<td>1.16</td>
<td>46</td>
<td>71</td>
<td>41</td>
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<td>46</td>
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<td>41</td>
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<tr>
<td>1A</td>
<td>1.50</td>
<td>120</td>
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<td>3</td>
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Table 3 – Summary of the computation of the compressive resistance of piles ES1, EI1 and EI2 on the basis of ground test results.
A load test program was prepared with the aim of determining the response of representative piles both in terms of settlement and limit load. The program consisted of 5 static load tests and 3 dynamic impact load tests. In the static load tests 3 of them had only measurements of the load and the settlement at the pile head (bored piles ES4, ES5 and ES2) while in the other 2 tests the piles were fully instrumented with strain gauges in such a manner that the base and shaft resistance can be derived separately from the measurements (bored piles EI1 and EI2).

Figures 2 and 3 refer to piles EI1 and EI2, respectively, with the soil profile and the load distribution corresponding to a load stage close to the expected SLS load and to the maximum test load. Some gauge levels were eliminated from the analysis due to measurement imprecision (error).

The parabolic shape of the load distribution curve (Figures 2 and 3) and the almost linear distribution of the unit shaft resistance (Figure 4) are in agreement with the soil conditions and with the increase of effective stresses with depth.

5 DISCUSSION OF RESULTS

The following figures show the load-settlement curves of the piles allowing a direct comparison of the results. Figure 5 refers only to the static tests, while Figure 6 provides a comparison of load-settlement curves of static and dynamic load tests.

From the analysis of Figures 5 and 6 one can observe quite a distinct response in the various piles. The piles EI1, EI2 and ES4 (the latter has a diameter of 600 mm) had better performance highlighting the progressive mobilization of the shaft resistance and the base resistance. On the contrary, piles ES2 and ES5 exhibited quite high settlements, mainly for load stages beyond 3000 kN.

Although some differences in soil conditions occur among the locations of the distinct load tests, the discrepancies in the pile load test results are probably due to installation effects, which have a strong influence on the mobilization of the shaft and mostly of the base resistance of non-displacement piles.

The comparison of load-settlement curves shown in Figure 6 reveals a satisfactory agreement between the dynamic and the static load tests for loads up to 3000 kN. For higher loads, the accordance is only good between piles EI1 and EI2 (static load tests) and ES1, RS1 and RS2 (dynamic load tests).

Figure 7 shows the distribution of the unit shaft resistance \( q_s \) determined from the dynamic load tests. For the static load tests the unit shaft resistance can be calculated from the change of load between the strain-gauge levels divided by their distance...
and pile perimeter. However, such differentiation can originate erratic distributions of $q_s$ due to imprecision of the strain measurements and their conversion to load in the piles. Therefore, only average values of the unit shaft resistance were calculated for the upper horizons (GH2 and GH3) and for the lower horizon (GH1) (see Table 4).

One can see from Figure 6 and Table 4 that the ultimate resistance lies in the range between 6100 kN and 8700 kN, which is much lower than the mean values indicated in Table 3 (11761 kN to 14422 kN), mainly due to overestimation of the base resistance by the semi-empirical method.

Regarding the unit shaft resistance, the values obtained from the static load tests are in good agreement with the average values derived from the dynamic load tests. The values vary between 100 kPa and 120 kPa for the upper horizons (GH2 and GH3) and between 150 kPa and 180 kPa for the lower horizon (GH1). Therefore the shaft resistance is underestimated by the semi-empirical method (see values of $q_{sh}$ in Table 2).

6 CONCLUSIONS

The paper presents a case on non-displacement piles in Pliocene sands, with the compressive axial resistance of the piles being evaluated by a semi-empirical method using in situ soil tests and by static and dynamic load tests. The overall analysis of the results allowed the following conclusions to be drawn.

The combined execution of static and dynamic load tests permitted the assessment of the compressive resistance, with comparable results and similar load-settlement curves.

For the expected service load, the settlements were generally lower than 8 mm with the predominant contribution of the shaft resistance. For the maximum load, the settlements were, in some cases, quite high and there was an important mobilization of the base resistance.

The mean values of the compressive pile resistance estimated by the semi-empirical method correspond to an overestimation of the resistance found in the load tests. The main reason may be attributed to the high values of $q_s$ obtained from correlation with $N_{SPT}$ in refusal conditions ($N_{SPT} > 60$).

In two test piles the response was clearly different from the other ones and the reason cannot be simply assigned to soil variability; this is a clear indication of the installation effects, which have a strong influence on the mobilization of the shaft and, mostly, of the base resistance of non-displacement piles.

The application of Eurocode 7 to the results of the pile load tests would provide values of the design compressive resistance for ULS in reasonable agreement with those presented in Table 3, calculated through the semi-empirical method. Then, the safety conditions would be satisfied by both methodologies.

The resistances found in the pile load tests are satisfactory for the pile design loads, both for SLS and ULS conditions. Moreover, the working piles had a minimum length of 18 m, about 2 m more than the test piles, which means that the design was sufficiently conservative.

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REFERENCES


