Pile Setup over a Period of Seven Years Based on Dynamic Load Tests in Overconsolidated Clay

L.B. Passini, L.B. Benetti, A.C.M. Kormann

Abstract. This paper presents the results of a field investigation into pile setup in overconsolidated clay soil that was conducted during a period of almost seven years, at Guabirotuba Geological Formation, south of Brazil, where the experimentation site of the Federal University of Paraná is located. One driven precast prestressed concrete pile was subjected to dynamic load tests at four different events: at the end of driving (EOD) and at three re-strikes: after 113.5 h (4.7 days), 288 h (12 days) and 2342 days (6.4 years) of pile installation. Re-strike measurements confirm that pile setup occurred and the shaft resistance component, not the end-bearing, contributes predominantly to the increase in capacity along time.

Keywords: driven piles, dynamic load test, pile setup, overconsolidated clay soil.

1. Introduction

Driven piles are displacement piles where no (or minimal) soil emerges at the surface as a result of their installation (Salgado, 2008). Prestressed concrete piles are displacement piles that offer several benefits compared to other driven pile systems (e.g. Gerwick, 1968; Hussein et al., 1993a; Tomlinson, 1994; Fleming et al., 2009). Tensile stresses, which can arise in a pile during driving, can be better resisted due to prestressing forces and the pile is less likely to be damaged during handling. Bending stresses during driving are also less likely to produce cracking than in conventional precast concrete piles.

As is well known in foundation engineering, piles undergo a process of setup after installation (e.g. Hussein et al., 1993b; York et al., 1994; Chow et al., 1998; Axellsson, 1998; Axellson & Westin, 2000; Tan et al., 2004; Fellenius, 2008; Yan & Yuen, 2010; Lee et al., 2010; Steward & Wang, 2011; Lim & Lehane, 2014; Basu et al., 2014; Afshin & Rayhani, 2015), leading to an increase in capacity along time. It is suggested that this happens both because of the dissipation of positive pore pressure excess and of soil aging (particles rearrangement around the pile shaft) along time after installation. Steward & Wang (2011) define aging as the increase of the soil friction angle at a constant effective stress over time, similar to the one of the secondary compression after primary consolidation is finished. However, it is also known that the two phenomena happen almost at the same time.

Increase in pile capacity after initial driving was well observed in clays and sands over decades. This phenomenon is definitely favorable to engineering designs such as in many important practical implications, regarding testing methods, during the programming of foundation construction and for the reassessment of existing driven pile capacities. Studies related to pile setup have been developed in the field (e.g. Fellenius et al., 1989; Axellsson, 1998; Bullock et al., 2005a; Lee et al., 2010; Ng et al., 2013a; Attar & Fakharian, 2013) and in scale models from laboratory tests (e.g. Lim & Lehane, 2014; Rimoy et al., 2015; Afshin & Rayhani, 2015). Although literature provides knowledge regarding the subject of the pile setup phenomenon, the complete contributing mechanisms to the setup are not well understood.

Nevertheless, it is known that the setup phenomenon is related to the disturbance caused by pile installations such as buried, monotonically jacked and driven piles, where displacement piles had larger shaft capacity gains along time (e.g. Afshin & Rayhani, 2015). In addition, higher stress level (σ′, - confining vertical effective stress) appears as an important factor in the occurrence of resistance gains along time (e.g. Lim & Lehane, 2014). The most part of the engineers do not attempt to assess setup during construction (Bullock, 2008).

The evaluation of pile resistance over time can be achieved by restricting the pile at different times using the tool of dynamic load tests used for foundation control (e.g. York et al., 1994; Hussein & Likins, 1995; Axellsson, 1998; Chow et al., 1998; Axellson & Westin, 2000; Tan et al., 2004; Yan & Yuen, 2010; Lee et al., 2010; Ng et al., 2013a; Attar & Fakharian, 2013; Afshin & Rayhani, 2015). The dynamic load test is a high strain dynamic test used to assess the bearing capacity of a pile (shaft and tip) by applying a dynamic load at its top (a falling mass) while recording acceleration and strain near its head. Additionally, this
test provides the assessment of time dependent soil strength changes, determining dynamic pile stresses under hammer impacts, pile structural integrity and investigating hammer and driving system performance (Hussein et al., 1993b).

In order to contribute to a better understanding of high strain dynamic testing of driven pile shafts, this paper describes the results of the research about the behaviour of precast prestressed concrete pile regarding the increase of its capacity over time (setup effects), at Guabirotuba Geological Formation in the State of Paraná, south of Brazil, which is in progress at the experimentation site of the Federal University of Paraná - UFPR.

