Behaviour of Single Piles under Axial Loading

Analysis of Settlement and Load Distribution

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Abstract

The purpose of this thesis is to analyse the behaviour of single piles under axial loading, as far as settlement and load transfer mechanisms are concerned. It includes a literary review of two elastic theory-based methods, the Poulos and Davis method and the Randolph and Wroth method, as well as axisymmetric elastic modelling using the finite element-based program Plaxis. The results given by each method are organized in dimensionless charts of load-settlement ratio and proportion of load transferred to the pile base in terms of the pile slenderness ratio and the soil inhomogeneity and compared amongst each other.

This thesis also includes two comparison studies which involve axisymmetric elastoplastic modelling in Plaxis, considering the Mohr-Coulomb failure criterion. The first one is a comparison with two previous finite element simulations subject to very similar conditions: a GEFdyn 3D simulation and a CESAR-LCPC 2D axisymmetric simulation. It showed that in the simulation performed by Plaxis the value of load transferred to the pile base was lower than the others, although the total load-settlement curve was very similar in all three cases. The second one is a case study of the simulation of a static load test performed on a test pile. It includes the geological description of the site and the justification of the choice of parameters introduced in the model. Although limited information is available regarding the geological and geotechnical conditions of the site, the overall results were quite satisfactory.

Key words: axially loaded pile, numerical modelling, soil-pile interaction, Poulos and Davis, Randolph and Wroth

1 Introduction

Piles are deep foundations, necessary when the bearing capacity of shallow foundations is not enough to ensure the support of the superstructure. This superstructure results in vertical forces, due to its weight as well as additional loads, which are axially transferred to the pile and, through its shaft and base, to the soil, possibly reaching a stiffer layer.

The analysis of the load transfer mechanism in single piles under axial loading is therefore an essential basis for deep foundation design. It is very important that the physical interaction between pile and soil is carefully studied. The settlement analysis is also fundamental, for the maximum allowable settlement of a foundation is often the most important criterion in its design. Thus, it should be estimated accurately.

The behaviour of single piles under axial loading, as far as load distribution and settlement along the pile are concerned, have been analysed through numerous methods. They can be divided into three main categories, according to Poulos and Davis (1980).

1) Load-transfer methods, which involve a comparison between the pile resistance and the pile movement in several points along its length;

2) Elastic theory-based methods, which employ the equations in Mindlin (1936) for surface loading within a semi-infinite mass (such as the Poulos and Davis method), or other analytical formulations that impose compatibility between the displacements of the pile and of the adjacent soil for each element of the pile (such as the Randolph and Wroth method);

3) Numerical methods, such as the finite element method.
Elastic theory based methods, such as the ones presented in this work, do not explain the behaviour of the pile near failure. In this thesis, their results are used in comparison with the results of a finite element method program, Plaxis 2D version 8. Numerical methods are powerful and very useful tools when used carefully and calibrated with the appropriate tests. They also constitute a valuable way of performing a sensitivity analysis of the soil parameters.

This thesis has three main objectives:

1) To describe two elastic theory-based methods of analysis of single piles under axial loading;
2) To compare solutions given by finite element 2D elastic modelling of piles with the results of the elastic theory-based methods;
3) To perform finite element 2D elastoplastic modelling of single piles under axial loading, validating its results with former simulations and comparing them with a real case study.

2 Elastic Theory-Based Methods for Analysis of Single Axially Loaded Piles

In this thesis, two elastic theory-based methods are analysed: the Poulos and Davis method, introduced by Poulos and Davis (1968) and the Randolph and Wroth method, firstly described in Randolph and Wroth (1978). Elastic theory-based methods usually consist of dividing the pile into uniformly-loaded elements. Shear stress, \( \tau \), acts along the shaft, whereas normal stress, \( \sigma \), acts on the base of the pile. These are assumed to be uniform in each division, and the resultant is equal to the total applied load, \( P_t \). Equilibrium and compatibility between the displacements of the pile and of the soil adjacent to it are imposed for each element.

