

Deep excavation for the new Central Library of Lisbon

Excavation profonde pour la construction de la nouvelle Bibliothèque Central de Lisbonne

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ABSTRACT

In this paper the main design and execution criteria for the solutions adopted for a 40 m deep excavation performed for the construction of 11 underground floors of the New Central Library and Municipal Archive of Lisbon are presented. The geological and geotechnical characteristics of the site are shown, as well as the geometrical and topographical conditionings. The adopted solutions consist of bored pile walls, jet grouting columns and reinforced shotcrete, supported by permanent and temporary anchors. Results of the finite element static and dynamic calculations are presented. The monitoring plan is described and some of the results obtained until present (January 2007) are shown.

RÉSUMÉ

Dans cet article les critères principales des solutions adoptés dans une excavation de 40 m de profondeur exécutée pour la construction de 11 étages enterrés pour la Nouvelle Bibliothèque Centrale et Archive Municipal de Lisbonne sont présentés. Les caractéristiques géologiques et géotechniques du local sont décrits, aussitôt que les conditionnements géométriques et topographiques. Les solutions adoptées consistent d'une parois aux pieux moulés, colonnes de jet-grouting et béton projeté, supportés par des ancres permanentes et temporaires. Des résultats des calculs aux éléments finis en statique et en dynamique son présentés. Le plan d'observation est décrit et quelques résultats obtenues jusqu'à présent (Janvier 2007) sont présentés.

Keywords: deep excavation, bored piles, jet-grouting, monitoring

1 INTRODUCTION

The new building for the Central Library and Municipal Archive of Lisbon (BCAML) is being constructed at Mouzinho de Albuquerque Av., in Lisbon (see Figure 1). The site is located at the South of the Santo Antonio valley, close to the Tagus river. It is an urban area with significant topographical level variations.

The BCAML building will have a rectangular geometry with a plan area of about 100×40 m². A total of seventeen floors will be built, including five underground ones. The main entrance to the building will be at the North-East side, from the Mouzinho de Albuquerque Av. and the other entrance will be located at the 5th floor at the South-West side, from Álvares Fagundes Street. Due to the geometry of the building, the topography of the site and the geotechnical and geological conditions, an excavation with an overall height varying between 40m at South-

West and 10m at North-East (see Figure 2) using three different solutions for the four sides of the excavation was designed and is being performed.

Figure 3 shows a general view of the site before excavation (a) and when the excavation depth was about 30 m (b). In Figure 3b the solutions adopted for the two main sides can be seen; they will be described in section 3.

2 MAIN CONDITIONS

2.1 Geological and Geotechnical Conditions

Results of the two geotechnical characterization programs performed show that the soil at the site is composed by fills with maximum thickness of about 20 m, underlain by Miocene heterogeneous materials composed by sands with silts and clays with intercalations of limestones. The geotechnical materials were classified into three main geotechnical zones:

- ZG3 – Layer of heterogeneous fills with thickness varying between 1,5m and 20,0m, with the major thickness at South - West, close to Álvares Fagundes St;
- ZG2 – Miocene sands and clays with fossils, with NSPT values under 60 and a maximum thickness of 19 m;
- ZG1 – Sands, clays and loam limestones with NSPT values over 60.