2. Materials and Methods

2.1. Geotechnical experimental field

The Geotechnical Experimentation Site of the UFPR is located at the Polytechnic Center Campus in Curitiba, Brazil. One of the reasons for choosing this particular site was because its stratigraphy and soil properties have been extensively studied and are very well documented (e.g. Salamuni, 1998; Chamecki et al., 1998; Kormann, 1999; Negro et al., 2012). The natural soil belongs to the tertiary Guabirotuba Geological Formation. Overconsolidated silty clays and clayey silts with high plasticity are soils commonly present in this sedimentary formation. Polished and shiny surfaces are commonly seen in the clayey soil mass. These slickensides follow a pattern of difficult identification. Well defined tectonic structures are also present. Lenses of sands, rich in feldspar, frequently occur inside the clayey mass. Some conglomerate and carbonate deposits may occur at specific sites.

Two Standard Penetration Tests (SPT) and two Cone Penetration Tests (CPTU) were performed in the area very close to this pile location (Kormann, 2002). The clay soil, typical of the Guabirotuba Geological Formation, is present until approximately 5.0 m of depth in red or variegated shades, which are usually associated with weathering. These layers have a soft to stiff consistency. Below them and until the end of the SPT boring, there are gray clay materials of hard and stiff consistency. Quartz and feldspar grains are disseminated in the silty-clay matrix. The occasional presence of sand is a characteristic of the area of study. Water table is located at about 2.0 m of depth. Between 5.0 and 7.0 m of depth, a tougher layer is evidenced by the cone tip resistance that exceeds 10 MPa. SPT also accuses a greater number of blows in this region. In general, remarkably high pore pressures were generated during cone penetration. Figures 1 and 2 show the geotechnical profiles by SPT and CPTU, respectively.

2.2. Precast prestressed concrete pile

The pile installed and tested is a precast prestressed concrete pile, with a square and solid section of 0.26 x 0.26 m, having no seams. Total pile length is 10 m and length inside ground is equal to 9 m. The driven system used had a free-fall hammer with a drop height of 0.60 m and weight of 29.4 kN. Figure 3 displays the driven pile record, where the number of blows indicated is the mean for every 0.50 m of pile penetration.

An analysis of Fig. 3 shows that, between 5.0 m and 7.0 m of penetration, the number of blows increased. This behaviour is compatible with the geotechnical investigation data (in particular SPT-7 and CPTU-8), which accused greater resistance in this region.

After penetration reached 6.0 m, the installation of the instrumentation was executed. It consisted of fixing a specific pair of strain transducers and a pair of accelerometers, positioned at 0.60 m from the top of the pile. Sensor pairs were installed diametrically opposite one another, aiming to compensate the bending effect on the pile, which tends to occur when hammer blows are applied. For every blow, sensors data were processed in the field by the Case Method (e.g. Goble & Hussein, 1994), providing signals representing the change in intensity force obtained from the measured strain and velocity, integrated from the acceleration data at pile length along time. These signals were monitored and stored using a Pile Driven Analyzer® (PDA), a data logger equipped with a memory card. The Case Method is based on simplified pile and soil behaviour assumptions (free end and plastic soil), resulting in a closed form solution related to the impact and its reflection from the tip, by using the wave propagation theory (Paikowski et al., 2004).

During penetration between 6.0 m and 9.0 m, the maximum average axial compression at the instrumentation level (CSX) ranged from 8.0 to 10.0 MPa. Maximum tensile stress below the sensors (TSX) reached 2.6 MPa. These stress levels are acceptable, since the pile structural resistance is greater than that.

2.3. Dynamic load tests

Dynamic load tests were performed at four distinct events. The first immediately after installation, the second after 113.5 h (4.7 days), the third after 228 h (12 days) and the fourth and last one at 2342 days (6.4 years) after installation. The procedure of increasing dropping hammer heights was applied (Aoki, 1989). For the three first tests a hammer with weight of 29.4 kN was used. For the last dynamic test a hammer with weight of 22.6 kN was available. In spite of the fact a distinct hammer was used in the fourth test, the data assessment provided in the following paragraphs will present evidences that full soil resistances were mobilized in all restrikes. In such case, the use of a lighter hammer in the fourth test does not affect the interpretation.

During the first test, the pile received 7 blows, at heights of 0.2, 0.4, 0.6, 0.8, 1.0, 1.2 and 1.4 m. At the second test, the pile received 6 blows, at heights between 0.2 and 1.2 m. In the third test, the pile received 11 blows, at heights between 0.2, 0.4, 0.6, 0.8, 1.0 (twice), 1.2, 1.4, 1.6,
1.8 and 2.0 m. At the last test, the pile received 5 blows, at heights between 0.2 and 1.0 m.