For both elastic theory-based methods, a cylindrical pile is considered: length \( L \), diameter of the shaft \( d \), pile radius \( r_0 \) and cross section area by \( A \). The soil is considered to be an ideal isotropic elastic mass, being the shear modulus, \( G \), the Young’s modulus, \( E \), and the Poisson’s ratio, \( \nu \), its linear elastic parameters that are not influenced by the presence of the pile. It is assumed that both the pile and the soil are initially stress-free. The total depth of the soil layer is represented by \( h \). The Young’s modulus of the pile, \( E_p \), is assumed to be constant.

There is an applied axial load \( P_t \) in the pile head. The settlement and load results refer to the applied load on the pile.

Since the conditions of this analysis are purely elastic, the interface between the soil and the pile is considered to be rigid – there is no relative movement between them. The relevant parameters that influence the vertical displacement, \( w \), of a floating pile under axial loading are stated in eq. (1):

\[
 w = f(P_t, L, r_0, E_p, G, \nu) \tag{1}
\]

The described conditions are used in this and the following chapter to compare elastic theory-based methods and the finite element method.

It is useful to have dimensionless solutions for the pile behaviour, so as to simplify and quicken their employment. Results from the methods subsequently presented are therefore arranged in dimensionless units, such as the pile slenderness ratio, \( L/r_0 \), the proportion of load transmitted to the pile base, \( P_b/P_t \), and the load settlement ratio, \( P_t/(w r_0 G_L) \).

**Poulos and Davis Method**

This method, firstly presented in Poulos and Davis (1968), allows a quick estimation of both the proportion of load which reaches the pile base and the total settlement of a pile in the conditions described in the last section. The load settlement ratio is expressed in terms of a coefficient, \( I \), as shown in eq. (2):

\[
 \frac{P_t}{w r_0 G_L} = \frac{4(1 + \nu)G}{L \times G_L} \tag{2}
\]

The coefficient \( I \) is function of the pile compressibility, distance to a hard layer and Poisson’s ratio of the soil.

The proportion of load transferred to the pile base, \( P_b/P_t \), for a floating pile may be calculated by eq.(3), first presented in Poulos (1972):

\[
 \frac{P_b}{P_t} = \beta_0 C_0 C_p \tag{3}
\]

This expression accounts for the pile compressibility and the Poisson’s ratio of the soil.

In order to account for the soil vertical inhomogeneity, an average Young’s modulus of the soil is used. This is a good simplification when the soil is divided into different layers but the Young’s modulus does not vary much.
A relevant form of soil non-homogeneity is one in which the shear modulus varies linearly with depth. A measure of this variation is the inhomogeneity factor, \( \rho \); it is calculated through eq. (4):

\[
\rho = \frac{G_{L/2}}{G_L}
\]  

(4)

The factor of inhomogeneity enables a comparison between different types of soil, with more or less vertical inhomogeneity.

**Randolph and Wroth Method**

This method, first introduced by Randolph and Wroth (1978) has been developed in order to explain the axial load transfer process between pile and soil. It is particularly useful in cases where the soil is non-homogenous. Initially, the shaft and base behaviours are studied separately. An imaginary horizontal plane AB at the depth of the pile base separates base and shaft; it is considered that above that plane the soil deforms due to the pile shaft only, and that below the plane the soil deforms due to the pile base only. The deformation of the soil surrounding the pile is similar to shearing of concentric cylinders.

Distinction is made between rigid and compressible piles. For rigid piles, the shaft settlement, \( w_s \), is independent of the depth, since there is no shortening. However, not always may the pile be considered to be rigid and relative displacement within the pile must be taken into consideration. Thus, for compressible piles, the shaft settlement will vary with depth.

The authors of this method have not provided any means of distinction between rigid and compressible piles. According to Fleming (1992), eq. (5) can be used as a general rule to determine if a pile is may be considered as rigid, in which \( \lambda \) is a soil-pile stiffness factor:

\[
\frac{L}{r_0} < \frac{1}{2} \sqrt{\lambda}
\]  

(5)

This method may account for vertical inhomogeneity of the soil, using eq. (4). Factor \( \eta \) refers to the stiffening effect of the soil above the loaded area, and usually adopts the unity. \( \mu_L \) is a measure of the pile compressibility.

