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Design and Behavior of Salgueiros Station for Porto Metro

António Topa Gomes
Universidade do Porto, Portugal

António Silva Cardoso
Universidade do Porto, Portugal

Jorge Almeida e Sousa
Universidade de Coimbra, Portugal

José Carlos Andrade
CJC – Engenharia, San Paulo, Brazil

Carlos Augusto Campanhã
CJC – Engenharia, San Paulo, Brazil

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António Topa Gomes
Faculdade de Engenharia da Universidade do Porto
Portugal

António Silva Cardoso
Faculdade de Engenharia da Universidade do Porto
Portugal

Jorge Almeida e Sousa
Faculdade de Ciências e Tecnologia da Universidade de Coimbra - Portugal

José Carlos Andrade
CJC – Engenharia
S. Paulo - Brazil

Carlos Augusto Campanhã
CJC – Engenharia
S. Paulo - Brazil

ABSTRACT

The paper presents the design and construction of an elliptical excavation for Porto Light Metro. Due to its original shape the solution took full advantage of the arch effect in the ground, adapting the Sequential Excavation Method to the vertical direction and achieving a novel, light and economic solution. In a first phase, the paper describes the construction method, referring also to economic and planning aspects of the excavation performed. The design is presented and discussed, including the presentation of the geotechnical tests performed and a brief analysis on the parameters assumed. The final part of the paper analyses monitoring results, focusing on the most surprising aspects of the behavior and on the main differences to the numerical calculations.

LOCATION OF THE WORKS

The excavation presented in the paper was performed during the construction of Porto Light Metro, and is located in the city center, in an abandoned football field. Figure 1 presents an aerial view of the excavation, extracted from *Google Earth,* with the coordinates of the center of the excavation.

Fig. 1. Location of the excavation in the city center – Image from Google Earth

The solution adopted (Normetro, 2003) was a cut and cover excavation with average dimensions in plant of, roughly, 80m by 30m and a depth of 22m. The initial proposal was to construct a diaphragm wall with several levels of anchors, solution adopted in several other stations of the Metro system. The large, and free, surface space opened the opportunity to implement a novel solution: contain the rectangular shape of the excavation within two ellipses, and taking full advantage of this new shape, by mobilizing the arch effect on the ground. The excavation was performed using the Sequential Excavation Method and the support materialized by a shotcrete membrane whose thickness varied from 0.30m, in the upper part, to 0.60m at the base of the excavation.

THE SOLUTION: CONSTRUCTION TECHNIQUE AND MAIN CHARACTERISTICS OF SALGUEIROS STATION

Description of Salgueiros Solution

Figure 2 presents a plan view of the site, with reference to the most important dimensions and the site tests performed. The shape of the excavation is the most important aspect of this novel solution. It should be, optimally circular or elliptical in order to take fully advantage of the arch effect on the ground, reducing to a minimum the thickness of the support.

The construction method is based on the idea of the Sequential Excavation Method but implemented it in the vertical direction.

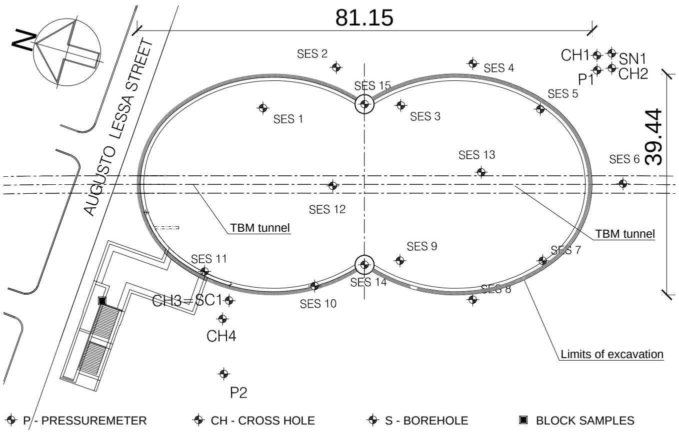

Fig. 2. Plan view of the station with the location of the most significant boreholes

The first step should be the construction of the capping beam, which has the function to guarantee an adequate stiffness of the structure while the ring is not closed. In the structure herein described this element has a cross section of $0.60\times1.00\text{m}^2$. The next phase is the excavation and construction of the support, step by step, till the bottom of the excavation. Figure 3 presents a schematic picture of the excavation sequence.

The excavation can proceed to the next ring only after the completion of the previous one. The height of these rings can very from work to work but heights between 1.0m and 2.0m are usual. The aspects determining this dimension are the characteristics of the ground and the dimensions of the excavation, either in plant either in height. Salgueiros Station had a ring height of 1.8m and a total excavation height of 22m.

Each ring can not be opened all at once, but subdivided in panels with limited width, depending on the dimension of the excavation and the ground quality. When the dimension in plant is big enough, there can be more than one panel open, at the same time, if distance enough between them exists. At Salgueiros Station this width varied from 6 to 12m and there were a maximum of 4 panels open simultaneously. Figure 4 presents a picture of one of these panels.

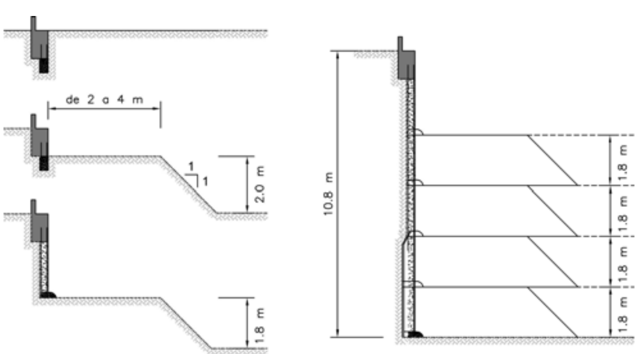

Fig. 3. Construction sequence for the Salgueiros Station

Fig. 4. Example of the excavation for a pane at Salgueiros Station

Immediately after the completion of the excavation of a panel, support should be provided in order to reduce to minimum the ground self support time and, consequently, the degradation of its properties.

The support is formed by shotcrete with 2 levels of wire mesh. This wire mesh is calculated in accordance with the stresses installed in the support, as a normal concrete section.

After the completion of a ring, the excavation proceeds ring by ring until the bottom of excavation is reached. Figure 5 presents the excavation in its final phase.

The general description herein presented fits in the perfection to continuous closed rings. In some situations, more or less important modifications to the standard solution may be looked for in order to adjust the shape of excavation to other needs. Salgueiros Station is one of these examples, as the dimension in plant for the Metro Station was $70\times30\text{m}^2$,

Fig. 5. Final excavation phase for Salgueiros Station

adjusting an ellipse to this rectangle would cause such an over excavation that the economy of the solution would be completely lost. The final shape was achieved by the intersection of the two ellipses, reducing to a minimum the unused area and maintaining the philosophy and advantages of this type of solution.

Figure 6 presents a comparison between the necessary area and the used area. The necessary area was $2100m^2$ and the area of the adopted bi-elliptical shape was roughly $2450m^2$, what represents an over excavation volume of 16%.

The intersection of the two ellipses produced unbalanced forces, which could not be equilibrated by the thin shotcrete membrane, adopted as support system. In order to equilibrate these enormous forces, a frame constituted by two circular columns of 3.50m diameter and a rectangular beam with cross section $1.60\times2.00m^2$ was casted *in situ* previously to the beginning of the excavation.