The set (permanent vertical displacement or penetration of the pile and plastic deformation of the soil) and the rebound (elastic compression of the driving head, pile and soil) resulting from the hammer impact at the pile top were recorded for all blows (as executed during pile installation) and signals from sensors were monitored and stored using the Pile Driven Analyzer® (PDA) for all tests.

Figure 4 illustrates the signals monitored in dynamic load tests. The solid line represents force and the dashed line corresponds to the velocity multiplied by impedance along time. This representative figure refers to the second dynamic load test after 113.5 h (4.7 days) of pile installation (end of driving - EOD), blow number four and drop height equal to 0.80 m.

For all tests, pile length under sensors ($L$) is equal to 9.40 m and the time required for the wave of the hammer impact to spread until the tip of the pile and return to the top ($2 \frac{L}{c}$, where $c$ is the wave travel velocity) is equal to 5.7 ms.

For concrete piles PDI (2003) states that the wave propagation velocity ($WS$) must be determined for each pile. It can be determined during driving, if wave up indicates some tension reflection (local “valley” in wave up at $2 \frac{L}{c}$). With this type of $WS$ determination, the variability in pile properties and the degradation of pile material during repeated hammer blows are considered.

ASTM D4945 (2012) recommends that the wave velocity for concrete piles would preferably be determined from an early impact event if a tensile reflection from the pile toe is clearly identified.

Therefore, for all tests, the wave propagation velocity ($WS$) equal to 3300 m/s (e.g. Hussein et al., 1993a; Kormann, 2002; Robinson & Iskander, 2008) was measured on the basis of the ascending wave (wave-up), looking to identify tension reflections during the time corresponding to the pile tip response.

Figure 1 - Geotechnical profiles by SPT.
Wave propagation velocity is used to calculate the dynamic elastic modulus (EM), which according to the one-dimensional wave propagation theory is given by concrete pile density multiplied by the squared wave propagation velocity \( (\rho \times WS^2) \). Considering concrete pile density as equal to 24.5 kN/m\(^3\) (e.g., Kormann, 2002; PDI, 2003; Robinson & Iskander, 2008; Cintra et al., 2014), the EM obtained was approximately equal to 27 GPa.

During dynamic load tests, compressive and tensile stresses were controlled, in order to prevent damage to the pile. The maximum values for compressive stress (CSX) were obtained at the third test, being equal to 18.5 MPa and for traction stress (TSX) the value was equal to 2.6 MPa, at the fourth test.

The ratio between nominal energy and measured energy, which quantifies efficiency of the hammer, ranged from 11.9% to 35.7%, increasing as the drop height increases. Minimum and maximum values were obtained at the third test.

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Table 1 displays the number of blows, drop heights, EMX (measured energy), RMX (total capacity calculated by Case Method), set and rebound for each dynamic load test.

Figure 5 shows the RMX vs. EMX curve of each test. The shape of the curves indicates that the full capacity of the pile was mobilized since after the RMX value reached a peak there wasn’t further growth of the mobilized load capacity as a function of energy increase. From Table 1, it can be seen that after the peak value, the pile starts to penetrate in the soil with high sets and almost constant rebounds, indicating that soil-pile failure was achieved.

The increase of the maximum value of RMX along the tests clearly indicates that the pile load capacity experienced an increase along time.

3. Results and Discussions

3.1. Predicted results

The predicted results of pile load capacity were calculated by selecting some semi empirical methods, routinely applied, which are based on field investigation tests (SPT and CPTU), such as Aoki & Velloso (1975, 1985, 1996), Philipponnat (1979), Bustamante & Gianselli (1982),
Décourt & Quaresma (1978) and Amaral et al. (1999), according to Kormann et al. (2000). The number of blows of SPT was the mean of the two available tests. Tip resistance ($q_c$) and lateral friction resistance ($f_s$) of the employed CPTU values came from interpreting the chart in Fig. 2 regarding CPTU-8.

Data in Table 2 show a significant dispersion, suggesting the pile ultimate load capacity ranging between 567 kN and 1033 kN, depending on the semi empirical method applied. Furthermore, it can be seen that shaft friction contributes most to this wide range. The dispersion can be associated with the empirical factors (related to the soil and pile type, for example) considered in the load capacity predictions, which are influenced by the local geology, regional constructive and field tests practices from the database that was considered in the establishment of the method.

