THE USE OF PILES IN GEOTECHNICAL WORKS

Case Study – Enlargement of the Hospital da Luz's underground parking

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ABSTRACT: Piles are used as deep foundation when surface soils do not have the appropriate mechanical properties to withstand the heavy loads of construction. The inaccessibility of these elements after their construction does not allow the immediate confirmation of their structural integrity. For this reason, certain tools which monitor the quality of these elements had been developed. This paper focuses on the use of static load tests in this context, and the interpretation of their results upon the Brazilian standard.

In another perspective of the utilization of piles in geotechnical works, these ones appear as an element of retaining walls. The growing occupation of urbanised areas has been leading to the need of taking advantage of the available subsoil space, often regardless the existing geological scenario and the boundary conditions. Therefore, the geotechnical interventions present themselves as a resource that allows the underground city growth.

To this extent, this paper is centered on the enlargement of the Hospital da Luz's underground parking as a case study, whose retaining wall consisted of a bored pile curtain, anchored all along and shored by concrete slab bands on one side. The existence of an instrumentation and observation plan had been crucial on the analysis of the displacements during the different stages of the construction process, and had also allowed a further comparison between the displacements monitored *in situ* and the ones obtained by a numerical modeling of the solution using the finite element software, *Plaxis 2D*.

KEYWORDS: static load tests; bored pile curtain; concrete slab bands; instrumentation; modeling; back analysis.

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1. INTRODUCTION

This study analyzes the use of piles in geotechnical works in two perspectives: piles as foundation element and piles as peripheral earth retaining element. The first topic was essentially developed in Rio de Janeiro, Brazil, during an exchange program IST-UFRJ. The second one is fundamentally based on a worksite in Lisbon, as a case study.

In situations where piles are used as foundations elements, the confirmation of their structural integrity is not immediate. As follows, the use of tests for this evaluation is indispensable in the quality control of these elements.

Among the verification tests of calculation criteria, static load tests prove to be essential when the objective is to investigate the ultimate resistant capacity of the piles, aiming a more rational design. Tests like this complement in some way the theoretical models, which do not take in account factors that can influence the load capacity of a pile, like the convenient control of the installation process.

The latest use of piles as peripheral earth retaining element may be a solution to the problem of the shortage of space in urban centers. The densified urban network configuration makes the construction of the infrastructure required by the demand of current society standards impossible. In this way, the adoption of underground structures becomes essential.

This space limitation arises, also, as an obstacle to the actual implementation of these structures, implying the vertically of the excavation, which reduces the surrounding space available for the construction.

Depending on the local geology, neighborhood conditions and the presence or absence of groundwater level, there are several types of constructive solutions to apply in peripheral retaining walls, such as bored pile curtains, diaphragm walls, Berlin or Munich walls, among others.

The implementation of these structures requires the control of their own safety and the safety of surrounding infrastructures, so that, the estimation of the behavior of these structures in the construction phase, during the design phase, is particularly important. In this regard, the adoption of advanced analysis tools, such as finite element modulation, contributes to a more realistic prevision, as it will be investigate within the presented case of study: the enlargement of the Hospital da Luz's underground parking.

2. Piles used as deep foundation

Structures transmit loads to the underlying ground through their foundations. If the foundation transfers building loads to the earth very near the surface, it is called shallow foundation. Otherwise, if loads are transferred to a subsurface deep layer because of the inadequate characteristics of the superficial ones, it is a deep foundation.

According to (Hachich, 1998), the phase of the foundations of a worksite is the one whose confirmation of the executed elements becomes more difficult due to the impossibility of viewing these elements. Thus, pile load test is a fundamental part of pile foundation design as it is an effective way to check the uncertainties in soil parameter measurement and design assumptions.

2.1 Static Load Tests

Static load test consists in the direct measurement of the pile head displacement in response to a physical applied test load. The main purpose of this type of test is to verify the expected structural behaviour in design (load capacity and settlements). The result is a curve load-settlement, which provides the pile settlement for each load level, therefore, it may be considered as a “stress-strain” test.

In Brazil, the current standard for the design and the execution of foundations is the **NBR 6122/2010**. In this standard is also defined the conditions to obtain the ultimate load from static load tests, which are:

- The load tests must be static;
- Static load tests must be defined on design stage and executed at the beginning of the works, so that, the design can be suitable to the rest of the piles;
- The load applied in static load tests must reach twice of the value of the design ultimate load.

2.2 Test Procedure and Devices

In general, static load test involves the application of static forces in successive stages and the direct measurement of the pile head displacements for each stage.