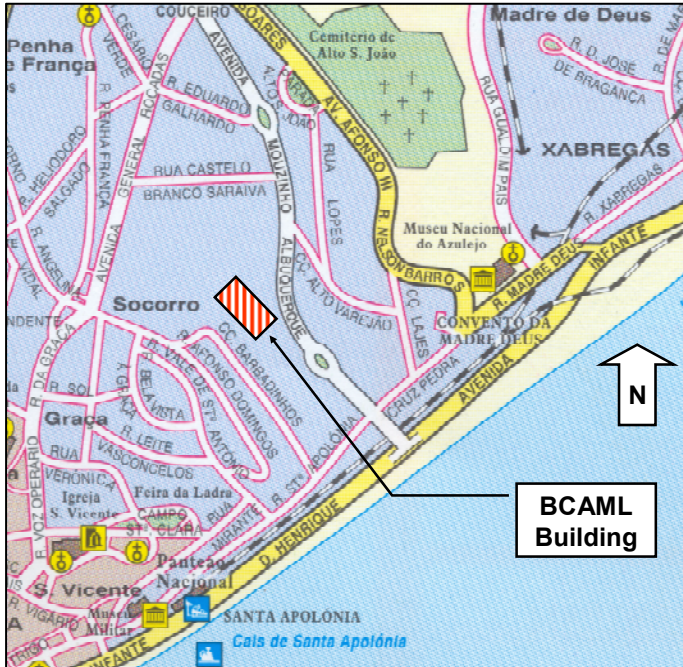


Figure 1. Plan view of the location of the excavation site.

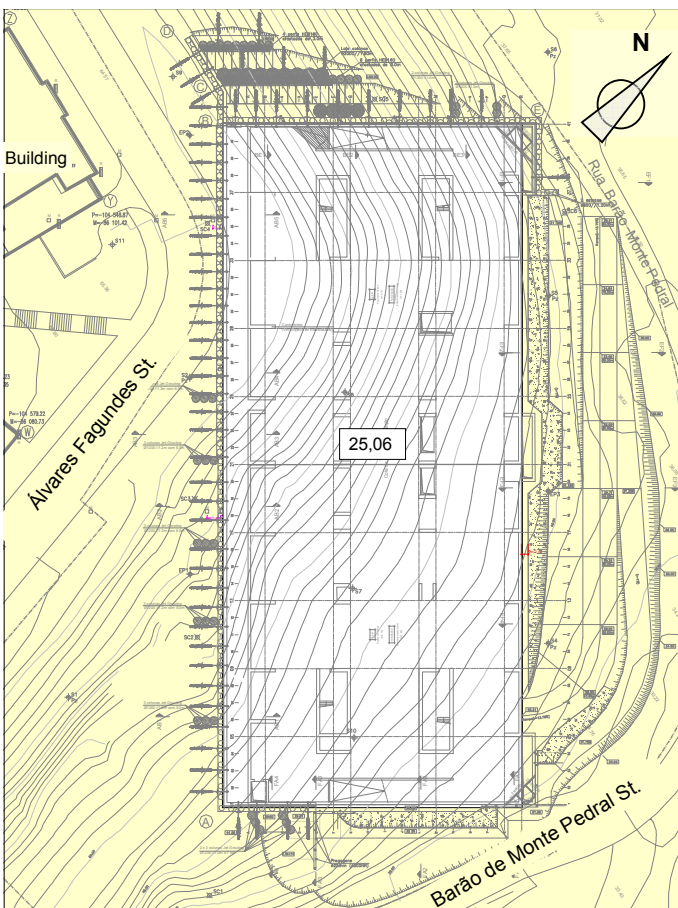


Figure 2. Plan view of the design of the excavation.



(a)



(b)

Figure 3. Site before excavation (a) and when the excavation depth was about 30 m (b).

Table 1 presents the geomechanical parameters adopted for the three geotechnical zones above mentioned. They were obtained by the interpretation of pressuremeter tests and uniaxial and triaxial laboratory tests.

Table 1. Main geomechanical parameters

Zone	N _{SPT}	γ (kN/m ³)	c' (kPa)	ϕ' (°)	E (MPa)
ZG1	> 60	21	80	45	140
ZG2	15 a 60	19	20	36	40
ZG3	9 a 30	17	0	25	7

2.2 Other Conditions

The adopted solutions were also chosen because of the following other conditions:

- The building could not support the earth pressures caused by the differences of the ground level at both longitudinal sides of the site (South-West and North-East).
- There are streets, services and structures nearby the construction site which performance could not be affected by the excavation.
- There are executive difficulties caused by the topography of the site.

