MODELLING THE BEHAVIOUR OF A RETAINING WALL MONITORED DURING THE EXCAVATION FOR A DEEP STATION IN METRO DO PORTO

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Résumé : *Le document présente la modélisation pour la rétro-analyse du comportement de la structure de soutènement observé dans la construction de la station Aliados de Metro do Porto. Ce construction avec des proportions inhabituelles (en plan et en profondeur), a été construit dans les formations granitiques typique de Porto, et la excavation a été effectuée avec des pieux en béton moulés périphériques, supportés para un système multi-ancré. Ce rideau a été instrumenté de manière exhaustive, au cours de sa construction, en particulière dans un section et correspondant profil de sol, dans le but de la vérification des paramètres de conception adoptée, et l'avenir de la définition optimisée des lois constitutives nécessaires pour la modélisation avec um code numérique de éléments finis. À cette fin, une extensive caractérisation en laboratoire a été effectuée sur des échantillons de haute qualité, dans les systèmes triaxiaux avec de l'instrumentation précis. Ces échantillons ont été recueillies au cours de la fouille, ce qui permet la définition des paramètres pour les modèles considérés sur le logiciel commercial Plaxis®: Mohr Coulomb et Hardening Soil. Ces exercice de rétro-analyse des forces et des déplacements registrés au cours de la construction, dans la section instrumentée, a pris en compte les solutions structurelles qui ont été adoptées dans le cadre du projet, mais, compte tenu de certaines modifications à la séquence constructive lors des fouilles et l'activation du système de soutien (de légères modifications pendant le travail qi ont des influences dans les résultats de l'observation). La l'influence de l'écoulement de l'eau du sol au cours de la fouille a également été étudié, prouvant qui est un facteur clé pour l'ajustement des tassements de la surface.*

***Mots-Clefs** : Sols Résiduels, Multi-Anchored murs de soutènement, Modèles, Observation*

***Abstract** – The paper presents the modelling for retro-fitting the observed behaviour in the Aliados' station of Metro do Porto's peripheral retaining wall. This work, with unusual proportions (in plant and in depth), was built in the typical Oporto's granite formations, and the excavation was carried out with peripheral retaining walls, using mainly the multi-anchored concrete wall made of cast in situ bored piles. This curtain was thoroughly instrumented and monitored during its construction, with some detail, in a particular ground profile and respective structural section, with the purpose of verification of the adopted design parameters, and future definition of optimized constitutive laws for modelling in a FEM numerical code. For that purpose, subsequent laboratorial characterization tests were performed over high quality samples, in triaxial systems with precise instrumentation. These high quality soil samples were collected in the course of the excavation, allowing the definition of input parameters for Mohr-Coulomb and "Hardening-Soil" models, for a multi-phased the numerical simulation based on the commercial software Plaxis®. These exercise of retro-analysis of the displacements and forces monitored during the construction, in the specific well instrumented section, took into account the structural solutions that were adopted in the project, but considering some specific changes of the constructive sequence actually implemented during the excavations and activation of supporting system (which has suffered small changes during the work and had influences in the results of observation). The specific, but determinant, influence of the ground water flow developed during the excavation, as a key factor for the matching of the surface settlements, was also studied.*

***Key words –** Residual soils, Multi-anchored retaining walls, Constitutive models, Monitoring*

1. Introduction

The Metro station of Aliados is part of the yellow line (line D) of Metro do Porto and is one of its most important stations. This structure is buried in the avenue with the same name and was built by the "cut and cover" method, with an excavation of about 24 m deep. The project of the retaining structure considered diverse solutions, in accordance to the different ground classes involved, being the multi-anchored (with 6 levels) bored pile wall the most used solution. All the work was properly instrumented, with emphasis to the Northwest area, due to the presence of highly weathered materials, namely residual soils (Viana da Fonseca & Quintela, 2009).