NBR 12131/2006 includes information about the devices for load application and measurement, test procedures and how the results must be analysed. According to this standard, the tests may be included in the following groups:

- SML (slow maintained load) – the load stages are maintained until the stabilization is reached;
- QML (quick maintained load) – load stages are maintained during a pre-established period of time (commonly 15 min);
- CLT (cyclic load test) or SCT (swedish cyclic test) – the designer predicts a pattern of loading and specifies the pattern for the test.

There is also a procedure proposed by Alonso (1997) and, currently, included in the mentioned standard which combines the SML with the QML procedures.

Concerning the devices, the reaction system must be based on the foundation element, so that the load application device is properly supported. The load application device acts against the reaction system via one or more hydraulic jacks. Regarding the static load compression tests, the different types of reaction systems are shown in Fig. 1.

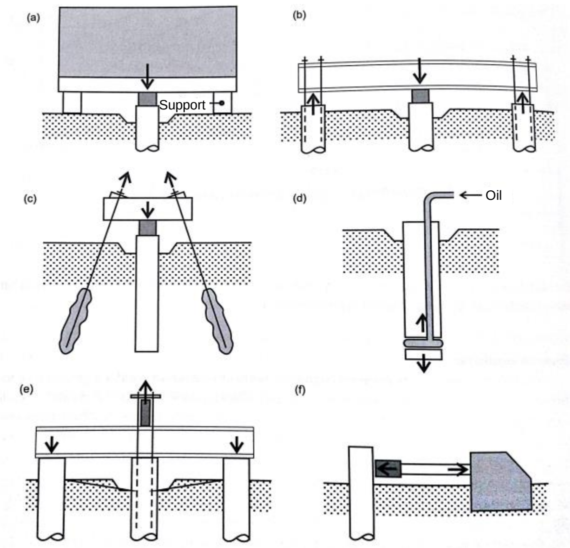

Fig. 1 - Reactions systems in static load tests (Velloso *et al*., 2012)

The loading is applied by a hydraulic jack that reacts against a reaction system, which can be:

- Weighing platform (Fig. 1a)
- Beams attached to nearby piles of the proof pile, which will be tensioned (Fig. 1b);
- Beams anchored on soil (Fig. 1c).

There is also an alternative procedure in which an expansive cell is introduced into the pile, as shown in Fig. 1d, and compresses the upper and lower parts of the pile. This cell is called Osterberg cell (O-cell).

Fig. 1e shows a traction pile load test, where the hydraulic jack reacts against the beams which are linked to the nearby piles.

At last, Fig. 1e presents a horizontal load test, where the hydraulic jack reacts against a nearby pile or a reaction block.

The pile head displacements are measured by four strain gauges installed on two orthogonal axes and fixed on reference beams. These devices also ensure the nonrotation of the pile head, which can occur if the alignment pile - hydraulic jack reaction system is not correct.

2.3 ULTIMATE LOAD DETERMINATION

As mentioned before, the result of a static load test is a load-settlement curve, which provides the pile settlement for each load level.

NBR 6122/2010 establishes that the load capacity in static load tests is defined when the pile collapses. This happens when continuous deformations of the pile occur, without increasing the load. However, not always a pile when submitted to a static load test collapses. Usually, for economic reasons or even insufficient reaction, the test is stopped. NBR 6122/2010 presents the two next circumstances for non-collapse of the pile:

a) The ultimate resistance of the pile is higher than the applied load during the test;

b) It is not possible to identify an ultimate load on the curve load-settlement, but settlements increase with the increasing load.

In these situations an extrapolation of the curve load-settlement should be done. The referred standard presents a method for the extrapolation (Fig. 2). According to this, the ultimate conventional load corresponds, on the load-settlement curve, to the conventional settlement obtained by the following expression:

$$\Delta r = \frac{P \times L}{A \times E} + \frac{D}{30}$$

Δr – Collapse conventional settlement [mm];

P – Ultimate conventional load [kN];

L – Pile length [m];

A – Transversal pile area [m^2];

E – Young's modulus;

D – Circumscribed pile circle diameter.

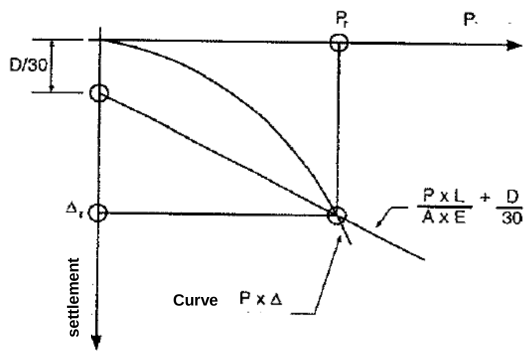

Fig. 2 - Ultimate conventional load (NBR 6122/2010 adaptation)

2.4 CURVE LOAD-SETTLEMENT INTERPRETATION

In order to apply the previous standard on the interpretation of a load-settlement curve, two cases of study were considered.