Figure 7 presents the dimensions of the most relevant structural elements. It must be refereed that these dimensions are mostly related to the excavation phase, period where the geotechnical design assumed its most relevance. The shell that forms the support has very thin elements, with thicknesses that vary from a minimum of 0.30m to a maximum of 0.60m. It should be emphasized that there aren't structural elements external to this shell, as anchors, nails or struts. This represent a great economy of materials and an enormous time saving as there isn't the need to stop the works in order to change equipments or wait for the strengthening of certain materials.

<u>Scheduling of the construction</u>

Figure 8 presents the real scheduling of the works. The time for the total excavation, around 55,000 $m^{3,}$ was roughly 36 weeks, what proves the efficiency of the solution. Nevertheless, more that one third of this time was dedicated to the construction of the vertical shafts, with 3.5m diameter, due to the natural slowest beginning of the works. After this phase, the construction time was 5 approximately months.

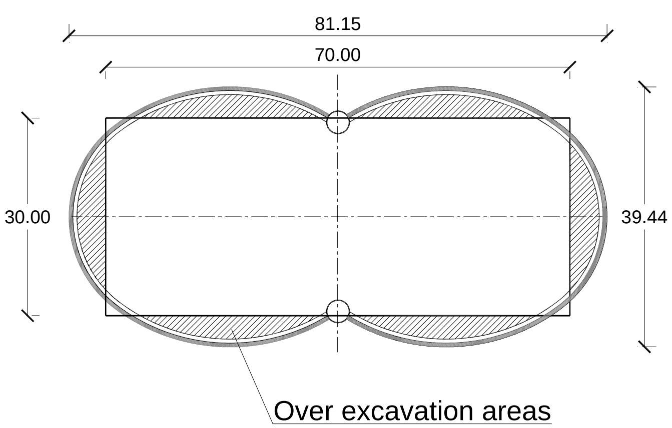

Fig. 6. Comparison between the necessary area and the used area in order to maintain an elliptical shape

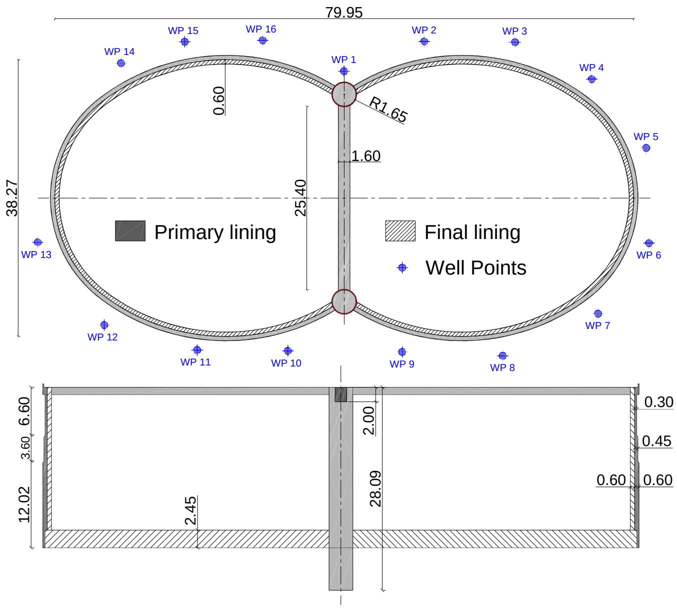

Fig. 7. Geometry of the most relevant structural elements of the excavation

<u>Drainage during construction</u>

The solution was implemented in the granite residual soils, material that has certain cohesion and the suction which level may have a significant contribution its stiffness and strength of the soil. In such conditions the drainage of the excavation, assuring that the water table is constantly below its base, assumes vital importance in the implemented solution.

A previous lowering of the water table was achieved by the construction and dewatering through 16 well points, all around the excavation, 2m distance from the retaining wall, whose position can be observed in Figure 7.

These well points had an internal diameter of 0.20m and a perforation diameter of 0.40m. The annular space between the inner PVC tube and the outer part of the perforation was

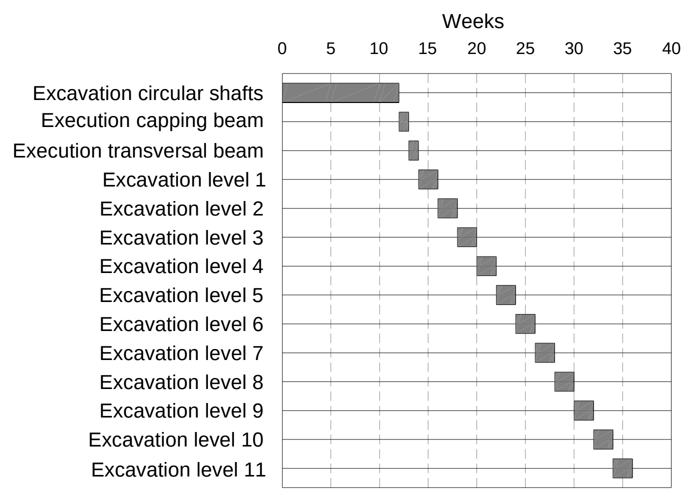

Fig. 8. Detailed validated scheduling of the works

fulfilled with granular material. The total depth of the wells was 31m and the installed submersible pumps had a capacity of 3m^3/h.

As redundant and security procedure, short sub-horizontal drains with 2 inches diameter and a total length of 4m were installed during the excavation. Proceeding in this manner, an additional guarantee of the unsaturated condition was achieved in the material in contact with the support.

The analysis of the results of the piezometers confirmed the effectiveness of the lowering, being the water table constantly below the bottom of the excavation.

GEOTECHNICAL CHARACTERIZATION

Due to the innovative character of the solution, an intense program of geotechnical characterization, both, *in situ* and laboratory, took place. Additional and careful geotechnical characterization was performed for the PhD thesis of the first author (Topa Gomes, 2009), a research mostly devoted to this novel solution and interpretation of its behavior in granite residual soils.

Brief description of granite residual soils

Granite residual soils cover a large area of the northern part of Portugal, being the material predominant at most superficial horizons in the city of Porto, where the excavation here presented was performed. These soils, well characterized by Viana da Fonseca (1996, 2008) and Topa Gomes *et al.* (2008) are saprolitic materials, preserving the natural fabric of the original rock, with less than 10% clay, around 20% silt and almost 70% sand. A small percentage of gravel also appears. Figure 9 presents the results of 12 granulometric curves performed on samples collected during the boreholes for site

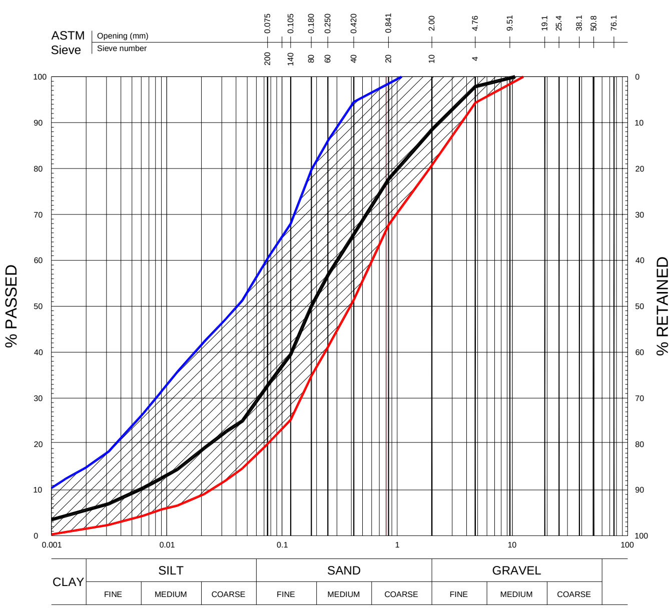

Fig. 9. Granulometric curves of the residual soil obtained from the samples collected on the boreholes

investigation. The soil may exhibit, in certain cases, some plasticity.