### 3.2. Measured results

In order to evaluate the mobilized resistance in four dynamic load test events, signals previously selected were submitted to conventional analysis using CAPWAP (CAse Pile Wave Analysis Program). This type of analysis is an iterative process that uses the wave propagation theory involving signals of force or velocity measured in the field as a boundary condition to match a curve modelled by soil parameters like: static resistance and its distribution along the pile shaft and under its tip and dynamic parameters of the pile and of the ground. Additionally, these analyses simulate the top and tip static load-displacement relationships (e.g. Likins et al., 1992; Hussein & Likins, 1995).

The study using the numerical analysis CAPWAP included the four final blows from the first test as well as three, four and two blows from the second, third and fourth tests, respectively. The selected values are justified by the higher total resistance (RMX) mobilized at the field for a specific blow (e.g. Fellenius et al., 1989) obtained by data generated in the Case Method data processor. A reduction in impedance was evidenced by the velocity signal being above the force signal at approximately two and three meters below the pile top, for all tests (Fig. 4). This reduction was modelled with slacks and impedance adjustments. Results of the analysis are summarized in Table 3.

The elastic deformation of the soil along the pile shaft ($\text{shaft quake} - Q_s$) did not show a clear behaviour when comparing all blows and tests (e.g. Alves et al., 2009). However, the elastic deformation of the soil at the pile tip ($\text{toe quake} - Q_t$) showed a clear increase as the drop height of the hammer was increased. Additionally, it could be observed that $\text{toe quake}$ values were close to the set values obtained for a specific blow. The high values obtained for the elastic deformation of the soil at the pile tip ($\text{toe quake}$) can be associated with the resilient behaviour (e.g. Aoki & Alonso, 1992), as well as with the pile being re-driven into the soil.

Comparing the blows with the same level of measured energy (EMX), it can be seen that the $\text{toe quake}$ of the re-driven pile tended to be lower than that measured at the end of the first test. For example, the fourth blow of the three initial tests (EMX ranges between 5.80 kNm to 7.50 kNm) and the fifth blow of the last test (EMX equals to 6.67 kNm) present decreasing $\text{toe quake}$ values, to be specific: 11.16, 5.34, 4.87 and 4.75 mm.
According to Smith (1960), soil quake is defined as the maximum elastic soil deformation. Therefore, a reduction in toe quake means a gain in tip stiffness. This behaviour was more pronounced between the first and the second tests, when tip resistance gain was 22% (from 337 to 412 kN). Among the other events (second, third and fourth tests), the reduction of its elastic limit was attenuated, as it also was for the set, reflecting in closer tip resistance values, specifically: 412, 402 and 399 kN, respectively. The decrease of stiffness gain and the small decreasing tendency of tip resistance observed in the last three tests suggests a stabilization behaviour at the tip.

Viscous forces which are function of velocity also resist pile penetration (PDI, 2006). Damping factors (shaft and toe damping) represent the dynamic component of the soil’s resistance. The results from CAPWAP analysis for shaft damping ($J_s$) and toe damping ($J_b$) did not show a clear behaviour (e.g. Paikowski & Chernauskas, 1996), the same happening with the shaft quake (e.g. Alves et al., 2009). The model that was best adjusted to the signals was Smith’s damping, for both moments: before and after the full mobilization of the pile tip resistance. This model yields good results in soils with high values of toe quake.

The Case Method damping factor ($J_c$) values, obtained by correlation with results from CAPWAP analysis, exhibited a small dispersion, as can be seen in Table 3 (last column). The mean values were equal to 0.61, 0.64, 0.56 and 0.64 for the first, second, third and fourth tests, respectively.