A specific geotechnical and geological characterization of the all area where the station was placed, based on *in situ* tests, such as SPT, was presented in another paper to this symposium (Viana da Fonseca & Quintela, 2009), making reference to the experience of this type of soil characterization that can be found in previous works. There, the definition of physical characterization and mechanical properties was discussed, in the light of thorough laboratory tests executed over high quality samples, including oedometer and high-precision triaxial tests. Representative model parameters for Mohr-Coulomb and "Hardening-Soil" laws based on those tests, for the constitutive modeling of the observed structure, are now used, in a multi-phased FEM numerical simulation using the commercial software Plaxis®(Brinkgreve et al., 2004), with the purpose of retro-fitting the displacements and forces monitored during the construction. A similar approach has been implemented in another work of retro-fitting the behavior of an excavation and retaining structure in residual soil from granite of a underground stations of the Metro of Porto, and presented with some detailed description of the modeling premises for the best fitting of the available monitoring, using the same FEM code, by Rios Silva et al. (2008).

2. Modeling and results back-analysis

2.1. Modeling of excavation

The modeling of the excavation was made according to the software recommendations to excavations supported by multianchored structures, for the elements used and their parametric characterization. In first place, the geometry and the model to be used, with their respective boundary conditions, were defined. The dimensions of the excavation, situated near the studied area of Aliados' station are about 22 m wide and 24 m in height. The double symmetry of the station along the North-South and East-West axes was used to model only half of excavation.

2.2. Description of the geometry, mesh, types of elements and materials

The horizontal limits of the geometry were 55 m, exceeding the estimates obtained by Clough and O'Rourke (1990), that the settlements would be null, in sands, at a distance from excavation limits about 2 times its depth (24 x 2 = 48 m). The distance used is slightly more than two and a half times. In relation to the vertical dimensions, it was admitted that rock would be practically located 23 m below the base of the excavation, a value near the height of excavation. Thus, the size of adopted mesh is, horizontally, 66 m (11+55) and, vertically, 47 m (3+21+23). It is important to evoke that the first 3 meters considered in vertical dimensions resulted from the fact that the top-caps of the piles are situated 3 m below the surface in the periphery formed by the boundaries of work and the road surface of the Aliados' avenue. The boundary conditions consist in double support in the model base and simple support in the lateral side, without any restriction of the vertical component of displacement in these borders.

The items to be included in the model were those who are discriminated in the following. The wall was constituted by bored piles of 1m in diameter spaced 1,36 m between them, taking in account the design option of Viaponte (Normetro, 2002a and b) that is reproduced, being their behaviour modeled with "plate" elements. Although this type of elements is usually chosen for modeling continuous structures (i.e. reinforced concrete diaphragm walls), this is adequate to be used in this case, as far as bending is considered. This will be defined by the parameters of

bending and axial rigidity (EI [kN.m^2/m] and EA [kN/m], respectively), the weight [kN/m/m] and the Poisson ratio of the material of the panel. The material used in the retaining structure, considered elastic, was reinforced concrete C30/37 with a elasticity module equal to 32 GPa (Eurocode 2, 2004), weight volume of 25 kN/m^3 and a Poisson ratio of 0.2. The axial stiffness of this type of contention is the product of elasticity module of concrete and the area per meter, resulting in: EA = 1.85×10^7 kN /m. The bending rigidity is given by the product of modulus elasticity module of concrete and the mobilized inertia per meter, EI = 1.15×10^6 kN·m^2/m. The weight (W) follows from the account of only the self weight of the piles, per meter of development and height of the piles (W=14.4kN/m/m). The anchors were defined by elements "node to node anchor", as suggested by the software's manual. The axial stiffness of each anchor was 1.9x10^5 kN, and the longitudinal spacing equal to 2.8 m. The sealing bulbs were modeled, as suggested by Raposo (2007), introducing geogrid elements, resisting only to tension, linked to the "node to node" elements. This last element is characterized by its axial tension rigidity (EA [kN/m]), equal to 10^5 kN/m. It was also applied an interface between the curtain and the soil using joint elements without thickness. Besides those definitions, an overload on surface of 2 kN/m^2 was considered, in order to simulate the avenue surface traffic adjacent to the station (under service during the work) and a slope of transition between the piles's heads and the landfill. The mesh was defined with a medium discretisation, increasing along the sealing bulbs, where stress concentration was expected. It consists of 857 elements (general elements with 15 nodes each), 7175 nods and 10824 Gaussian points (Figure 1).