Tables 1 and 2 shows the expressions of load settlement ratio and proportion of load transferred to the pile base for pile and soil type.

### Table 1: Load settlement ratio equations for different pile and soil type.

<table>
<thead>
<tr>
<th>Pile Type</th>
<th>Soil Type</th>
<th>Load Settlement Ratio</th>
</tr>
</thead>
</table>
| Rigid Piles | Homogeneous (\( \rho=1 \)) | \[
\frac{P_t}{Gr_o w_t} = \frac{P_b}{Gr_o w_b} + \frac{P_s}{Gr_o w_s} = \frac{4}{\eta(1-\nu)} + \frac{2nL}{\xi r_0} 
\] (6) |
|           | Non-homogeneous (\( \rho\neq1 \)) | \[
\frac{P_t}{Gr_o w_t} = \frac{4}{\eta(1-\nu)} + \frac{2nL \tanh(\mu L)}{\eta(1-\nu) \pi \mu L} 
\] (7) |
| Compressible | Homogeneous (\( \rho=1 \)) | \[
\frac{P_t}{G_L r_o w_t} = \frac{4}{\eta(1-\nu)} + \frac{2nL \tanh(\mu L)}{\eta(1-\nu) \pi \mu L} \left[1 + \frac{4L \tanh(\mu L)}{\eta(1-\nu) \pi \mu L} \right]^{-1} 
\] (8) |
|           | Non-homogeneous (\( \rho\neq1 \)) | \[
\frac{P_t}{G_L r_o w_t} = \frac{4}{\eta(1-\nu)} + \frac{2nL \tanh(\mu L)}{\eta(1-\nu) \pi \mu L} \left[1 + \frac{4L \tanh(\mu L)}{\eta(1-\nu) \pi \mu L} \right]^{-1} 
\] (9) |

### Table 2: Proportion of Load Transferred to the Pile Base equations for different pile and soil types.

<table>
<thead>
<tr>
<th>Pile Type</th>
<th>Soil Type</th>
<th>Proportion on Load Transferred to the Pile Base</th>
</tr>
</thead>
</table>
| Rigid Piles | Homogeneous (\( \rho=1 \)) | \[
\frac{P_b}{P_t} = \frac{1}{1 + \frac{P_s}{P_b}} = \left[1 + \frac{\pi(1-\nu) L \eta}{2\xi r_0} \right]^{-1} 
\] (10) |
|           | Non-homogeneous (\( \rho\neq1 \)) | \[
\frac{P_b}{P_t} = \frac{1}{1 + \frac{P_s}{P_b}} = \left[1 + \frac{\pi(1-\nu) L \eta}{2\xi r_0} \right]^{-1} 
\] (11) |
| Compressible | Homogeneous (\( \rho=1 \)) | \[
\frac{P_b}{P_t} = \cosh(\mu L) + \frac{\pi(1-\nu) L \eta \sech(\mu L)}{2\mu L r_0} 
\] (12) |
|           | Non-homogeneous (\( \rho\neq1 \)) | \[
\frac{P_b}{P_t} = \cosh(\mu L) + \frac{\pi(1-\nu) L \eta \sech(\mu L)}{2\mu L r_0} 
\] (13) |
Comparison Between Elastic Theory-Based Methods and the Finite Element Method in the Analysis of Single Axially Loaded Piles

Nowadays, the use of finite element method-based programs in structural and geotechnical analyses is generalised. In this chapter, the finite element method program Plaxis 2D version 8 is used to model single piles’ behaviour under axial loading. Its results are compared to the results given by the two described elastic theory-based methods, the Poulos and Davis method and the Randolph and Wroth method. Therefore, the applied conditions for those methods are simulated.