3 SOLUTIONS

Three main solutions were adopted in the design project to support the excavations for the construction of the new building:

1. An anchored bored pile wall;
2. An anchored bored pile wall combined with a slope stabilization solution using jet-grouting columns;
3. A shotcrete nailed and drained excavation slope.

Solution 1 was used in the South-West side (Figure 4), where the maximum excavation depth is 40 m (including 20 m of fills). This solution corresponds to the main wall, which can be seen in Figure 3b). Where the excavation depth was greater than 36 m, the bored piles were 1 m of diameter, spaced 1.3 m between axes; for excavation depths less than 36 m, they were 0.8 m diameter and spaced 1.5 m. The stability of the wall was ensured by 10 levels of permanent anchors, sealed in the soil mass at ZG1. Its horizontal spacing is 2.6 m (excavation depth over 36 m) and 3.0 m (remaining area). Jet-grouting columns were performed between the piles in the depth of ZG2 and ZG3. In ZG1, shotcrete was used to protect the soil between the piles. Maintenance and anchor monitoring will be possible due to the distance of 3.0 m between the wall and the building structure, as shown in Figure 4.

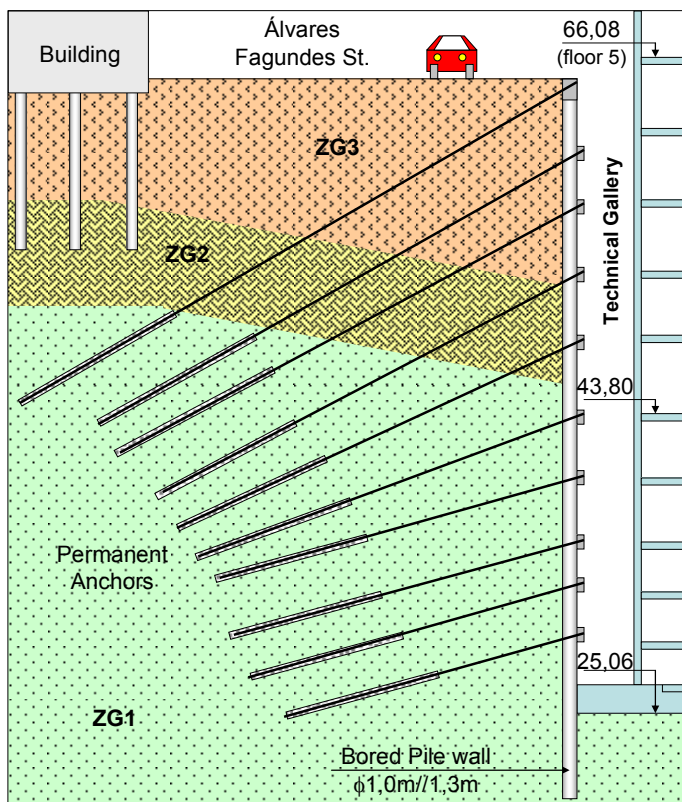


Figure 4. Cross section of the South-West side (solution 1).

Solution 2 was used in the North-East side (Figure 5). It can be seen in the right part of Figure 3b and in Figure 6. Piles are 0.8 m of diameter and the spacing between axes is 1.5 m. Temporary anchors were used to stabilize the wall. This pile wall was used below floor 0. Above floor 0 (ZG3) a slope stabilization solution using jet-grouting columns installed before excavation was used. Top concrete beams and temporary anchors were designed and performed.