Most superficial materials have a weathering degree where the structure of the material completely disappeared. As the weathering degree decreases, the massif maintains the fabric of the original rock but with strength and deformability typical of soils.

The granite mass has sudden changes in its weathering degree, being completely erratic the changes both in vertical and horizontal directions. This fact leads to significantly different results between adjacent tests, creating an increased difficulty to define a reliable geotechnical model.

Field tests

The plan view presented in Figure 2 also represents the location of the several field tests performed for Salgueiros Station. It includes the tests performed during the design phase and the ones performed later, during the construction. Also presented is the local where good quality block samples were collected in order to carry out representative triaxial tests.
Table 1 presents a brief description of the number of tests performed and the maximum depths reached.

Table 1. Summary of the field tests performed

Test	Number Plant locations	Max. Depth	Total individual results
Boreholes	17	25	-
SPT	16	23.5	146
Ménard Pressuremeter (PMT)	3	24	13
SBPT	2	16	11
Cross-Holes (CH)	2	25	16
Lefranc Permeability	9	24.5	52

Below the depth of 12m, roughly, the evolution with depth of the material conduced to rock like materials, with weathering degrees W4 or W3.

Figure 10 presents the results of G_0 derived from the CH tests, performed in two opposite locations of the excavation. Below the depth of 10/12m, roughly, there is a clear increase in the values of G_0 values. This difference marks the transition between soil like material and rock like materials. The importance of these first 10/12m in the overall behavior of the excavation is of paramount importance as they correspond to the less resistance and more deformable materials. Hence, part of the results presented from hereon will focus on these first horizons.

For the first 12m, Figure 11 presents the average of the SPT tests and the average values from the CH tests. Below this depth, the average value of N_{SPT} was systematically above 100, although local values where significantly smaller.

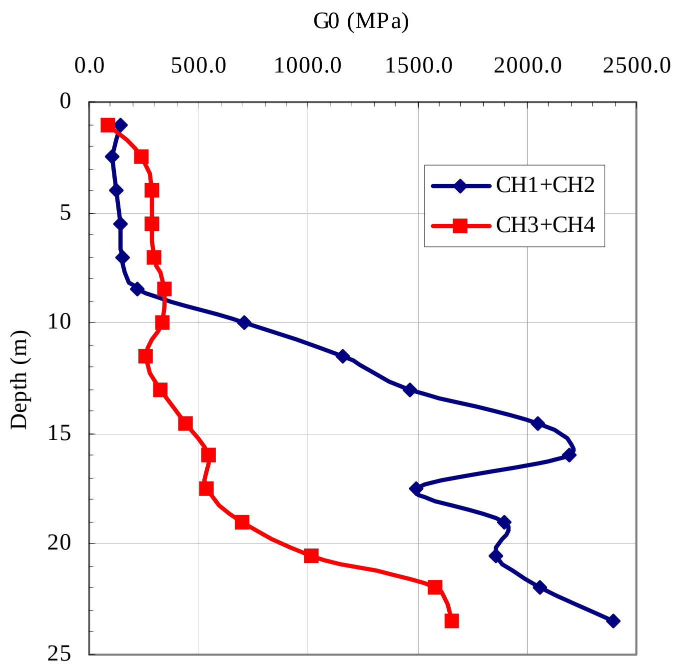

Fig. 10. Evolution of G_0 from the CH tests

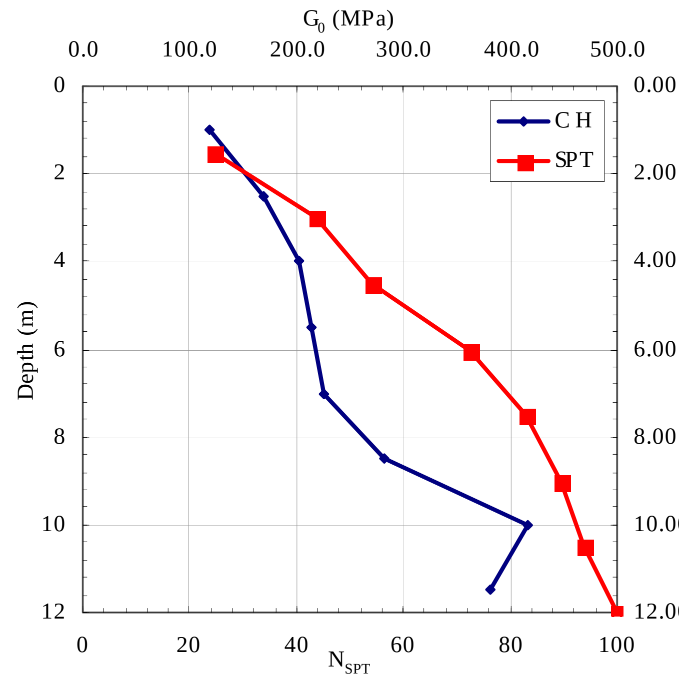

Fig. 11. Evolution with depth for G_0 values and N_{SPT} results in the residual soil horizons

Besides the above presented results, the tests provided several values of stiffness and strength parameters, that will be discussed in a devoted section.

The Lefranc tests resulted in an average permeability coefficient, k, of 6.9×10^{-6}m/s, with values ranging from 4.7×10^{-6}m/s to 1.2×10^{-5}m/s. These permeabilities are sufficiently high to admit an effective lowering of the water table could be achieved by the well points system already presented, without major concerns about the compressibility of the material.

Laboratory tests

Table 2 presents a summary of the laboratory tests performed during the design. It should be referred that the construction of the station was enclosed in a pack including the construction of 2 TBM tunnels (~7km), one NATM tunnel (~300m) and another 10 underground stations. In all the cases, granite residual soil, with several weathering degree, was the geotechnical material predominantly interested by the excavation. Hence interpretation of the geotechnical data was performed all together.

Table 2. Summary of the laboratory tests performed for the design

Description	Nº. tests
Granulometric curves	12
Direct shear tests	4
Triaxial tests	6

Later on, a large number of triaxial tests was performed for research purposes. These tests carefully conduced with the state of art techniques, included the evaluation of different suction levels on the soil parameters as well as different stress paths. As these results were not used for the design, they will not be presented although the existent preliminary results will be taken into account to draw the most relevant conclusions.