Curves of the load settlement ratio (fig. 1) and of the proportion of load taken by the pile (fig. 2) base in terms of the pile slenderness, are traced from the results provided by the finite element program. The analysed piles are cylindrical and isolated; the analysis is axisymmetric. Loading is performed axially through prescribed displacements. 15-node triangular elements are used to create the mesh. For the boundaries, standard fixities are used. The simulation is performed under drained conditions.

The model consists of two materials: the pile and the soil. The length of the pile, L, defines the rest of the geometry; its value varies between 4m and 100m. The pile radius, r₀, is given the value of the unity. The distance between the surface and the rigid layer, h, is 2.5L. This is an elastic analysis, and therefore it is considered that the materials do not yield. The soil-pile interface is rigid. The Poisson’s ratio of the soil is 0.15, the pile’s 0.3 and Young’s modulus of the pile, E_p, is 30GPa. The soil-pile stiffness ratio, λ, is constant and equal to 975.
4 Numerical Validation of Elastoplastic Modelling of a Single Axially Loaded Pile

In this chapter, a calculation performed through the finite element method-based program Plaxis is validated with a set of published results provided by two other programs, under the same conditions: CÉSAR-L CPC, by Neves (2001b) and GEFdyn, by D’Aguiar (2008). The soil parameters are determined in Neves (2001a).

The analysed pile is cylindrical and isolated, subject to axial loading, applied through prescribed displacements. The simulation is axisymmetric and 15-node triangular elements are used, in a total of 11559 elements and 95613 nodes. The model consists of five different materials: the pile, 10m long and 0.4m diameter, and four layers of granular soil. The water table is located 0.4m above the pile base. The total dimensions of the model are 10×20m², as used in the previous simulations.

Both the soil and the interface are modelled using the Mohr-Coulomb failure criterion. The interface strength factor, R_{inter}, is calculated based on the friction angle of the soil. These conditions are as similar as possible with the ones of the other simulations, the most significant difference being the interface, whose Young’s and shear modulus, friction angle and cohesion could all be manually entered in the other softwares. Table 3 shows the material properties used in the Plaxis simulation.

In the two previous analyses, the simulation was carried out as if no residual stresses due to the pile installation existed. Thus, the weight of the pile was completely transmitted to the base, no shear stress being developed along the shaft. In order to replicate these conditions, the initial stress field is generated from the different layers of soil only. The soil-pile interface is considered completely smooth in the first step (introduction of the pile with no loading). The displacement field is reset and the proper interface strength R_{inter} value is restored when loading is applied.

In the first calculation, a sensitivity analysis of the interface is performed, as extreme values of interface strength are tested: R_{inter}=0.01 and R_{inter}=1.00. Fig. 3 displays the results of this calculation, showing that the applied load necessary to cause a certain displacement is much higher when the interface is rigid. Interface strength has a much greater influence over the shaft load than over the base load. The load transmitted to the pile shaft is, as expected, almost null in the case of smooth interface.

Table 3: Material properties.

<table>
<thead>
<tr>
<th>Layer</th>
<th>L (m)</th>
<th>Material Model</th>
<th>Material Type</th>
<th>γ [kN/m³]</th>
<th>c' [kN/m²]</th>
<th>φ' (º)</th>
<th>ν</th>
<th>k₀</th>
<th>E [kN/m²]</th>
<th>R_{inter}</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile</td>
<td>10.0</td>
<td>Linear Elastic</td>
<td>Drained</td>
<td>24.0</td>
<td>-</td>
<td>-</td>
<td>0.3</td>
<td>-</td>
<td>29×10⁶</td>
<td>1.00</td>
</tr>
<tr>
<td>Layer 1</td>
<td>6.3</td>
<td>Mohr-Coulomb</td>
<td>Drained</td>
<td>16.7</td>
<td>13</td>
<td>26</td>
<td>0.12</td>
<td>0.562</td>
<td>9150.0</td>
<td>0.870</td>
</tr>
<tr>
<td>Layer 2</td>
<td>2.0</td>
<td>Mohr-Coulomb</td>
<td>Drained</td>
<td>18.8</td>
<td>12</td>
<td>23</td>
<td>0.12</td>
<td>0.609</td>
<td>13510.0</td>
<td>0.857</td>
</tr>
<tr>
<td>Layer 3</td>
<td>2.8</td>
<td>Mohr-Coulomb</td>
<td>Drained</td>
<td>19.8</td>
<td>14</td>
<td>23</td>
<td>0.07</td>
<td>0.609</td>
<td>13570.0</td>
<td>0.857</td>
</tr>
<tr>
<td>Layer 4</td>
<td>8.9</td>
<td>Mohr-Coulomb</td>
<td>Drained</td>
<td>20.0</td>
<td>17</td>
<td>23</td>
<td>0.05</td>
<td>0.609</td>
<td>19300.0</td>
<td>-</td>
</tr>
</tbody>
</table>