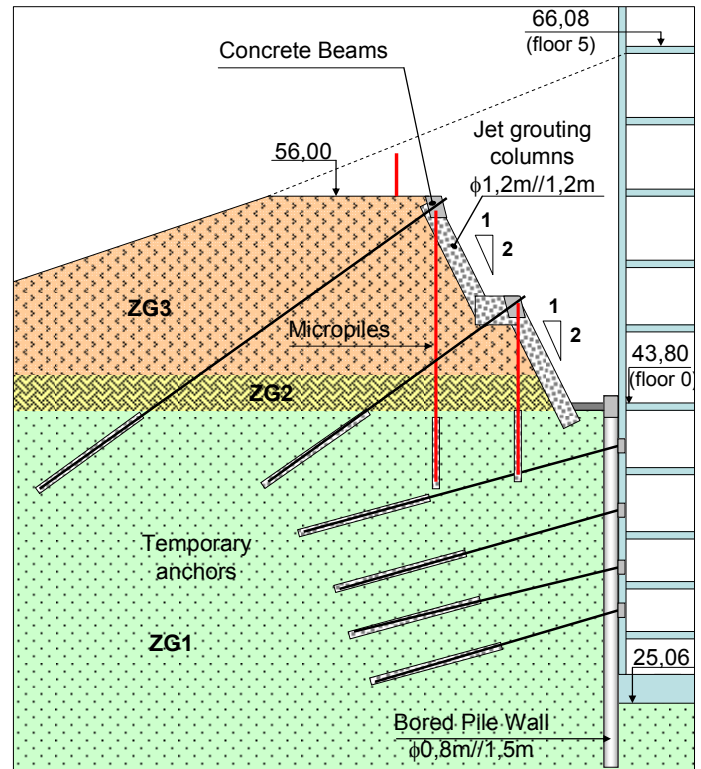


Figure 5. Cross section of the North-East side (solution 2).



Figure 6. Excavation on the North-East side (solution 2).

Solution 3 was used in both North-East and South-East sides, due to the relatively small excavation depth (10 m maximum). Near the East corner, a small shotcrete soldier-pile wall with diagonal struts was used, due to the street proximity (see Figure 2).

This solution is not yet implemented at present (January 2007).

4 DESIGN

All retaining structures were analyzed using a finite element numerical model (Plaxis V8.4). Internal forces in the retaining structure, displacements of the structure and of the supported soil and anchor load changes due to the construction were determined for each stage, using phased numerical calculations to simulate excavation and pre-stress. Design of the retaining structures was performed based on the results of the numerical simulations. Examples of displacements obtained for solution 2 are presented in Figure 7.