The first one involves the static load test made in one test pile before the construction of a condominium in Rio de Janeiro. It was a slow maintained load test in which the loading included 20 stages and the discharge 4 stages. The maximum load applied was 2891 kN, which matched at the end a settlement of 73,95 mm. After the discharge, the residual settlement took the value of 66,80 mm.

According to NBR 6122/2010, the pile did not collapse once it is not possible to identify an ultimate load on the curve load-settlement, but settlements increase with the increasing load - Fig. 3. In the same image, the green line represents the application of the method set by the mentioned standard for the ultimate load extrapolation. The line intersection with the curve is the ultimate load – 2432 kN.

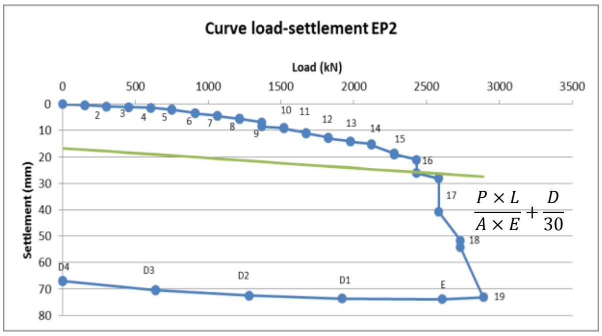

Fig. 3 - Pile EP 2 load-settlement curve

The second study case is a static load test done also in Brazil, in Rio Grande do Sul, at a wind complex worksite. It was a slow maintained load test in which the loading included 10 stages and the discharge 4 stages. The maximum load applied was 3081 kN, reached in stage 10. The settlement value for this loading was 25,22 mm. After the discharge, the settlement took the value of 18,74 mm (Fig. 4).

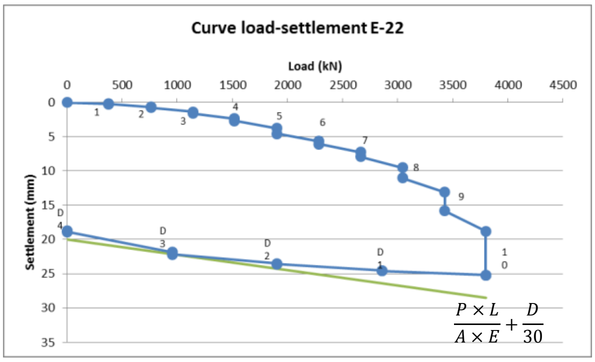

Fig. 4 - Pile E-22 curve load-settlement

In this test, the application of the method supported by NBR 6122/2010 does not materialize in obtaining an ultimate load value. This means that, according to the standard, the ultimate load is far from being reached.

In conclusion, the settlements required for the ultimate load definition must be appreciable, once the standard defines the maximum test load shall be twice the workload and, for that level of loading, strains not always reach values in the order of D/30.

The second study case, where the criterion presented by the standard does not lead to a specific result, another method could be applied for determining the ultimate load, in particular, the method of Van der Veen, which is a method widely used in Brazil.

In Portugal, there is no standard related to load tests. The guiding lines are defined in Eurocode 7.

According to Eurocode 7, the determination of the characteristic value of the ultimate resistance to compression capacity of a pile, $R_{c;k}$, from $R_{c;m}$ values, measured in load tests must take in account the variability of the soil and the effects of the construction method. These values will then be affected by certain correlation coefficients set in the Eurocode 7. The expression for $R_{c;k}$ is:

$$R_{c;k} = Min\left\{\frac{(R_{c;m})_{mean}}{\xi_1};\frac{(R_{c;m})_{min}}{\xi_2}\right\}$$

3. Piles as retaining wall elements

3.1 Flexible Retaining Walls

In the flexible support structures group, according to Peck (1972), are included all the supporting structures whose deformations induced by the soil pressure have a significant effect on the distribution of those pressures, as well as on the magnitude of the pulses, on bending moments and shear forces.

Flexible structures can be anchored or shored, depending on whether are used anchors, or struts, respectively, to their support. The flexible support structures may have, depending on the geometry adopted, one or several levels of support. The use of anchors presents itself as an advantageous solution, insofar as it allows more available space inside the excavation area, when compared to the use of struts. Consequently, all the construction works tend to occur more easily and more quickly.

In accordance with Matos Fernandes (1983), the various types of flexible support structures differ by their components, the materials and the constructive process. Regarding the materials used in such structures, reinforced concrete bored piles are often used in bored pile curtains and metallic piles in sheet piles retaining walls. Nowadays, modern methods of support and retaining of deep excavations have been applied, like, for instance, JetGrouting columns. The performance of this type of structures has been very positive, which makes this support solution economically competitive. Also, it is important to consider its incorporation into the final structure, and its function of foundation support, sealing and coating (Matos Fernandes, 1983).