Fig. 3: Total load, load transferred to the pile shaft and load transferred to the pile base in terms of the settlement at the pile top, for smooth and rough interfaces.

Fig. 4: Total load, load transferred to the pile shaft and load transferred to the pile base in terms of the settlement at the pile top, obtained through Plaxis and GEFdyn.
In Fig. 4, the results of the second calculation, with the normal values of interface strength (shown in Table 3), are displayed. The curves of total load show very good correspondence. However, there is an evident discrepancy in the other two curves – results provided by Plaxis indicate that a lower proportion of load is transmitted to the base, i.e. the base is not as mobilized as in the other models. As proved in Fig. 3, changing the value of the interface strength, $R_{\text{inter}}$, barely alters these results, since it affects the load that reaches the pile base very little. In fact, the curve that corresponds to the maximum possible base load (smooth interface, $R_{\text{inter}}=0.01$) is still below the one given by GEFdyn, as shown in Fig. 5. Therefore, the calculated value of the interface strength may be considered acceptable, and the inconsistency of results must be explained by something else.

As an attempt to improve these results, a calculation including dilatancy of the soil beneath the pile base is performed. It is expected that if the soil at the pile base has a positive dilatancy angle, the base will be more easily mobilized as the load is applied, since the soil will expand. The introduced value for the angle of dilatancy, $\psi$, is the same as the friction angle, $\phi$ ($\psi=\phi=23^{\circ}$). Fig. 6 shows the results of this calculation, compared to the ones given by GEFdyn. This chart shows that introducing dilatancy as a soil property does indeed have the effect of mobilizing the base. In this case, an angle of dilatancy equal to the friction angle is still not enough to obtain results identical to the ones given by GEFdyn; however, it does approximate the base load curves, and the difference decreases with loading.

5 Case Study of a Static Load Test Performed on a Test Pile

In this chapter, a calculation performed by the finite element method-based program Plaxis is compared to the results of a static load test performed on a test pile. The static load test is part of a planned set of tests that support the design of the project “Modernização da Linha do Norte, Sub Troço 14 – Azambuja/Vale de Santarém, Viaduto de Santana do Cartaxo”, the construction of a viaduct. The static load test was performed by Mota-Engil, Engenharia e Construção, S. A. and the results were analysed by Instituto de Engenharia de Estruturas, Território e Construção do Instituto Superior Técnico (ICIST).

The geological prospection allowed the distinction of several soil layers, which constitute the lithological profile of the area. According to the report by Viaponte (2003), four different lithostratigraphic units are identified, following in the design of the project “Modernização da Linha do Norte, Sub Troço 14 – Azambuja/Vale de Santarém, Viaduto de Santana do Cartaxo”, the construction of a viaduct. The static load test was performed by Mota-Engil, Engenharia e Construção, S. A. and the results were analysed by Instituto de Engenharia de Estruturas, Território e Construção do Instituto Superior Técnico (ICIST).

The geological profile displayed in Fig. 7 was drawn by Mota-Engil as the test was performed, and it is used in the model.
The static load test (SLT) was performed on a cylindrical concrete pile with a total length of 40.6m and diameter of 80mm. The pile cap was built 2.1m below the soil surface, and the shaft was extended to the hard layer. This geometry is displayed in fig. 7.