### Table 1 - Four dynamic load test records.

<table>
<thead>
<tr>
<th>Test</th>
<th>Blow</th>
<th>Drop height (cm)</th>
<th>EMX (kNm)</th>
<th>RMX (kN)</th>
<th>Set (mm)</th>
<th>Rebound (mm)</th>
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<tbody>
<tr>
<td>End of driving (EOD)</td>
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<td>20</td>
<td>0.8</td>
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<td>2.0</td>
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<td></td>
<td>2</td>
<td>40</td>
<td>2.4</td>
<td>550</td>
<td>2.0</td>
<td>2.0</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>60</td>
<td>4.3</td>
<td>570</td>
<td>5.0</td>
<td>3.0</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>80</td>
<td>7.5</td>
<td>605</td>
<td>11.0</td>
<td>4.0</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>100</td>
<td>9.4</td>
<td>597</td>
<td>13.0</td>
<td>4.0</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>120</td>
<td>11.1</td>
<td>618</td>
<td>15.0</td>
<td>4.0</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>140</td>
<td>13.5</td>
<td>604</td>
<td>20.0</td>
<td>5.0</td>
</tr>
<tr>
<td>Restrike after 113.5 h (4.7 days)</td>
<td>1</td>
<td>20</td>
<td>0.9</td>
<td>504</td>
<td>1.0</td>
<td>3.0</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>40</td>
<td>2.2</td>
<td>732</td>
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<tr>
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<td>3.4</td>
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<td></td>
<td>4</td>
<td>80</td>
<td>6.0</td>
<td>796</td>
<td>5.0</td>
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<tr>
<td></td>
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<td>100</td>
<td>8.4</td>
<td>799</td>
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</tr>
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<td></td>
<td>6</td>
<td>120</td>
<td>11.3</td>
<td>794</td>
<td>12.0</td>
<td>4.0</td>
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<tr>
<td>Restrike after 288 h (12 days)</td>
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<td>0.7</td>
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<td>1.0</td>
<td>3.0</td>
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<td></td>
<td>2</td>
<td>40</td>
<td>1.9</td>
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<td>3</td>
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<td>8.4</td>
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<tr>
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<td>7</td>
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<td>10.7</td>
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<td></td>
<td>10</td>
<td>180</td>
<td>17.8</td>
<td>855</td>
<td>20.0</td>
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<td></td>
<td>11</td>
<td>200</td>
<td>21.0</td>
<td>860</td>
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<td>Restrike after 2342 h (6.4 years)</td>
<td>1</td>
<td>20</td>
<td>0.71</td>
<td>542</td>
<td>0.0</td>
<td>2.0</td>
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<td>2</td>
<td>30</td>
<td>1.48</td>
<td>748</td>
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<td>5</td>
<td>100</td>
<td>6.67</td>
<td>984</td>
<td>4.0</td>
<td>4.0</td>
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</table>
Results from the analysis showed a significant resistance mobilized at all four dynamic load test events. Blows related with higher total resistance (RMX) from Table 1 (fifth column), were selected and analyzed using the CAPWAP program (Table 3), and did not accuse an increase in mobilized load capacity within the same event. At the first test, the full mobilized resistance occurred at blows 6 and 7, because of the higher and constant values obtained, such as 615 and 618 kN, with mean value equal to 616 kN. At the second test, the full resistance was mobilized at blows 4, 5 and 6, with values equal to 784, 750 and 767 kN and mean value equal to 767 kN. From these two tests an increase around 24.5% in total pile capacity along time (113.5 h ≈ 4.7 days) can be noted. At the third test, blows 4 to 8 were considered as achieving the full mobilized resistance, having values equal to 824, 819, 820, 825 and 833 kN, with mean value equal to 824 kN for total pile capacity. Comparing the first test with this third test, the increase in total pile capacity along time (228 h ≈ 6.4 years) was equal to 33.8%. At the fourth test, the blows considered were numbers 4 and 5, with values equal to 966 and 925 kN and average value equal to 945 kN for total pile capacity. Comparing one more time the first test with this fourth test, the increase of total pile capacity along time (2342 days/6.4 years) was around 53.5%.

The evolution of the total mobilized pile resistance over time can be seen in Fig. 6, where the pile shaft resistance and the mobilized tip resistance are also plotted. The plotted values are the mean of blows in which a full mobilization of resistance was considered. Figure 6 shows that the increase in total pile capacity was more expressive at the beginning (first days after pile driving), being equal to 24.5% during the first 4.7 days after EOD. Comparing the second test with the third test, the increase in total pile capacity along time was equal to 7.4% during the next 7.3 days. Then, comparing the third test with the fourth test, the increase in total pile capacity along time was equal to 14.7% during the following 2330 days/6.4 years.

The trend of the available data suggests that pile load (shaft and total) capacity was not yet stabilized along time at the moment of the last test. However, such behaviour cannot be confirmed due the significant time gap (in excess of 6 years) between the third and the fourth restrike.

Analyzing the increase of the resistance between tests, it can be seen that it is important to wait a minimum time before the installation of the pile to provide better information relating to the maximum available load capacity.