ASSUMED GEOTECHNICAL PARAMETERS AND HORIZONS

In the initial design quite simple models were assumed, both for the geotechnical zones and for the numerical modelling. Nevertheless, the very satisfactory behaviour of the excavation, without clear deviation from the initial design approaches, validates the assumed soil parameters (or at least confirms that they were suitably conservative). Horizontal layers of residual soil with different weathering degrees were assumed. Table 3 shows the depth of each of these layers, their weathering degree and the most relevant properties considered in the design.

Table 3. Geotechnical stratigraphy and most relevant parameters assumed for design purposes

Depth (m)	Weath. degree	c' (kPa)	ϕ' (º)	E (MPa)	ν	K_0
0 – 2		0	28	40	0.3	0.5
2 – 5	W6	10	32	45	0.3	0.7
5 – 18	W5	40	35	150	0.3	0.7
>18	W4	100	40	500	0.3	0.7

As Table 3 shows, the assumed model was very simple, with horizontal limits for the geotechnical horizons. This approach does not take into consideration the continuous evolution with depth of the soil parameters, as Figures 10 and 11 show, but proved to be a reasonable conservative approach to the design.

The tentative to back analysis the structural behaviour evidenced the need to consider a continuous evolution of the soil parameters with depth and also the strong influence of the suction level on the soil behaviour. This parameter does not only influence strength but also stiffness of the soil. This last aspect should be very relevant for the forces in the structure as to obtain the same deformations of the ground considering a stiffer material, much higher forces in the structure would be obtained.

Another important aspect to discuss is the value of K_0. The limited experience in the determination of this parameter for granite residual soils provided values around 0.5. In the site, after the design, SBP tests were performed giving values of K_0 around 0.6 for soil like materials and around 1.0 for rock like materials (Topa Gomes *et al.*, 2008).

NUMERICAL MODELLING

The design

As previously referred, the design assumed quite simple models, validating the shape of the excavation and its overall behavior. As references and finalized examples of such excavations are not abundant much of the inspiration was looked for in the tunnels literature.

In a first step, in order to have an idea of the total load acting on the shell, axysimetric models were run. This simulation led to a load configuration presented in Figure 12 and used for a first approach validation of the geometry and the thicknesses of the structural elements.

The diagram consists on a stress release on respect to the at rest horizontal stresses of 20%. Additionally, in the vicinity of the bottom of the excavation the stress is considered equal to zero, increasing to the above presented value at a distance of 0.3×H, measured from the bottom of excavation.

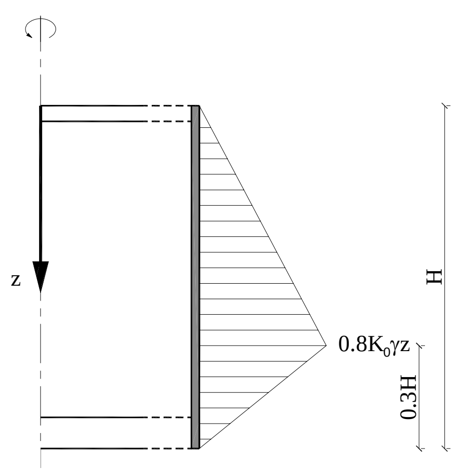

Fig. 12. Diagram for soil load considered in the design

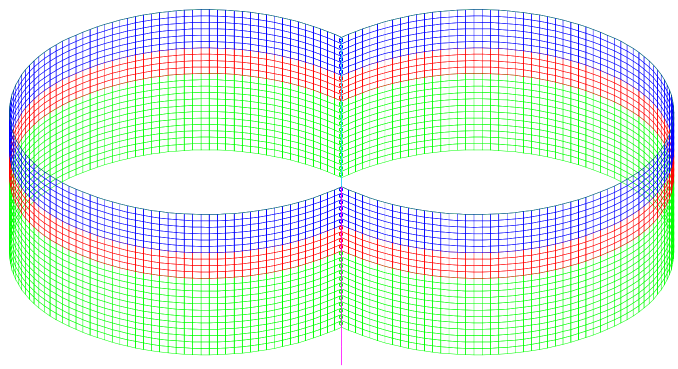

Fig. 13. Finite element mesh used in the design

Due to the elliptical shape of the structure, there were points diverting from the excavation. For these zones Winkler coefficients were assumed using the equation (1) proposed by Evison (1988).

$$k = \frac{1}{1+\nu} \cdot \frac{E}{R} \tag{1}$$

In this equation k represents the stiffness of the *Winkler coefficient*, ν the Poisson's coefficient of the soil, E its Young's modulus and R the radius of the excavation.

The numerical code STRAP, based on the Finite Element Method, was used to evaluate both displacements and stresses on the shell. Figure 13 presents a 3D view of the adopted mesh.

Additional calculations

In order to achieve a better understanding of the structure, additional calculations using a FEM numerical code developed jointly at the University of Coimbra and University of Porto were performed (França *et al.*, 2005). A 3D model with 20 nodes elements was used. Sixty three steps were adopted in order to simulate all the excavation phases. Figure 14 presents the finite element mesh adopted.

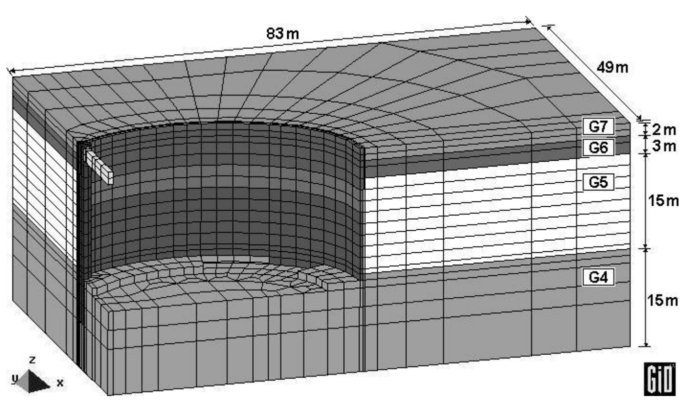

Fig. 14. Finite element mesh adopted for the additional calculations

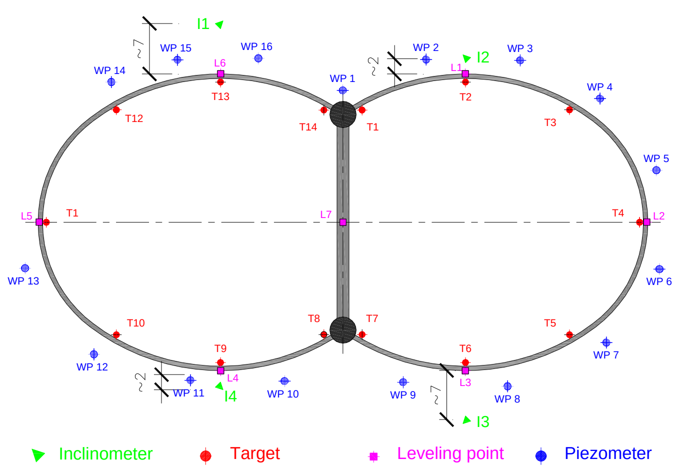

Fig. 15. Monitoring plan for the excavation

All the soil elements were assumed to have a Mohr-Coulomb failure criteria with the parameters presented on Table 3. These parameters were equivalent to the ones assumed in the design, being the geotechnical horizon G7 the one corresponding to the most superficial layer and for the other geotechnical horizons there's a correspondence between them and the weathering degrees of the granite residual soil.