3.2 Multi-anchored Retaining Walls

In this case, the struts are replaced by anchors which exhibit values of axial stiffness lower than the previous. This way, the question here is choosing the values of anchors' prestressing load that should be applied, in order to accomplish the desired behaviour of the structure. This behaviour is analysed by the values of the displacements of the flexible retaining structure, measured during the successive construction stages of the wall (Guerra, 2007). According to this same author, anchors work, essentially, by the alteration of the state of tension of the supported soil. Therefore, the following stages of the excavation are prepared.

3.3 Bored Piles Walls

Bored piles curtains are constituted by a set of reinforced concrete piles which are connected to each other through a caping beam at the top and some distribution beams in depth, depending on the number of anchors levels considered (Fig. 5). The piles are the main element of this type of retaining wall and are executed before the excavation.

Fig. 5 - Bored pile curtain in the works of enlargement of Hospital da Luz's parking

According to Meireles (2006) the most used construction methods are the continuous flight auger, the Kelly bar with recoverable tube and bentonite slurry.

These structures are mainly used when the neighbourhood conditions require special care and any disturbance may be critical.

4. Case study

This section is intended to frame the thesis's case study – the enlargement of the Hospital da Luz's underground parking, located in Avenida dos Condes de Carnide, in Lisbon. The new car parking has four underground floors and two connections with the original's hospital car parking on the lower floors (Fig. 6). This study is only focused on the excavation and peripheral temporary retaining structure stages.

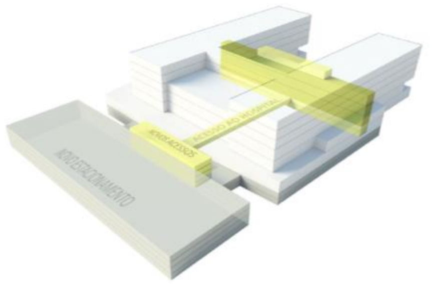

Fig. 6 - Connection between the old car parking and the new one (Risco, 2013)

4.1 GEOLOGICAL AND GEOTECHNICAL SCENARIO

The geological and geotechnical conditions were mainly determinate based on information obtained from some bibliography, like *Carta Geotécnica de Lisboa*, and geotechnical studies that had been done before the construction of Hospital da Luz, in 2002. Additionally, the following support jobs were carried out by *DeltaTau*:

- 3 mechanical boreholes (18 m deep) with SPT test, numbered SA1, SA2 and SA3;
- Installation of a standard piezometer in the SA3 borehole;
- Water analysis aggressiveness through chemical analysis.

From the interpretation of described tests, it was possible do establish the existence of 3 geological units (ZG1, ZG2 and ZG3). The geotechnical units' parameters are described in Table 1.

Table 1 - Geotechnical parameters of each zone

Geotechnical zone	**Description**	**γ (kN/m^3)**	**φ' (º)**	**c' (kPa)**	**E (MPa)**
ZG3	Landfill deposits	17	28	5	10
ZG2	*Prazeres* clays and limestones	19	30	40	20
ZG1	*Prazeres* clays and limestones	19	35	75	60

4.2 VICINITY CONSTRAINTS

The underground car park is located under a main road in Lisbon, which has a high volume of traffic. Therefore, alternative forms of circulation had to be created. The main vicinity constraints are represented in Fig. 7: the north constrain consist of a residential area, at south is located the hospital and, finally, at west is located the exit tunnel of the original car parking. The integrity of those structures must be considered during the excavation works and, also, during the construction of the bored pile retaining wall.

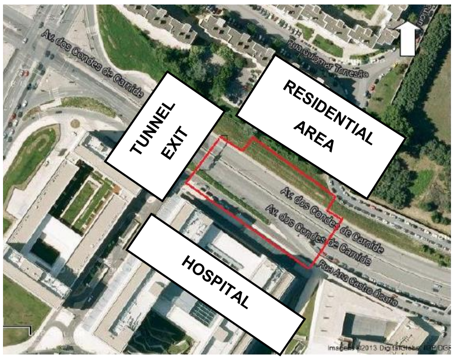

Fig. 7 – Main vicinity constraints

4.3 INSTRUMENTATION AND MONITORING PLAN

The instrumentation and monitoring plan reveals itself crucial on the execution of geotechnical structures. This plan allows to control the movements of the retaining structures, which are very sensitive to the displacements caused by soil removal, and the movements of the structures on the outskirts of the excavation area, as well. In this case study special attention must be paid to the exit tunnel of the original car parking, due to the possible impacts on it.

The conception of this plan at the design stage contributes to ensure safety in the work, insofar as it allows to confirm in construction works the assumptions considered on the design stage.