Although, ASTM D4945 (2012) doesn’t mention a specific waiting time, it recognizes that one of the factors that may affect the axial static capacity estimated from dynamic tests include the elapsed time since initial installation. Moreover, it states that if the test results are used for static capacity computations, dynamic measurements should (also) be performed during restrikes of the deep foundation, after waiting a period of time following the ini-

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**Table 2 - Predicted pile load capacity from semi empirical methods (Kormann et al., 2000).**

<table>
<thead>
<tr>
<th>Author</th>
<th>Field test</th>
<th>Ultimate load capacity (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Shaft</td>
</tr>
<tr>
<td>Aoki &amp; Velloso (1975)*</td>
<td>CPT</td>
<td>530</td>
</tr>
<tr>
<td>Aoki &amp; Velloso (1985, 1996)*</td>
<td>CPT</td>
<td>700</td>
</tr>
<tr>
<td>Philipponnat (1979)**</td>
<td>CPT</td>
<td>748</td>
</tr>
<tr>
<td>Bustamante &amp; Gianeselli (1982)**</td>
<td>CPT</td>
<td>375</td>
</tr>
<tr>
<td>Aoki &amp; Velloso (1975)*</td>
<td>SPT</td>
<td>390</td>
</tr>
<tr>
<td>Aoki &amp; Velloso (1985, 1996)*</td>
<td>SPT</td>
<td>520</td>
</tr>
<tr>
<td>Décourt &amp; Quaresma (1978)**</td>
<td>SPT</td>
<td>455</td>
</tr>
<tr>
<td>Amaral et al. (1999)</td>
<td>SPT</td>
<td>578</td>
</tr>
<tr>
<td>Mean</td>
<td>-</td>
<td>537</td>
</tr>
<tr>
<td>Standard deviation</td>
<td>-</td>
<td>135</td>
</tr>
</tbody>
</table>

tial installation sufficient to allow pore water pressure and soil strength changes to occur.

Pile Driving Contractors Association (2007) suggests the following minimum often used times between end of drive and restrike test: 1 day for piles in clean sands, 2 days for piles in silty sands, 3 to 5 days for piles in sandy silts, 7 days for piles in shales and 7 to 14 days for piles in silts and clays (longer times sometimes required). From the present case study, the significant increase in the load capacity until 12 days lends support to the waiting time suggested for piles in silts and clays.

In Fig. 6, the increase in total pile and shaft capacity along time showed to have an almost linear behaviour, because of the logarithmic scale used for time. Other authors found a similar behaviour in their graphic plots of field tests of piles, when performing the setup evaluation (e.g. Bullock et al., 2005a; Fellenius, 2008; Doherty & Gavin, 2013).

### 3.3. Side shear role on pile setup

Assessing the shaft resistance of the pile, an increase along time in those values from 279 to 354 kN is observed between the first and the second test, and from 422 to 546 kN between the third and the fourth test. At the first test, shaft resistance corresponded to 45% of the total capacity. At the second test it corresponded to 46% and at the third test it corresponded to 51%. In the last test, shaft resistance corresponded to 58% of total capacity. Therefore, an increase in shaft resistance along time can be observed. In contrast, pile tip resistance tends to stabilize over time (Fig. 7).

According to Lee et al. (2010), the major component of the pile bearing capacity gain along time is the gain in shaft resistance. Ng et al. (2013a) visualized the effects of setup along the pile shaft and at the pile toe in cohesive soils from field tests, with setup influencing the shaft resistance more than the end bearing did. Komurka & Wagner (2003), Bullock et al. (2005a, b) and Attar & Fakharian (2013) also observed the increase of capacity along time, mainly in shaft resistance.

In Fig. 7, the increasing pile shaft resistance along time vs. depth is displayed. As it is possible to see, this behaviour is compatible with the geotechnical investigation data (Figs. 1 and 2) and with the driven pile record (Fig. 3), which accused greater resistance with increased depth. Other authors found a similar behaviour in their graphic

![Figure 6 - Increasing pile capacity along time using log scale for time.](image_url)
According to Basu et al. (2014), based on results from one-dimensional finite-element analysis (FEAs), setup factors \((Fs)\) were observed to increase with time after pile installation and depend on both \(\sigma'\) (confining vertical effective stress) and OCR (overconsolidation ratio). \(Fs\) is defined as the ratio between the shaft resistance of the displacement pile available at any particular time \(t\) after pile installation and the shaft resistance of the pile immediately after its installation (end of driving - EOD).

Setup factors \((Fs)\) for shaft resistance were approximately equal to 1.0, 1.3, 1.5 and 2.0 respectively for the first, second, third and fourth average data of dynamic load test results, clearly showing the setup on the pile, as shown in Fig. 8. In this picture are also plotted the normalized results for total resistance over time, with ratios equal to 1.0, 1.2, 1.3 and 1.5, respectively for the first, second, third and fourth means of dynamic load test events. Other authors found similar behaviour in their graphic plots, such as Bullock et al. (2005a, b), Lee et al. (2010), Ng et al. (2013a), Attar & Fakharian (2013), Lim & Lehane (2014).