Structural elements like the capping beam, the transversal beam and the shafts were assumed to behave linear elastic with a Young modulus of 15GPa and a Poisson's coefficient of 0.25. For the shotcrete the Young's modulus was assumed to be 10GPa.

ANALYSIS OF THE BEHAVIOR OF THE EXCAVATION

Monitoring plan

Figure 15 presents a plan view of the site with the most relevant instruments, allowing the understanding of the overall behavior of the excavation.

Besides the targets shown in Figure 15, targets in the correspondent plant position were installed at depths of 2.5m, 7.25m, 12.0m and 17.0m, allowing the control of the support deformation.

Additionally, targets, leveling points, crackmeters and clinometers were installed in the surrounding buildings. Nevertheless, as Figure 1 shows, the closest buildings were more than 15m away from the excavation and no significant results were recorded.

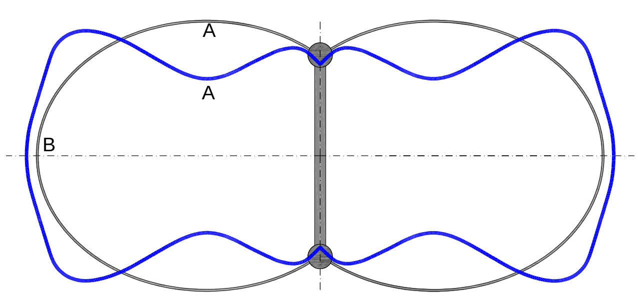

Fig. 16. Plan view of the deformed shape of the capping beam

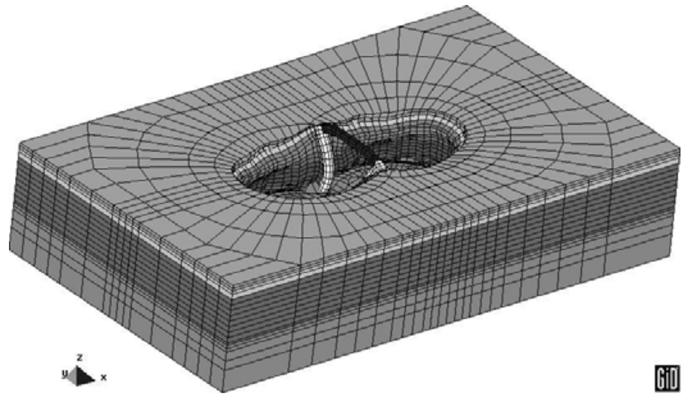

Fig. 17. Deformed mesh from the numerical calculations performed

Though the water table was lowered previously to the excavation and, hence, no important changes were expected in this parameter, the position of the water table was systematically controlled through the well points.

Overall behavior of the excavation

Figure 16 presents the deformed mesh of the capping beam, resulting from the design calculations and allowing a perception of the deformation of the shell, in plant. Figure 17 presents the deformed mesh obtained in the numerical modeling. These figures evidence that the frame constituted by the circular shafts and the transversal beam plays a key role on the overall stability of the excavation. Actually the frame gives represent the point giving stiffness to the structure, avoiding larger deformations.

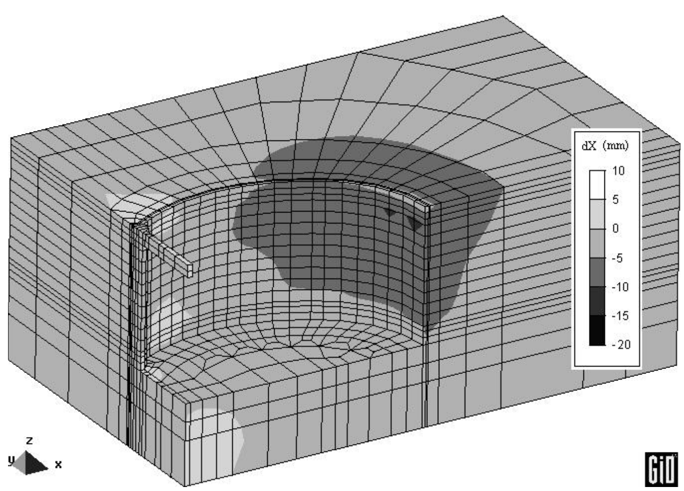

Fig. 18. Isochrones of longitudinal displacements

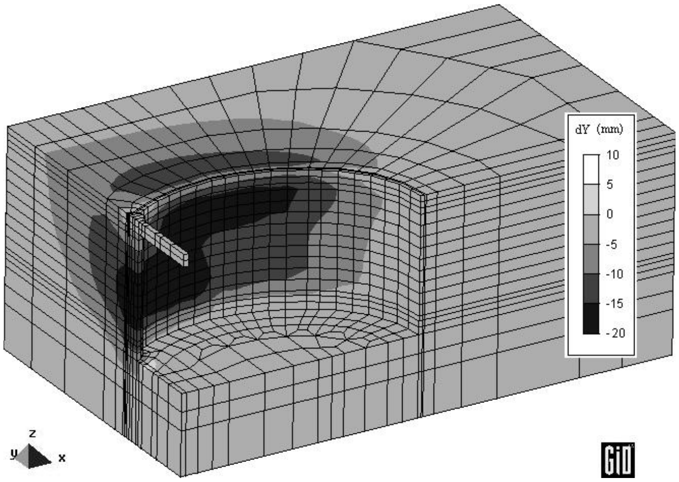

Fig. 19. – Isochrones of transversal displacements

Figures 18 and 19 represent the isochrones of displacements, in the longitudinal direction and transversal direction, respectively.

The zones with bigger deformations are the zones close to point A (Figure 16), where the ellipse has its bigger radius and, thus, bending forces tend to prevail over axial forces. Besides, the convergence of the excavation in this zone increases with depth, but reaching its maximum around 0.7H, being H the height of the excavation.

Another relevant aspect is related with the longitudinal behavior, occurring a divergence on the tops of the excavation.

In the following points it will be discussed with more detail some particular aspects of the behavior of this structure.

<u>Surface settlements</u>

Figure 20 presents the isochrones of the surface settlements, derived from the 3D numerical model developed for the calculations. Figure 21 presents the settlements registered in the leveling points (L1 to L6 – Figure 15) installed in the capping beam. As there weren't significant differences between the results on the leveling points, it was decided do