A retrievable casing tube was necessary due to the position of the water table and to the properties of the first layers of soil, being extended to the depth of 32.0m. The casing was removed as the pile was being concreted. The concrete had a resistance class of C35/45 and a Young’s modulus of $E_p=33.5\text{GPa}$. The execution of the test began on January 20th, 2005 at 11.10 a.m. Care was taken to ensure that enough time had passed since the building of the pile to let the vertical displacements and the properties of the soil to stabilize. The maximum applied load was 8000kN, since that was the maximum value the reaction system could take. The test was performed incrementally, the loading and subsequent unloading being executed twice, first to service load (2800kN) and then to maximum load (8000kN). For the load measurement at the pile top, a pressure gauge was used; for the settlement measurement at the pile top, a set of dial gauges and transducers were used and for the displacement measurement along the pile shaft, and consequent determination of the mobilized resistance, vibrating wire extensometers were used, being welded to the reinforcing cage.

The static load test is modelled by the finite element-based program Plaxis, in order to compare its results with the ones provided by the different types of instrumentation. The geological profile in the model is identical to the one given by the static load test report (fig. 7), and the different layers of soil are related to the lithostratigraphic units described above. The analysed pile is cylindrical and isolated, subject to axial loading, allowing for a two-dimensional axisymmetric simulation. 15-node triangular elements are used, in a total of 9970 elements and 84399 nodes. The dimension of the model is 50×55m$^2$. Loading is applied through prescribed displacements.

The model consists of nine different materials: the pile and eight layers of soil. The pile is 40.6m long, with a 0.8m diameter. The water table is located approximately at the top of the second layer of soil.

The soil properties vary considerably from layer to layer, but there is a clear increase in consistency with depth. The pile is modelled as linear elastic, and every layer of soil follows the Mohr-Coulomb criterion.

Table 4 shows the pile properties and tables 5 and 6 display the drained and undrained soil properties.

### Table 4: Pile properties.

<table>
<thead>
<tr>
<th>Material</th>
<th>L (m)</th>
<th>d (m)</th>
<th>Designation</th>
<th>Material Model</th>
<th>$\gamma$  [kN/m$^3$]</th>
<th>$\nu$</th>
<th>$E$  [MN/m$^2$]</th>
<th>$R_{inter}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforced concrete</td>
<td>40.6</td>
<td>0.8</td>
<td>Pile</td>
<td>Linear elastic</td>
<td>25.0</td>
<td>0.3</td>
<td>33500</td>
<td>1.000</td>
</tr>
</tbody>
</table>

### Table 5: Drained soil properties.

<table>
<thead>
<tr>
<th>Layer</th>
<th>L (m)</th>
<th>Designation</th>
<th>Material Model</th>
<th>Material Type</th>
<th>$\gamma$  [kN/m$^3$]</th>
<th>$\phi$' (º)</th>
<th>$\nu$</th>
<th>$E$  [MN/m$^2$]</th>
<th>$R_{inter}$</th>
<th>$k_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3.0</td>
<td>Superficial deposit</td>
<td>Mohr-Coulomb</td>
<td>Drained</td>
<td>18.0</td>
<td>30</td>
<td>0.3</td>
<td>25.0</td>
<td>0.630</td>
<td>0.500</td>
</tr>
<tr>
<td>5</td>
<td>2.5</td>
<td>Gravel</td>
<td>Mohr-Coulomb</td>
<td>Drained</td>
<td>20.5</td>
<td>45</td>
<td>0.35</td>
<td>180.0</td>
<td>0.577</td>
<td>0.293</td>
</tr>
<tr>
<td>7</td>
<td>4.0</td>
<td>Dense sand</td>
<td>Mohr-Coulomb</td>
<td>Drained</td>
<td>21.5</td>
<td>40</td>
<td>0.25</td>
<td>200.0</td>
<td>0.500</td>
<td>0.357</td>
</tr>
</tbody>
</table>
Table 6: Undrained soil properties.