3.4. Logarithmic trend for pile setup

In agreement with Komurka & Wagner (2003), Lee et al. (2010), Steward & Wang (2011), Ng et al. (2013b), Afshin & Rayhani (2015) several empirical equations have also been proposed to quantify the magnitude of the pile setup. The most popular one was proposed by Skov & Denver (1988), who introduced a linear relationship between the logarithm of time vs. the pile setup. The equation is based on four case histories of dynamic and static load testing in driven piles on different types of soil, including clay, where the estimated pile capacity \(R_t\) at different elapsed times \((t)\) is obtained from the pile capacity \(R_{EOD}\) at the end of driving - EOD \((t_{EOD})\).

\[
\frac{R_t}{R_{EOD}} = A \log_{10} \left( \frac{t}{t_{EOD}} \right) + 1
\]  

(1)

According to Attar & Fakharian (2013), in the past, setup effects were attributed to both tip and shaft resistances and the total capacity would have been considered in the relation between \(R_t\) and \(R_{EOD}\), but recent studies have attributed the setup to shaft capacity and stated that effects on the tip are not significant (e.g. Bullock et al., 2005a, b; Attar & Fakharian, 2013). Parameter \(A\) is the slope of the line, so the higher this value is, the more vertical is the line and the greater the gain of resistance along time also is. This parameter is closely related to the properties of the soil where the pile was installed. For instance, the soil could be clay, silty or sandy soil, the field can be layered or not, naturally or not cemented, normally consolidated or overconsolidated, with or without the presence of water level. But not only are soil properties relevant, the period of time of the pile setup observation also has an important contribution to parameter \(A\) because, at shorter periods of time, piles usu-
ally increase their capacity faster than at longer periods of time, therefore, parameter $A$ changes value.

Different authors present diverse values for parameter $A$, ranging from 0.1 until 0.6 and different values for the time at the end of driving - EOD ($t_{EOD}$), ranging from 0.01 until 100 days (e.g. Komurka & Wagner, 2003; Bullock et al., 2005a, b; Fellenius, 2008; Lee et al., 2010; Steward & Wang, 2011; Doherty & Gavin, 2013; Ng et al., 2013a, b; Attar & Fakharian, 2013; Afshin & Rayhani, 2015).

In this paper, it was considered $t_{EOD} = 0.1$ day, because this showed to be the best agreement of both shaft and total normalized pile capacity ($R/R_{250}$) vs. normalized time at $\log_{10}(t/t_{EOD})$. This was confirmed by the coefficients of determination ($R^2$) as shown in Fig. 9, being in the range between 0.975 and 0.957. Parameter $A$ ranged between 0.217 and 0.131 for shaft and total pile capacity, respectively. The greater value of the parameter $A$ of shaft capacity comparing with parameter $A$ of total pile capacity can be explained by a likely trend to negative pore pressure generation during shear at the pile tip, normally associated to highly overconsolidated clays. This trend could imply a less pronounced increase of end bearing resistance over time.

In order to have a better idea about the driven pile setup in the Geotechnical Experimentation Site at UFPR, the mean pile setup measured during this study was compared to those proposed for piles driven in clay, according to results reported by Bullock et al. (2005a) and Afshin & Rayhani (2015). In comparison with previous research, the average setup presented in this study exhibits a slightly smaller rate over shorter and longer periods of time. Therefore, parameter $A$ is smaller. Anyway, the results from this study, in a normalized capacity vs. time trend curve, showed to be consistent with the response observed from a wider database of pile tests in clay compiled from the literature.

The most similar results are from field tests obtained from static and dynamic load tests as well as from tests using o-cell, for example:

i) Bullock et al. (2005a) performed tests in the coastal plain soils of Florida on different places. Testing sites varied widely, from shelly and silty sands to moderately plastic clays, in prestressed concrete piles ($A_{pmax} = 0.22$ for pile side shear, $t_0 = 1$ day and $t_{max} = 1727$ days = 4.7 years);

ii) Doherty & Gavin (2013), in the research field located at Belfast harbor, composed by soft clay, on driven concrete piles ($A = 0.26$ for pile side shear and 0.25 for pile total capacity, $t_0 = 100$ days and $t_{max} = 3683$ days = 10 years);

iii) Ng et al. (2013a, b), on layered cohesive soil, in the state of Iowa, varying from normally consolidated to slightly overconsolidated, in steel piles ($A_{pmax} = 0.11$ for pile resistance, $t_0 = 0.001$ day and $t_{max} = 36$ days);

iv) Attar & Fakharian (2013), in layered soil deposited in marine conditions, on prestressed concrete driven piles ($A = 0.32$ for shaft resistance, $t_0 = 0.01$ day and $t_{max} = 574$ days).