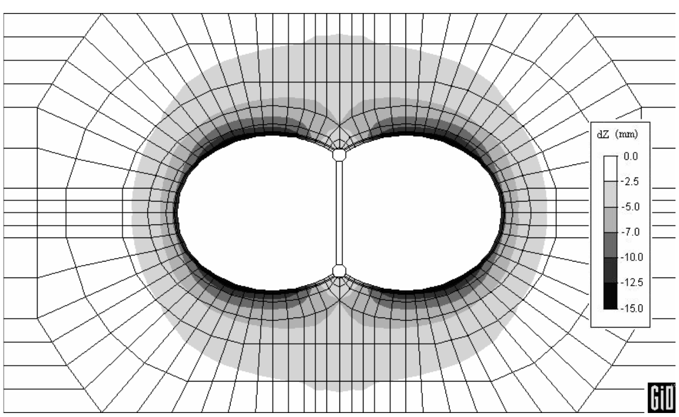

Fig. 20. Isochrones of surface settlements

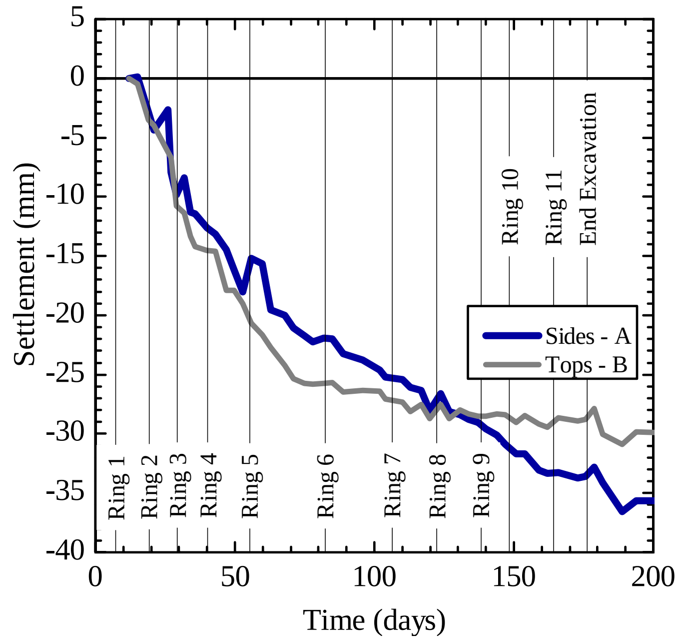

Fig. 21. Settlements measured in the leveling points (L1 to L6) in the capping beam

present the average of these instruments on the side (equivalent to point A) and on the tops (equivalent to point B).

The Figures show that, even between the top of the excavation, much stiffer, and the sides there weren't relevant differences on the measured settlements. The magnitude of these settlements was around 30mm, what was not motive for concern as the buildings were not close. Figure 20, although referring to numerical predictions, evidence this uniformity of settlements around the excavation. In any case, it must be emphasized that the zone with higher horizontal movements (A) tends to have a larger subsidence basin.

In what refers to the settlements at a certain distance from the support, the building closest to the excavation, 15m away in the northern part of the station, registered a maximum settlement of less than 15mm. It should be referred that part of this settlement, around 3mm, was due to the water table lowering.

<u>Transversal beam</u>

Figure 22 presents the evolution of the vertical movement in the middle of the transversal vertical beam. The monitoring results evidence a continuous heaving of this beam, in its central part, due to the bending of the vertical shafts. This heaving occurs as the excavation proceeds.

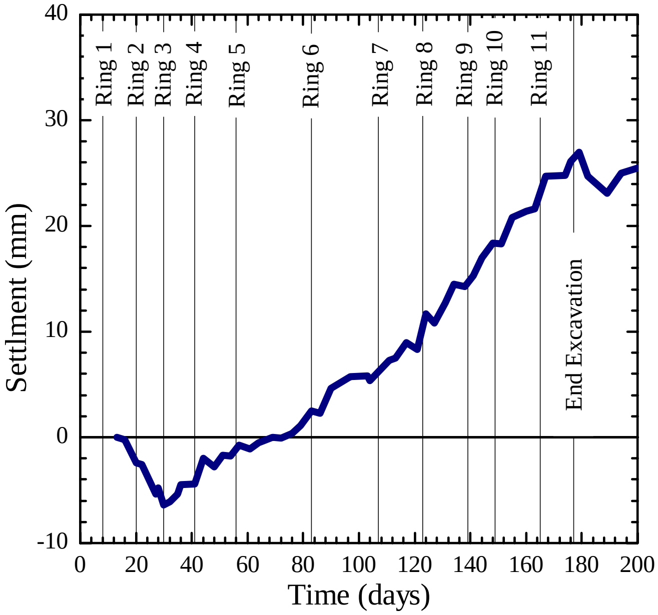

Fig. 22. Settlements measured in the middle of the transversal beam.

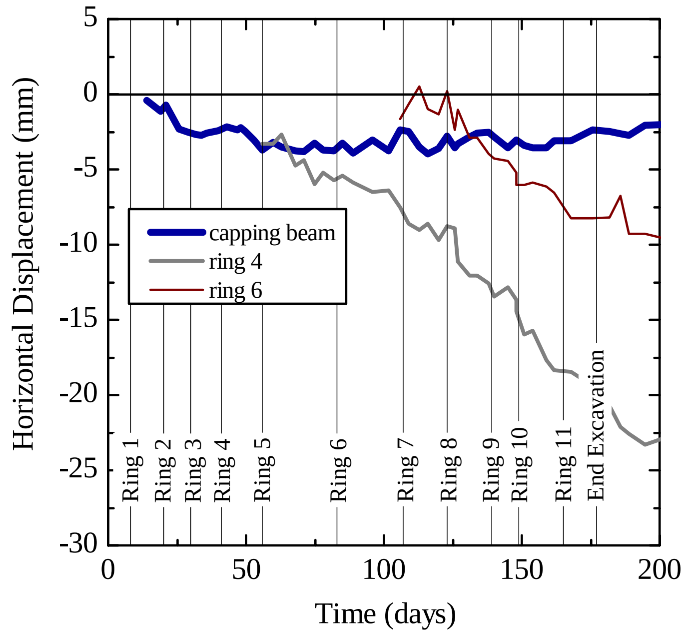

Fig. 23. Convergence on the central part of the excavation

The initial part, where a settlement can be observed, is due to the self weight of the beam. The magnitude of the final heaving evidence that the forces in this structural element were bigger than the ones initially foreseen in the design, as it clearly exceeds the design predictions. The assumed Young's modulus, 15 GPa, was quite reduced with the concern to adequately predict the long time deformation of the structure, taking into consideration phenomena like creep and concrete shrinkage. Nevertheless this potential, this procedure conduces to reduced forces in the structural elements and, hence, a non conservative approach for the ultimate strength design.

Another aspect that conduced to the above mentioned differences is certainly related with the simplified geotechnical horizons considered in the design and the evolution with depth of stiffness properties of the ground. Figure 11 evidence a continuous evolution with depth of this property whilst Table 3 shows the design assumed horizons with constant thickness and constant stiffness. The natural conservative approach of the design tends to underestimate the Young modulus and, thus, for equivalent deformations, the installed stresses in the structural elements tend to be lower.

In any case it should be referred that the non conservative prediction of forces in this beam was not, in any instant, a safety issue. Actually, if the resistance of the beam was exceed, cracking would occur at the connection to the vertical shafts, limiting the maximum bending moments and forcing the beam to work as a simple strut, case where the capacity of the beam was well beyond the reached forces.

The deformation of the support

In order to evaluate the deformation of the support the length of several segments derived from the targets installed during excavation was used. A first aspect that should be emphasized is that these targets, as they were installed after shotcreting the lining, only measure deformations after excavation. An important part of the soil deformation may not be detected with these instruments.

Figure 23 presents the average movement of the support (convergence divided by 2), transversally to the longitudinal station axis, measured on the vertical shafts at the levels of the capping beam, Ring 4 and Ring 6. The result obtained at the capping beam level confirms the high stiffness of the transversal beam as, at its level, the total horizontal displacements are reduced (around 3mm).