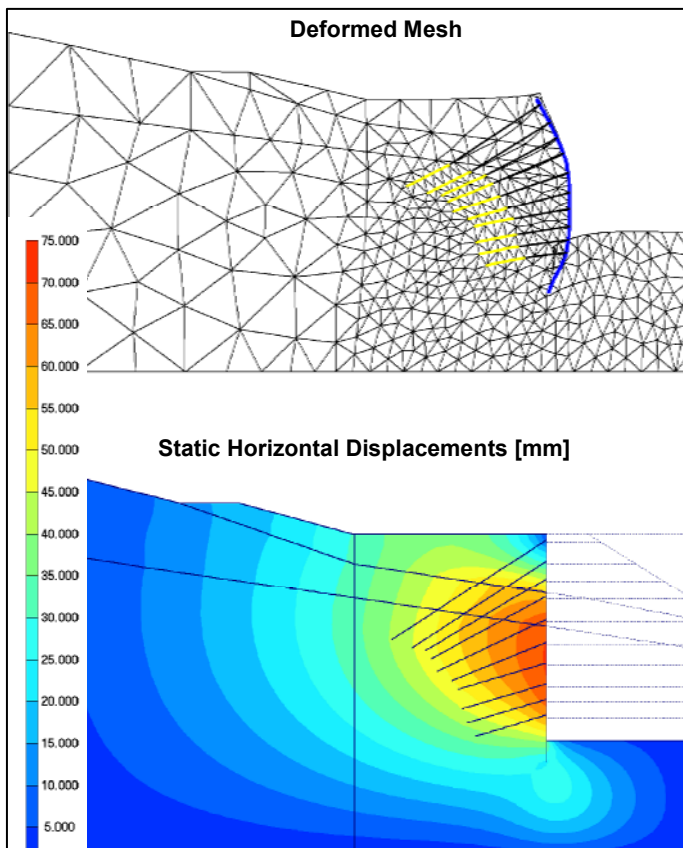


Figure 7. Example of results of static numerical calculation

Pseudo-static and dynamic analyses were performed using the same numerical tool for the South-West retaining wall, which will not be supported by the structure of the building. The seismic action was defined using the Portuguese code RSA. The construction site is located in seismic zone A, which has the highest seismic coefficient according to RSA. The geotechnical materials were classified as type II (hard to very hard cohesive materials and compact cohesionless soils). Type 2 seismic action was assumed, because of its conditioning nature for low frequencies: a greater magnitude and greater focal distance.

The pseudo-static analysis was performed using three seismic coefficients (β): 0.04, 0.10 and 0.16. A

hysteretic damping of 2 was assumed, due to the characteristics of the structure. The corresponding frequencies were 0.6, 1.0 and 1.2 Hz. Examples of displacements obtained are presented in Figure 8.

A dynamic analysis of the structure was also performed for the three values of the frequency above mentioned. Ten accelerograms were generated from a power spectral density function, which was obtained by a response spectrum defined by Eurocode N°8 (EN 1998).

Design of the permanent anchors took into account the demands of Eurocode N°8 regarding its resistance, its deformability and its geometry. Their resistance and deformability were confirmed by the results of the numerical pseudo-static and dynamic calculations. The geometry was previously defined considering a seismic free-length L_e increased, relatively to the static free-length L_s , by the factor $(1+1.5\beta)$. The coefficient β was considered in this factor equal to 0.1.

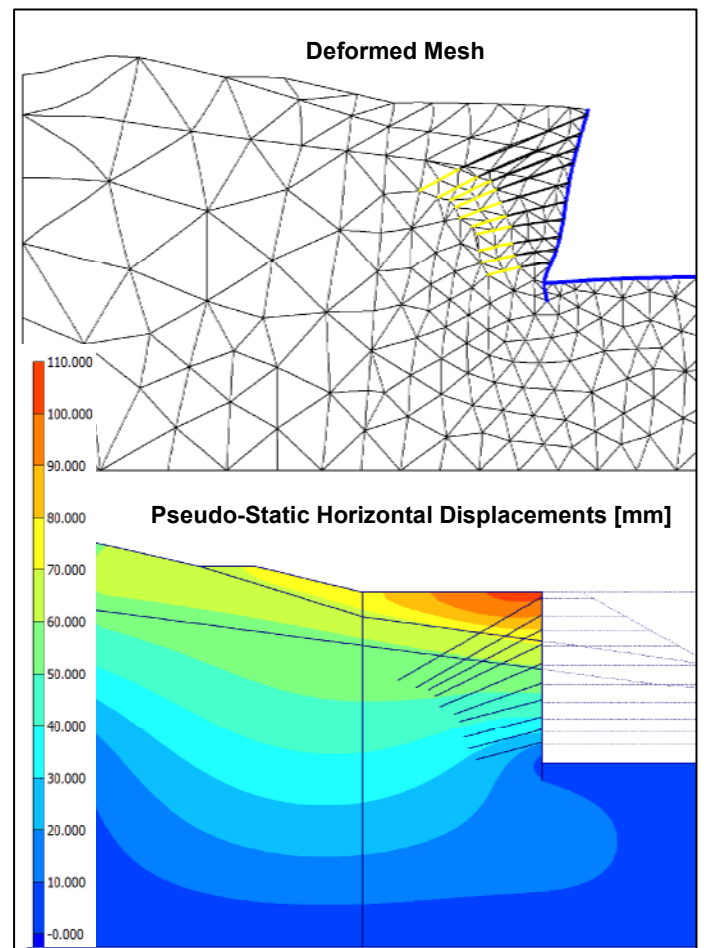


Figure 8. Example of results of pseudo-static numerical calculation

5 INSTRUMENTATION AND MONITORING

The instrumentation and monitoring plan was defined taking into account the main goal of performing the construction in safety conditions and economy, as well as the analysis of the behaviour of the

retaining structures and nearby infrastructures during the construction schedule.