The parameters comprising the equation as well as the equation itself that was proposed to quantify the magnitude of the pile setup, with a linear relationship between the logarithm of time and the pile resistance, showed to be simple and consistent with the database from literature above and with results from this research for both total and shaft pile capacity.

3.5. Simulations of static load tests

The simulations of static load tests from the four dynamic load tests are displayed in Fig. 10. They were obtained from CAPWAP program. Analyzing the curves load vs. displacement it is possible to conclude that:

i) As time progresses, comparing all four events, the end portion of the curves goes to the right (higher value of load), something expected given the phenomenon of setup. This behaviour is mainly due to the recovery of the lateral friction;

ii) The rigidity of the first straight stretch (inclination) of the curves changed from the first to the last event, the changes being more pronounced when the first and the second events are analyzed. When the third and the fourth events are observed, the increase of rigidity tends to be smaller. This observation can also be associated with the increase of pile shaft resistance over time;

iii) The rigidity of the second straight stretch (inclination) of the curves also changed from the first to the last event, the changes being more pronounced when the first and the second events are analyzed. When the third and the fourth events are observed, the increase of rigidity tends to be smaller. At this time, this observation can be associated with the mobilization of pile tip resistance and the increase stabilizes along time;

iv) There is a clear trend of increasing the second straight stretch of the curves as the drop height of the hammer increases. Such behaviour is caused by the analyzed blows, which tend to mobilize similar total pile capacities, but as the drop height of the hammer increases (increasing the energy) the toe quake also increases.

3.6. Predicted vs. measured pile capacity

Comparing the results predicted for shaft resistance with the measured results of all four dynamic load tests, it can be concluded that the predicted results showed to be optimistic for the first (EOD) and second (4.7 days) events. In these cases, the predicted value for shaft resistance closest to the measured value refers to Bustamante & Gianeselli (1982) method from CPT. For the third event (12 days), the predicted values by Aoki & Velloso (1975) and Décourt & Quaresma (1978) methods, both from SPT, were the closest. Finally, for the fourth event (6.4 years), the closest value was predicted by Aoki & Velloso (1975) method.
from CPT and Aoki & Velloso (1985, 1996) method from SPT.

For tip resistance, the predicted values were lower than the measured results in all tests. The value predicted by Aoki & Velloso (1985, 1996) method from CPT was the closest to the value measured in the first event (EOD), as well as, it was for the results of the other events, in which the tip resistance reached the stabilization.

For pile total capacity, the predicted values from Bustamante & Gianeselli (1982) method from CPT and Aoki & Velloso (1975) from SPT were closer to the measured results for the first (EOD) event. Aoki & Velloso (1975) method from CPT achieved the closest value for the second (4.7 days) event. Aoki & Velloso (1985, 1996) method from SPT reached the closest value for the third event (12 days). To close, Philipponnat (1979) from CPT data obtained the nearest value for the fourth event (6.4 years).

Since the semi empirical methods presented scattered results, the dynamic load test showed to be an useful procedure in the load capacity assessment. Additionally, it showed to be a proper tool for assessing setup. In this way, as Bullock (2008) pointed out, later restrikes tend to provide greater reliability for setup trend analysis.

4. Conclusions

This paper investigated the setup behaviour of a driven pile with the use of dynamic load tests during almost seven years. The study focused on one prestressed concrete pile, installed in a stiff clay experimentation site. It is important to note that a single precast pile element was available for testing. Thus, caution is required in any extrapolation of the procedures or results here described due the lack of a broader statistical significance. Indeed, the conclusions do not apply to other pile types than driven piles.

The results indicated that the shaft resistance increased around 95% and the tip resistance tended to remain stable during the period of testing. The total pile capacity increased approximately 53%. In addition, it could be noted that the increase in pile total capacity was more expressive at the early stages (first days after pile driving). This evidence supports the need for a minimum rest time after pile installation for the acquisition of more reliable information related to the maximum available load capacity.

A linear, normalized, capacity vs. time at logarithmic rate relationship was established to quantify the pile setup process, with $t_{50}$ equal to 0.1 day and parameter $A$ equal to 0.217 and 0.131 for shaft and total pile capacity respec-
tively, which showed to be in agreement with the literature database.

In conclusion, the positive effect of the setup when incorporated into a reliability-based framework highlights its potential benefit for the design processes of driven pile foundations.

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References


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