At the level of Ring 4, around 9m deep (40% of the excavation height), the displacement reaches after lining construction almost 25mm, proving that the vertical shafts are very flexible for the soil pressures they have to support.

In the location of the works there was no concern about settlements in the vicinity. If that wasn't the case, an effective control of the support deformation could easily be achieved by enlarging the vertical shafts or by introducing an additional transversal beam at mid height of the excavation. This solution would not introduce a significant construction complexity or delay on the excavation process, neither a huge difference from the point of view of costs.

At the level of Ring 6 (13m – 60% of excavation height) the final movements are much smaller than at Ring 4. This occurs

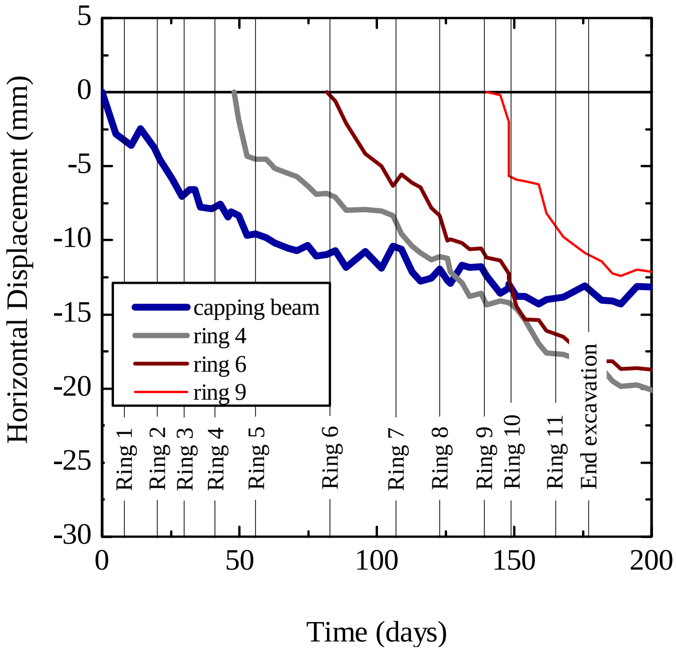

Fig. 24. Horizontal displacements on the central part of the excavation (point A)

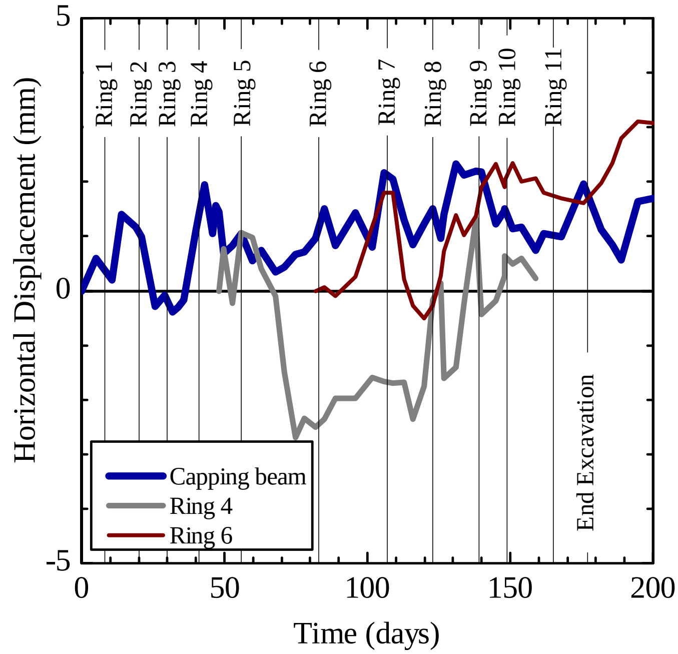

Fig. 25. Horizontal displacement in the longitudinal direction (point B)

because the distance to the excavation front is smaller and also so due to the significant increase on the stiffness of the ground with depth.

A relevant aspect is related with the rate of increment of these movements, smaller in Ring 6 than in Ring 4. For this aspect also contributes considerably the favorable evolution of the soil stiffness with depth.

Figure 24 presents the transversal movement of the support (convergence divided by 2) at the section of maximum with of the excavation (point A), while Figure 25 presents the same variable in the longitudinal direction (point B).

On Figure 24 it can be seen that, at the level of the capping beam, occurs, in Point A, important horizontal movements. This is natural as, at the points of maximum width of the excavation, there isn't the transversal beam and only the stiffness of the capping beam avoids displacements. Hence the final magnitude of the movements is of the same order of the registered maximum displacement, although slightly minor (14mm vs 23mm).

The registered displacements for Ring 4 and Ring 6 are very similar even being this ring much closer to the excavation front (bottom of excavation). This reveals the reduced stiffness of the support. If a deeper excavation was needed or settlements in the periphery of the excavation were a concern, a stiffer support had to be implemented.

This last aspect is also supported by the fact that the rate of increase of the horizontal displacements is bigger at Ring 6 than at Ring 4. The theoretical bigger loads may explain this aspect but, on the other hand, the favorable evolution on the quality of the granite mass with depth and the proximity of the bottom of the excavation would suggest the opposite behavior. Even at Ring 9 this rate of variation is observed.

In what refers to Figure 25, it is clear the divergence of the extreme points of the station during the excavation. In any case, the displacement is of reduced magnitude (around 3mm) and hence not much different from the precision of the targets. If the abnormal convergence during the excavation occurred between Rings 5 and 6 is neglected, the divergence is minimum at the level of the capping beam, maximum at the level of Ring 4 (3.3mm) and decreases slightly at the level of Ring 6 (around 3.0mm). This behavior is in accordance with the convergences observed in the transversal direction, although these conclusions should be regarded with a certain prudence as the measured values are in the same order of the targets precision.

<u>The ground deformation</u>

As Figure 15 shows, there were 4 inclinometers, installed previously to the excavation, where the real deformation of the soil could be monitored. These instruments were roughly installed in the alignment of the sections with maximum width, what should correspond to the maximum horizontal deformation of the support and, thus, the maximum horizontal deformation registered on the ground. Two of the inclinometers were installed very close to the support (2m distance), I2 and I4, and the other two inclinometers were installed a little bit further, around 7m away from the support.