The following apparatus were installed:

- 4 inclinometers;
- 28 topographic targets;
- 23 anchor load cells;
- 5 topographic marks on the buildings;
- 11 topographic marks in the nearby streets.

The location of the equipments is indicated in Figure 9. Measurements are being performed, at least, once per week until the construction of the building allows the temporary anchors to be deactivated.

Based on the calculation results, alert and alarm criteria were established for the measurements regarding the displacements of the retaining walls and the loads on the anchors. Reinforcement construction measures were established, in case the alert conditions occurred.

not final because excavation is not completed (33 m deep on the end of January 2007).

The inclinometer was installed at 21-06-2006 and at 23-06-2006 the anchor on top was pre-stressed. Measurements were performed the day after excavation phases indicated on Table 2. Works have been stopped between 23-10-2006 and 24-01-2007.

Table 2. Main phases of construction

Date	Phase - Excavation	Excavation Depth (m)
30-06-2006	1 st level	4.8
13-07-2006	2 nd level	8.8
09-08-2006	3 th level	12.8
31-08-2006	4 th level	17.1
06-09-2006	5 th level	21.7
25-09-2006	6 th level	25.6
09-10-2006	7 th level	29.6
01-02-2006	8 th level	32.6

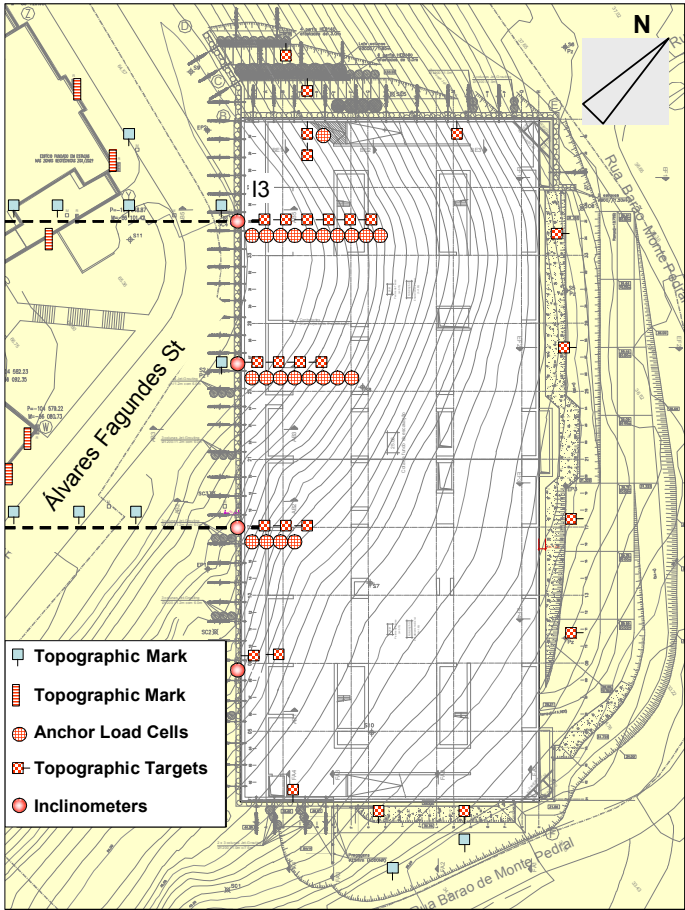


Figure 9. Location of the monitoring equipments.