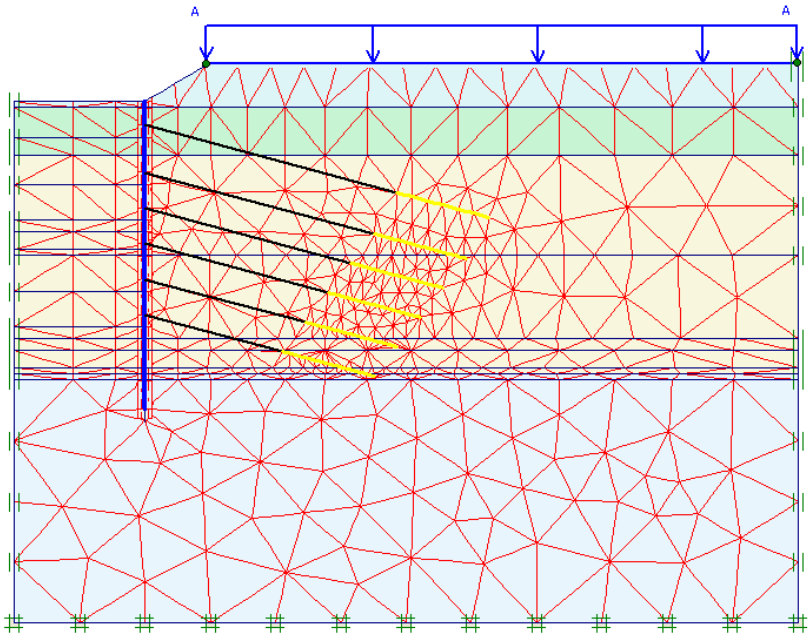


Figure 1 – Finite elements base mesh for the numerical simulation of the excavation

The Mohr-Coulomb and Hardening Soil models were implemented in Plaxis® software taking into account local regional experience for these soils (Viana da Fonseca, 2003, Viana da Fonseca & Almeida e Sousa, 2002, 2003, Viana da Fonseca & Quintela, 2009).

2.3. Constructive stages considered in the numerical analysis

In each stage of the excavation, part of the soil is removed the front of the curtain and the hydraulic head on each of the known points (in the base of excavation and one meter from the curtain, as the values situated 20 and 55 m far way were always constant). It is necessary to redefine the water pressures when the position of water changes, and since there is a difference between each of the hydraulic heads, it is necessary to activate a subprogram that exists in Plaxis® code ("ground water flow calculations") that calculates the water pressures taking in account the seepage network established in the mass with the new geometry.

2.4. Obtained results and comparison with the monitoring

The obtained results through numerical modeling, considering the parameters derived from the analysis, presented in Viana da Fonseca & Quintela (2009), were treated according to the available elements and results of instrumentation installed, allowing to proceed to the back-analysis. This set would be based primarily on the analysis of the inclinometer (one vertical and one horizontal), topographic marks (situated in many points) and load cells.

2.4.1 Vertical and horizontal inclinometers

The studied vertical inclinometer, named as I4, was placed close to one of the piles that was sustaining the excavated soil. Measurements were made as far as it has progressed. In the Figure 2a it is the displacement corresponding to the last phase (maximum excavation) and obtained by the Plaxis calculations.