This plan also includes the alarm and alert criteria, whose purpose is to ensure that no excessive displacements are achieved in both retaining structures as in neighboring borders.

Besides the confirmation of the hypotheses adopted on the design phase, monitoring also allows optimizing the design with corrective actions, due to possible unforeseen situations. In this context, this plan may be considered as a security investment instead of an additional irrelevant cost.

4.4 EXECUTED SOLUTIONS

Given the existing constraints, specially the geological and geotechnical and the neighboring structures, it was advocated to the generality of the intervention area a peripheral retaining solution consisted of a set of bored piles curtains, with anchors all over it, except one side with concrete slab bands (Fig. 8):

- North and east elevations: anchored curtain of piles;
- West elevation: curtain of piles with concrete slab bands as locking system.

- South elevation: this section confronts directly with the existing diaphragm wall of the original car's hospital parking.

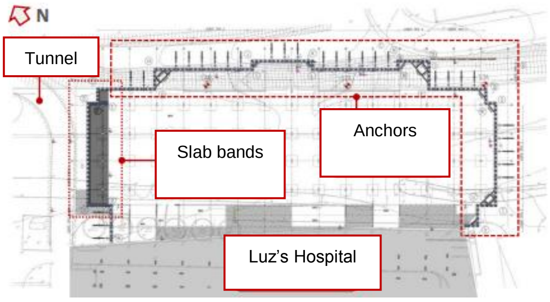

Fig. 8 – Worksite plan and the solutions adopted (Pinto *et al*., 2014 adaptation)

4.5 Construction processes

A special feature to highlight in the construction processes was the use of *Rodiostar* system. The bored piled wall was executed using *Rodiostar*, a development of the traditional continuous flight auger. The technological improvement brought some advantages like, for instance: more execution control during the pile concreting, acquisition and printing data automatic system that allows restarting the pile excavation in case of anomaly (Fig. 9); concrete with an improved formulation that leads to a gain of resistance, among others (Rodio, 2006a).

This new system introduces the reinforcement all along the pile length, which allows the execution of longer piles. If this does not occur, the reinforcement is removed and introduced again. The improved formulation of concrete also helps in this matter.

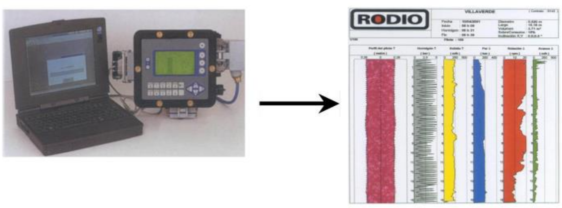

Fig. 9 - Rodio equipment and registration of execution parameters (Rodio, 2006a)

5. Solutions' modeling

5.1 Modeling

The executed solutions were modeled in the finite element program *Plaxis 2D*, which simulates the ground by dividing the domain in similar triangular finite elements with homogeneous individual characteristics. To simulate the two kinds of locking systems that exists in the case study, the two sections in Fig. 10 were chosen – Section 1 and Section 2.

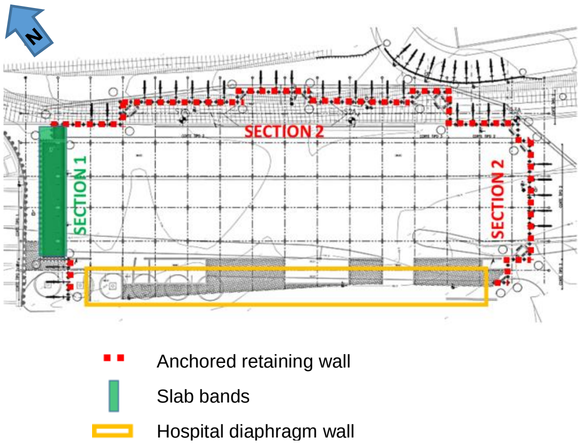

Fig. 10 - Worksite plan representative scheme

Due to the possibility of respecting the model geometry, as well as the structure and the geological settings, the results provided by this software are very close to the real ones. As a result, this modeling aims to compare the values of the displacements obtained by this numerical calculation software, with the monitoring displacements. As a consequence of this comparative analysis comes up its calibration, with the purpose of a back analysis. The back analysis consists of an improvement of the geotechnical scenario in order to approach the modeling results with actual results, in contemplation of study other constructive solutions potentially more economical.

The model used in *Plaxis* to simulate the soil behavior was the Hardening Soil, considering that is the model that most faithfully replicates the soil response. The Table 2 shows the parameters used to characterize the soils.