An example of the displacements of the retaining wall is presented in Figure 10. In this figure, results of the displacements of inclinometer 3 (located in the South-West wall) in the direction normal to the wall are shown. Positive displacements are those occurring towards the excavation. Displacements are

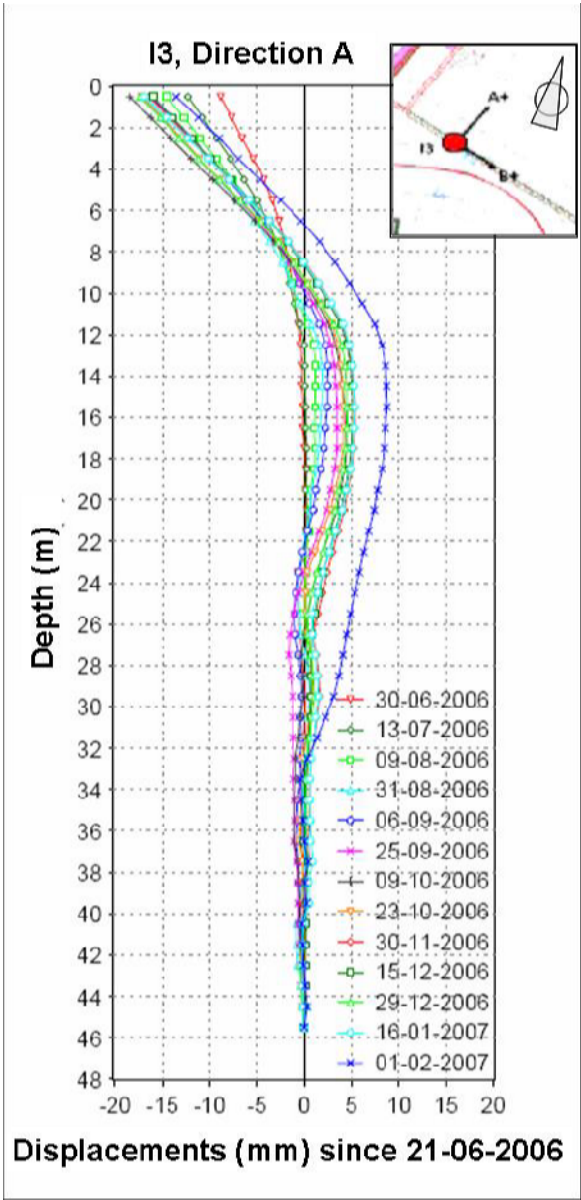


Figure 10. Results of the displacements in the direction normal to the excavation wall obtained from inclinometer I3.

Considering the depth of the excavation and the mechanical characteristics of the supported soils, the obtained displacements (which are confirmed by other inclinometers) are small ($\Delta h/H = 0.009/33 \approx 0.03\%$) and in general lesser than the design prediction.

Topographic measurements, although affected by some reading errors, roughly confirm these results.

Anchor load cells indicate, in general and until the present date, a quite small anchor load changes.

6 MAIN QUANTITIES

It is possible to point out the following main quantities regarding the geotechnical works:

- 2.175 m of bored pile Ø800;
- 1.600 m of bored pile Ø1000;
- 7.920 m of permanent ground anchors;
- 800 m of temporary ground anchors;
- 133.000 m³ of excavation.

All the ground anchors were tested previously to lock off according to the EN 1537 and were also pre-grouted before the installation inside the bore-holes (see figures 11 and 12).



Figure 11. Drilling works for the installation of permanent ground anchors



Figure 12. Internal pre-grouting of permanent ground anchors

7 MAIN CONCLUSIONS

In this paper the main design and performance issues of an unusual urban deep excavation, located in Santo António valley in Lisbon, were presented. The main adopted conditions and solutions were described and some monitoring results were presented. Those results show that, in general and until the present date, the obtained displacements were small and in general lesser than the predicted ones, confirming, together with the respect for the initial schedule and for predicted quantities, the overall suitability of the adopted solutions.

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