Figure 26 presents the displacement normally to the excavation of inclinometer 2 while Figure 27 corresponds to

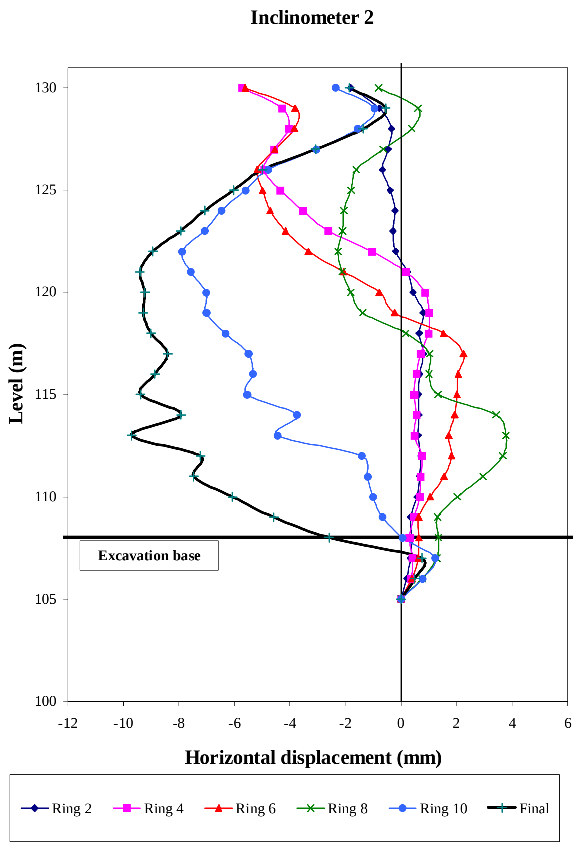

Fig. 26. Horizontal displacements normal to the excavation - Inclinometer 2

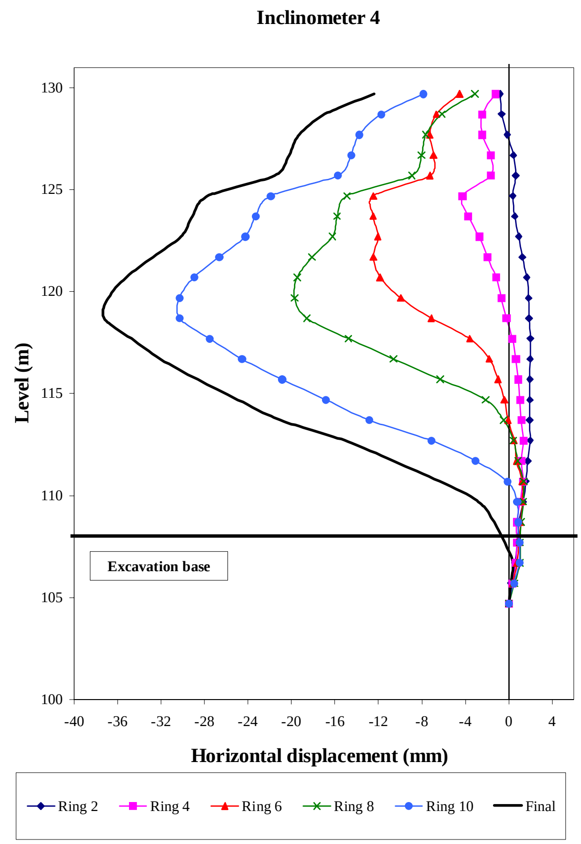

Fig. 27. Horizontal displacements normal to the excavation - Inclinometer 4

inclinometer 4, the 2 inclinometers closest to the support. The first aspect that should be emphasized is the notorious difference in the measured values in the two inclinometers. This is above all a clear sign of the heterogeneity of the ground mass. The differences were also present on the measurements of the targets although with less influence as the presented results were calculated on the average of both sides of the excavation. In any case it can be said that the final shape is similar besides he magnitude. In such situation, the main conclusions will be derived from the results of Inclinometer 4 as, having bigger amplitude of movements, the results are less influenced by the natural oscillation of instrument precision (roughly 6mm per 25m).

The first aspect to mention is the depth of the maximum horizontal displacement, 12m, slightly below the mid height of the excavation and corresponding to the excavation of Ring 6. At the same depth, the displacement before the excavation was roughly 12mm, reaching at the final stage of the excavation approximately 37mm. This means that the part of the displacement occurred before the "passage of front" is roughly 30% of the total displacement and hence the measurements achieved by the targets in the support represent the greatest part of the soil deformation.

Another relevant aspect to refer is related to the horizontal displacement of the capping beam, around 12mm. This movement, although captured by the numerical calculations underestimated. This aspect may be a consequence of the stiffness considered for the superficial soil horizons (E=40MPa). This value is of the same order of the Young modulus considered for W6 horizons (E=45MPa) and W5 horizons (E=150MPA) and Figure 11 clearly evidences the increase of the soil stiffness with depth.

Finally it must be referred that at the bottom of the excavation almost no displacements were registered. If the ground mass was uniform this would be unexpected, but in the present case is a consequence of the improvement on the soil quality with depth. At this level clearly dominates the presence of a rock like material (W4 granite) with values of G_0 well above 1000MPa.

In order to evaluate the evolution of the horizontal displacements with the distance to the support, Figure 28 presents the measurements on inclinometer 3, the one installed at a distance of 7m from the support with bigger displacements. As occurred with inclinometers 2 and 4, the heterogeneity of the ground mass is notorious, being the West side stiffer (inclinometer 3 vs. inclinometer 1). As the displacements on Inclinometer 1 were very reduced, they will not be presented.

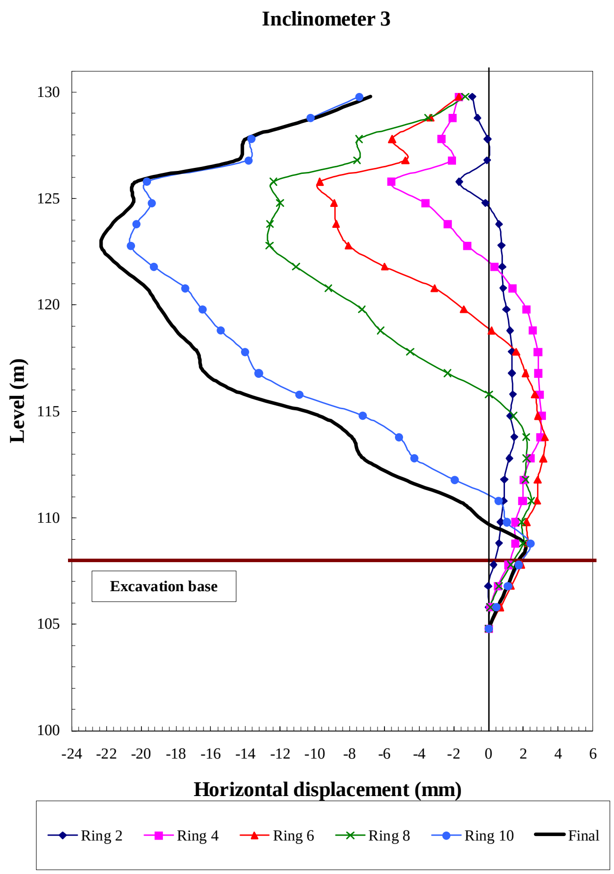

Fig. 28. Horizontal displacements normal to the excavation - Inclinometer 3

It is also visible a horizontal displacement at the level of the capping beam and, for inclinometer 3, the maximum horizontal displacement is reached at lower depths (~8m). Apart from this aspect related with the maximum horizontal displacement, the curves have a similar shape to the ones referring to inclinometers 2 and 4.

CONCLUSION

The design and behavior of a novel solution for a Metro Station excavation was presented and discussed. The solution was implemented with considerable success in what refers to geotechnical behavior, cost control and construction planning.

The design assumed simple geotechnical models but was able to predict satisfactorily the behavior of the excavation. Nevertheless, the detailed analysis of the monitoring results evidenced certain differences between what was expected in the design and the real behavior of the works, differences that were basically concentrated in the magnitude of the movements and in the forces in the transversal beam, a vital element in the overall stability of the structure. A 3D numerical modeling of the excavation, taking into consideration the non saturated behavior of the soil would certainly clarify certain aspects of the behavior.

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