<table>
<thead>
<tr>
<th>Layer</th>
<th>L (m)</th>
<th>Designation</th>
<th>Material Model</th>
<th>Material Type</th>
<th>γ [kN/m³]</th>
<th>cₜ₀ [kN/m²]</th>
<th>cᵤ/z [kN/m²/m]</th>
<th>ν</th>
<th>E [MN/m²]</th>
<th>R₀</th>
<th>k₀</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>12.0</td>
<td>Soft clay</td>
<td>Mohr-Coulomb</td>
<td>Undrained</td>
<td>16.0</td>
<td>8</td>
<td>1.5</td>
<td>0.35</td>
<td>165.75</td>
<td>0.60</td>
<td>0.55</td>
</tr>
<tr>
<td>3</td>
<td>9.0</td>
<td>Medium clay</td>
<td>Mohr-Coulomb</td>
<td>Undrained</td>
<td>18.0</td>
<td>26</td>
<td>2.0</td>
<td>0.35</td>
<td>77.7</td>
<td>0.60</td>
<td>1.00</td>
</tr>
<tr>
<td>4</td>
<td>4.0</td>
<td>Firm clay</td>
<td>Mohr-Coulomb</td>
<td>Undrained</td>
<td>20</td>
<td>44</td>
<td>0</td>
<td>0.35</td>
<td>176.1</td>
<td>0.60</td>
<td>1.00</td>
</tr>
<tr>
<td>6</td>
<td>6.0</td>
<td>Very stiff clay</td>
<td>Mohr-Coulomb</td>
<td>Undrained</td>
<td>21.5</td>
<td>200</td>
<td>0</td>
<td>0.25</td>
<td>200.0</td>
<td>0.50</td>
<td>1.00</td>
</tr>
<tr>
<td>8</td>
<td>14.5</td>
<td>Hard clay</td>
<td>Mohr-Coulomb</td>
<td>Undrained</td>
<td>21.5</td>
<td>600</td>
<td>0</td>
<td>0.25</td>
<td>200.0</td>
<td>0.50</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Unlike the case of the last chapter, the information about each material type is very limited. In some cases, Viaponte (2003) suggests a value or a range of values; in others, there is no indication. Appropriate values were chosen in terms of the normal ranges for each type of soil and of the analytical resistance, calculated and compared to the resistance that resulted from the SLT. Table 7 shows the analytical base resistance, Rₗ₀, and the analytical shaft resistance, Rₛ, calculated from the equations in Santos (2008a, 2008b).

Table 7: Analytical shaft, Rₛ, and base, Rₗ₀, resistances of the soil.

<table>
<thead>
<tr>
<th>Layer</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rₛ (kN)</td>
<td>37.05</td>
<td>307.62</td>
<td>475.01</td>
<td>265.40</td>
<td>266.82</td>
<td>1507.96</td>
<td>642.50</td>
<td>1658.76</td>
<td>5161.13</td>
</tr>
<tr>
<td>Rₗ₀ (kN)</td>
<td>2714.34</td>
<td>2714.34</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The initial stress field is generated with the layers of soil only. The next step is to remove the volume of soil at the pile top. For modelling purposes, it is important that this excavation does not cause a failure mechanism in the slope; thus, an inclination of 26.57º is used (10m×5m). In order to simulate the introduction of the pile, the correspondent volume of soil is replaced, using the appropriate values of interface strength so as to simulate a non-displacement pile. As far as the loading is concerned, the settlements recorded from the test are introduced in the program, and the consequent load along the pile representing the output.

Since the goal is to compare the results with the ones recorded from the static load test, the “reset displacements” option is activated before the first load is applied, for the measurements refer to variations on settlement and stress that result from loading.

Fig. 8 shows the load-settlement curve obtained by Plaxis for both loadings. Fig. 9 shows the load-settlement curves for the first loading and fig. 10 display the load-settlement curves for the two loadings, both comparing results obtained by Plaxis and by the measurements taken in the SLT. Fig. 11 displays the load transferred to the pile base and shaft, according to the calculations by Plaxis, to the Randolph and Wroth method and the SLT.
Figure 10: Load at the pile top in terms of the total settlement at the pile top, given by the SLT and Plaxis, for both loading cycles.