Table 2 - Hardening Soil parameters

Hardening Soil PARAMETERS	Geotechnical Zone		
	ZG3	ZG2	ZG1
Material type	Drained	Undrained	Undrained
γ_{unsat} [kN/m^3]	17	19	19
γ_{sat} [kN/m^3]	19	22	22
E_{50}^{ref} [kN/m^2]	10 000	20 000	60 000
E_{oed}^{ref} [kN/m^2]	10 000	20 000	60 000
E_{ur}^{ref} [kN/m^2]	30 000	60 000	180 000
c' [kN/m^2]	5	40	75
φ' [˚]	28	30	37
Ψ [˚]	0	0	0
m [-]	0,5	0,5	0,5

The soil layers, the retaining walls and respective anchors and concrete slab bands were defined according to the design. The both models of Section 1 and Section 2 are represented in Fig. 11 and Fig. 12, respectively.

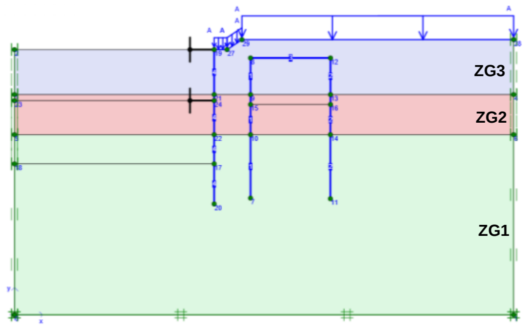

Fig. 11 - Section 1: Model geometry in *Plaxis*

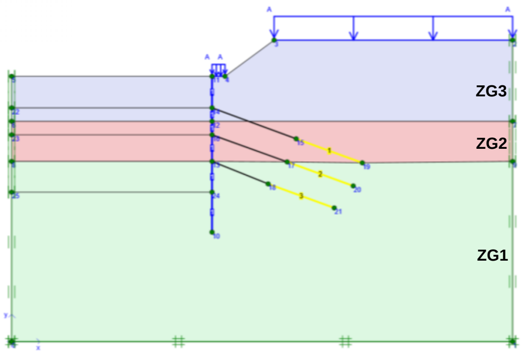

Fig. 12 - Section 2: Model geometry in *Plaxis*

Once the geometry model and the mechanical properties are defined, the next steps are the Calculations, where all constructive processes must be defined, and the Output results.

Fig. 13 and Fig. 14 show the deformed configuration for both Section 1 and Section 2, respectively, after the excavation. The horizontal movements are the most significant ones, becoming the main cause of concern and, so that, the main object of this analysis. In Section 1 the maximum horizontal displacement occurs at the top of the curtain, towards the inside of excavation and takes the value of 17 mm. In Section 2 the maximum displacement also occurs at the top of the curtain, towards the inside of the excavation, and takes the value of 25,5 mm.

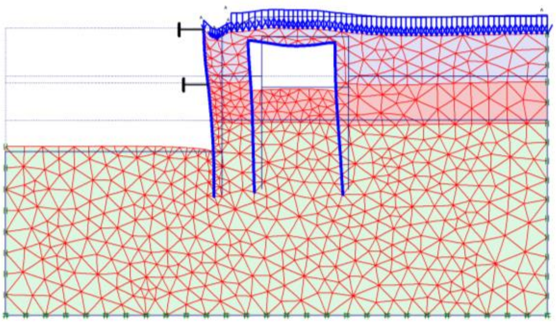

Fig. 13 - Section 1: Deformed configuration in *Plaxis* after the excavation

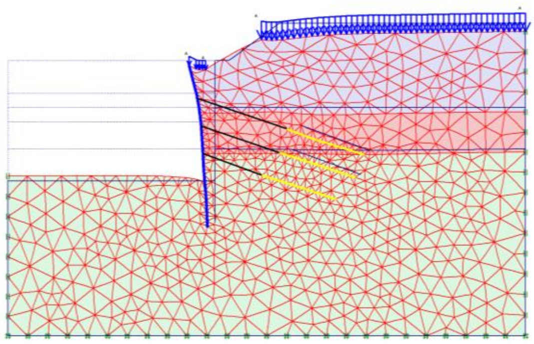

Fig. 14 - Section 2: Deformed configuration in *Plaxis* after the excavation

The efforts and the deformations on the curtain were also studied and it was found that these results were slightly higher than the ones obtained by the instrumentation. Because of this difference, a back analysis was carried out.

5.2 Back Analysis

After had analyzed the results of the instrumentation plan, and the range of values in which the displacements of the several elevations took part, it was chosen 15 mm as a valid and reasonable reference value for the instrumentation, in Section 2. This value is referred to the top of the curtain of the section were the displacements were maximum. Table 3 presents the values that were reached after several attempts, by changing each value, in order to understand better the behavior of the soil.

The modeling process was similar to the one that had been done before. The geometry and the structure properties remained the same, while the soil properties were changed in each attempt. The soil parameter that had more impact on the results was the Young Modulus. Nevertheless, friction angle was also reconsidered until the results provided by *Plaxis* were close enough to the real ones, measured in monitoring.