Figure 11: Total load, load transferred to the pile shaft and load transferred to the pile base in terms of the settlement at the pile top for the second loading cycle, obtained through Plaxis, the Randolph and Wroth method and SLT.

Fig. 12 shows the load distribution along the shaft for service load (2800kN, load step 4) and maximum load (8000kN, load step 19). Fig. 13 show the mobilized shaft load for the different soil layers – the bottom one is not fully mobilized. As a complement to fig. 13, Table 8 shows the mobilized shaft resistance between different depth levels for the maximum applied load in each loading cycle.

Figure 12: Normal stress along the pile shaft for applied load of 2800kN and 8000kN, given by the SLT and Plaxis.

Figure 13: Shear stress between different levels along the pile, obtained by Plaxis.

Table 8: Mobilized shaft resistance.

<table>
<thead>
<tr>
<th>Between levels</th>
<th>Load Step 4 – 2800kN</th>
<th>Load Step 19 – 8000kN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SLT</td>
<td>Plaxis</td>
</tr>
<tr>
<td>2 and 4</td>
<td>15 to 25</td>
<td>31</td>
</tr>
<tr>
<td>4 and 5</td>
<td>100 to 110</td>
<td>32</td>
</tr>
<tr>
<td>5 and 6</td>
<td>15 to 25</td>
<td>28</td>
</tr>
<tr>
<td>6 and 8</td>
<td>20 to 30</td>
<td>27</td>
</tr>
</tbody>
</table>

The chart in fig. 13 appears to indicate that, between levels 6 and 8, the mobilized shaft resistance has not reached its maximum for the maximum applied load. Given that this corresponds to the last layer of soil (very consistent clay), it makes sense that the maximum resistance has not been reached yet.
6 Concluding Remarks

The purpose of this thesis was to understand the load transfer mechanism in single piles under axial loading, as well as to determine the load settlement relation.

Chapters 2 and 3 lead to the conclusion that the approximation used in the Poulos and Davis method used in non-homogeneous soils is an oversimplification, and its results are worse the lower is the inhomogeneity factor. The points which originate the Poulos and Davis curves are calculated from non-linear charts – most of them must be interpolated by using non-linear functions obtained through the few available points. It should also be taken into account that many parameters are regarded as being independent from each other when they are not. In spite of all this, this method is very convenient and adequate for practical purposes, considering its simplicity. In the Randolph and Wroth method for rigid piles, the load settlement ratio increases approximately linearly with the pile slenderness ratio. In spite of providing a good approximation, the solution for rigid piles has limited use. In this method, the non-homogeneity and non-linearity of the soil are more thoroughly taken into account than in the previous elastic theory-based methods, resulting in an expeditious way to estimate both settlement and load distribution. However, the Randolph and Wroth method for compressible piles is not applicable to very short piles. For the same applied load, vertical displacements calculated through Plaxis are higher than those calculated by the other methods.

In Chapter 4, it was concluded that the applied load necessary to cause a certain displacement is much higher when the interface is rigid (see fig. 3). The curves of the load transmitted to the pile base are very close, indicating that, as expected, the interface strength has a much greater influence over the shaft load than over the base load. Although the total load-settlement curves shows good correspondence, the difference regarding the proportion of load transferred to the pile base was not negligible. In the simulation performed by Plaxis, base resistance was not as mobilized, even if a smooth interface was considered; this indicates that the algorithm used in Plaxis might have significant differences to the other ones’. When dilatancy in the soil at the pile base was introduced, the base resistance was indeed much more mobilized.

The simulation performed in chapter 5, however, lead to satisfactory results. Given the little available information of the site’s geology, the results provided by Plaxis showed very good correspondence with the ones from the instrumentation used in the SLT. Not all the parameters indicated in the geological report were used, since some of them did not seem appropriate; the ones which were used, however, lead to good results. As for the parameters for which there was no available information, values of the generally used ranges were successfully applied. Nevertheless, figs. 8 to 13 account for the good behaviour of the model.

References