Table 3- Parameters of the soil used in the back analysis

PARAMETERS	**Geotechnical Zone**		
	ZG3	**ZG2**	**ZG1**
Material type	Drained	Undrained	Undrained
γ_{unsat} [kN/m^3]	17	19	19
γ_{sat} [kN/m^3]	19	22	22
E_{50}^{ref} [kN/m^2]	20 000	40 000	80 000
E_{oed}^{ref} [kN/m^2]	20 000	40 000	80 000
E_{ur}^{ref} [kN/m^2]	60 000	120 000	240 000
c' [kN/m^2]	5	40	75
φ' [˚]	30	35	37
Ψ [˚]	0	0	0
m [-]	0,5	0,5	0,5

The maximum horizontal displacements obtained at the top of the curtain after this study can be compared to the real ones in Table 4, where can be verified the similarities between the results of the instrumentation and after the back analysis.

Table 4 – Maximum horizontal displacements at the top of the curtain in Section 2

MAXIMUM HORIZONTAL DISPLACEMENTS (mm)	
Instrumentation	15,0
Plaxis – Initial modeling	25,5
Plaxis – Modeling after back analysis	15,3

The results obtained for Section 1 are presented in Table 5.

Table 5 - Maximum horizontal displacements at the top of the curtain in Section 1

MAXIMUM HORIZONTAL DISPLACEMENTS (mm)	
Instrumentation	12,5
Plaxis – Initial modeling	17,0
Plaxis – Modeling after back analysis	10,5

Based on these results, can be assumed the existence of a soil with relatively better properties than those considered initially. In addition, the obtained displacements are small enough to reconsider the executed solution and to propose an alternative one, more auspicious economically speaking.

5.3 OPTIMIZATION SOLUTION

The optimization proposal presented in this chapter focuses, mainly, on how the retaining wall is shored. Hence, it follows a study of the maximum horizontal displacements observed in the two situations described below:

- Section 1: Second level concrete slab band removal;
- Section 2: Increase the space between anchors.

NOTE: These alterations on the geometry model were done considering the soil parameters contemplated at the end of the back analysis.

Section 1

This elevation showed very small displacements when compared to the other elevations. Notwithstanding the neighbourhood conditions in this section, (the tunnel safety cannot be compromised), the displacements results observed justify an alternative economical proposal.

In this context, the second level of concrete slab band was removed by using a different geometry model in *Plaxis*.

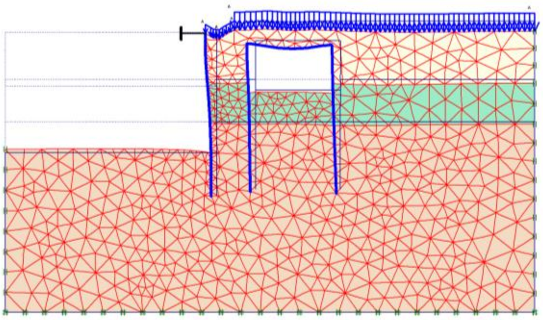

Fig. 15 - Section 1: Deformed configuration in *Plaxis,* after the second level concrete slab band removal.

The deformed configuration is similar to the initial solution (Fig. 15). On the other hand, the value of the displacements increased, as expected, once a shoring element was deleted. The maximum horizontal displacement occurred in the initial solution was 10,5 mm, while in this solution was 11,1 mm. It was also important to verify that the movements in the referred tunnel for this solution only increased 1 mm, which means that the security of this structure is guaranteed, in terms of admissible displacements.

Important to refer that, in case of adopting this solution, extra shoring elements should be designed, in order to guarantee the security of the structural assembly – slab band and metallic profiles.

Section 2

The spacing between the anchors was increased from 3,8 m to 5 m. However, the spacing between the piles and their diameter was established the same to maintain the arc effect, responsible for the soil stabilization.

The modification of the spacing between the anchors presupposes the alteration of the tensioning force value, per linear meter, for the three levels of anchors. After introducing this data in the software, the result (Fig. 16) show a slight increase of the displacements, as expected.

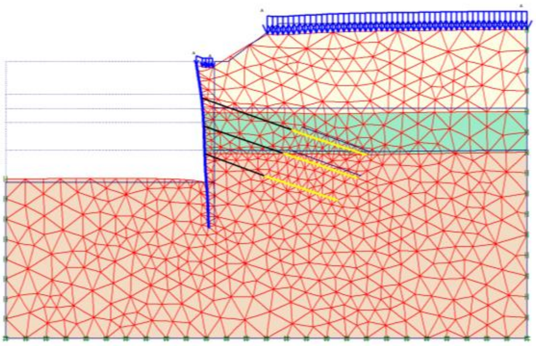

Fig. 16 - Section 1: Deformed configuration in *Plaxis*, after increasing the spacing between anchors.

The maximum horizontal displacement occurred in the initial solution was 15,3 mm, while in this solution was 17,4 mm. Considering the vicinity constraints, this displacements are acceptable, not despising a strict suitable monitoring plan.

5.4 ECONOMIC ANALYSIS

The proposed changes mean a reduction on the retaining wall costs, which will be evaluated in this section. The global cost of the bored piled wall executed, including its elements, is in Table 6.

Table 6 - Costs of the bored piled wall elements

ELEMENT	COST [€]
Bored pile wall	323 061,71
Distribution beams	159 740,59
Slab bands	16 707,03
Anchors	44 366,76
TOTAL	543 876,09

In order to simulate the cost of the new solution, it was admitted the following costs for the elements that would suffer an alteration:

- Slab bands: 8 853,51€ (cost of 1 slab band)
- Anchors: 34 283,41€ (cost of 17 units instead of the 22 initial units)

Not intending a very thorough analysis, it is although important to refer that these costs are very general. For instance, in the case of the slab band, this cost does not contemplate the extra shoring elements needed if the lower level slab band was suppressed.

Analyzing Table 6, it can be noted that the elements "slab bands" and "anchors" are not the ones with the most significant cost on retaining wall. However, by using this alternative solution, it is observed a decrease of about 30% of the combined value of slab bands and anchors, which is a very significant percentage. This final result is in Table 7, where both solutions' costs are compared.

Also important to mention the consequence of these alterations on the project duration, which would allow to save a lot of resources.

Table 7 - Analysis of the costs of slab bands and anchors in both solutions

INITIAL SOLUTION	Slab bands	16 707,03€
	Anchors	44 366,76€
	TOTAL	**61 073,79€**
ALTERNATIVE SOLUTION	Slab bands	8 853,51€
	Anchors	34 283,41€
	TOTAL	**43 136,92€**

6. Conclusions and Further Investigation

6.1 Conclusions

In the use of piles as deep foundation, static load tests are a very important instrument on the post execution control, even when the implementation process is rigorous. These tests allow to verify the expected structural behaviour in design (load capacity and settlements), contributing in some cases to the design optimization and, in others, to the early detection of anomalies. Both situations subscribe some way the compatibility between security and economy.

Plaxis 2D software, through the finite element method, allowed the modulation of the retaining solution adopted in the enlargement of Hospital da Luz's underground parking. The results of this modulation and of the subsequent back analysis were then compared to the instrumentation results. This software also conceded the modulation of an alternative solution using the result parameters of the back analysis, with the aim of optimizing the actual solution.

The alternative solution included two new geometric models where the number of anchors and the number of slab bands was reduced in each one. It was tested in terms of its own feasibility and admissible displacements.

After a brief economic analysis of the impact that these changes would have on the cost of the retaining wall, it was found that an intervention in the bored pile wall would have a more significant impact on reducing costs, due to its representation in the total cost of retaining. By acting at this level is meant the decrease in the diameter of the piles, for example, or increasing the spread between them. However, the spacing between piles should not exceed twice its diameter in order to maintain the arc effect, so this was not an option in this case.

Despite such alternative does not consider those elements whose cost reduction would have a greater impact on the global cost of the retaining wall, it still represents a significant reduction on the cost of the set of the elements that are being modified.

Finally, it is important to mention the multiple functions that piles present. In Portugal, where the geology is very heterogeneous, the foundations solutions that are adopted should quite fit the geological/geotechnical singularities of each scenario.

In this context, the piles which were introduced as a foundation element, present nowadays themselves as a crucial element of retaining walls. This was possible due to technological breakthrough that occurred in this area, together with the need to create underground infrastructure on the urban areas.

6.2 Further Investigation

It would be interesting to simulate the static load tests analysed in the finite element program *Plaxis* 2D, by varying some parameters of the soil, in order to understand the effect that these have on the ultimate strength of a single pile. In addition, both interpretations of the curve load-settlement refer to the ultimate limit states. Hence, it would also be interesting to do an admissible displacements study, taking into account the type of structure.

It is also suggested to use software that allows a complete 3D simulation of the retaining wall behaviour. This procedure would consider the whole area of excavation and not only the two elevations that were analysed, allowing the reproduction of the arc effect between the curtain piles.

The realization of a more detailed back analysis, as well as the use of other constitutive models for the soil, would allow to other several calculation assumptions. Additionally, the investment in a geological and geotechnical exploration plan which includes a greater number of surveys and also laboratory tests.

Other useful study, given the worksite location in the city of Lisbon, which turns out to be an area of great seismic risk, is a seismic risk analysis of solutions.

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