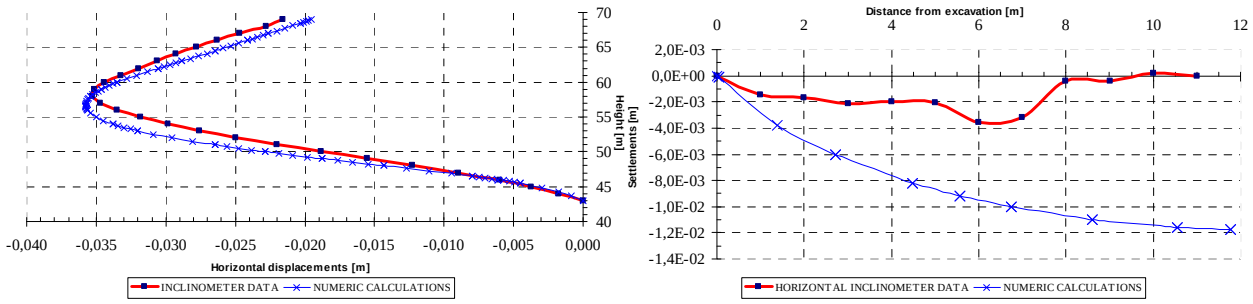


Figure 2 – Comparison between the displacements obtained in Plaxis® e observed in: a) vertical inclinometer; b) horizontal inclinometer

As shown in figure, the results of numerical modeling and the data of the inclinometer converge satisfactorily. However, the adjustment is hardly achieved in the areas of the top and bottom of the curtain. To improve this, taking in account the “idealization” of the geological and geotechnical model adopted for the studied area, slight modifications were tested to the thickness of various horizons (mainly with regard to dimensions that define the most relevant horizon, a W5-W4 granite, thicker in depth, where a more complete characterization was done, as described in Viana da Fonseca & Quintela, 2009).

Two horizontal inclinometers (IH1 and IH2) were installed in the same height of the first anchor level, and both were slightly distanced from the section under study (one located further North and the other further South). IH2 was chosen because he was situated in a similar geological and geotechnical profile and closer to the studied section. The inclinometer was end-fixed in the rock, with an embedment of 11 m below the base of the excavation. The settlements obtained through numerical simulation are substantially higher than those obtained by the inclinometer (Figure 2.b). The reasons for this difference will be discussed further below.

2.4.2 Topographic marks

For verification of the parametric options that aim to improve modeling of the structure’s behaviour, only three types of marks were studied: one situated in the first anchor level beam, four situated on the surface along the horizontal inclinometer (IH2) and another located in the surface (inside the field, in the building site limits).

The mark situated on the beam allowed the registration over time of displacement values in the three main directions. The determination of the displacements from the calculations with Plaxis® (only in the perpendicular plane to the station, by the fact the software version that was used is 2D) was done by the evolution of the displacement of the first anchor level’s head. The

comparison between the values recorded and those obtained in the software is shown in the graphs of Figure 3. Four marks were placed on the surface (M1, M2, M3 and M4) in the direction of IH2 inclinometer, distanced from the excavation by 5.5 m, 8.5 m, 19.5 m and 25 m, respectively. Through these, it was able to record vertical displacements of these points along the time. However, in these dates it was impossible to record data (mainly in marks M2, M3 and M4). A comparison of instrumentation and numerical simulation is in the graphs of Figure 4.

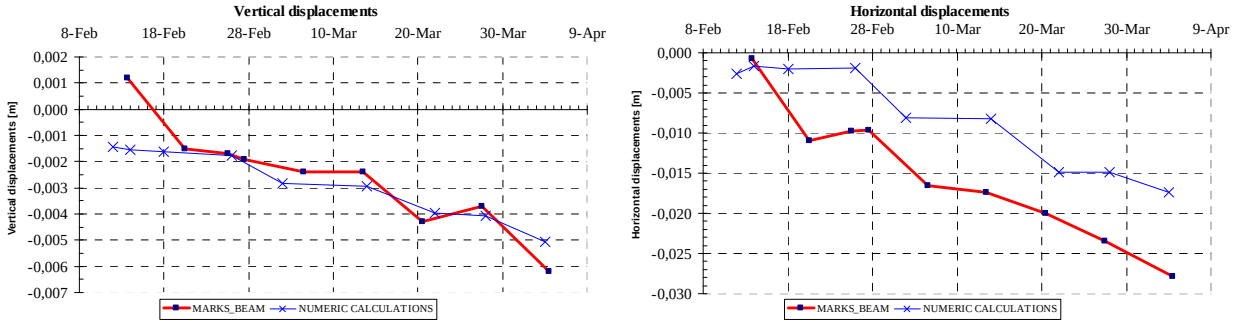


Figure 3 – Vertical displacements observed in the beam and obtained in Plaxis® calculations

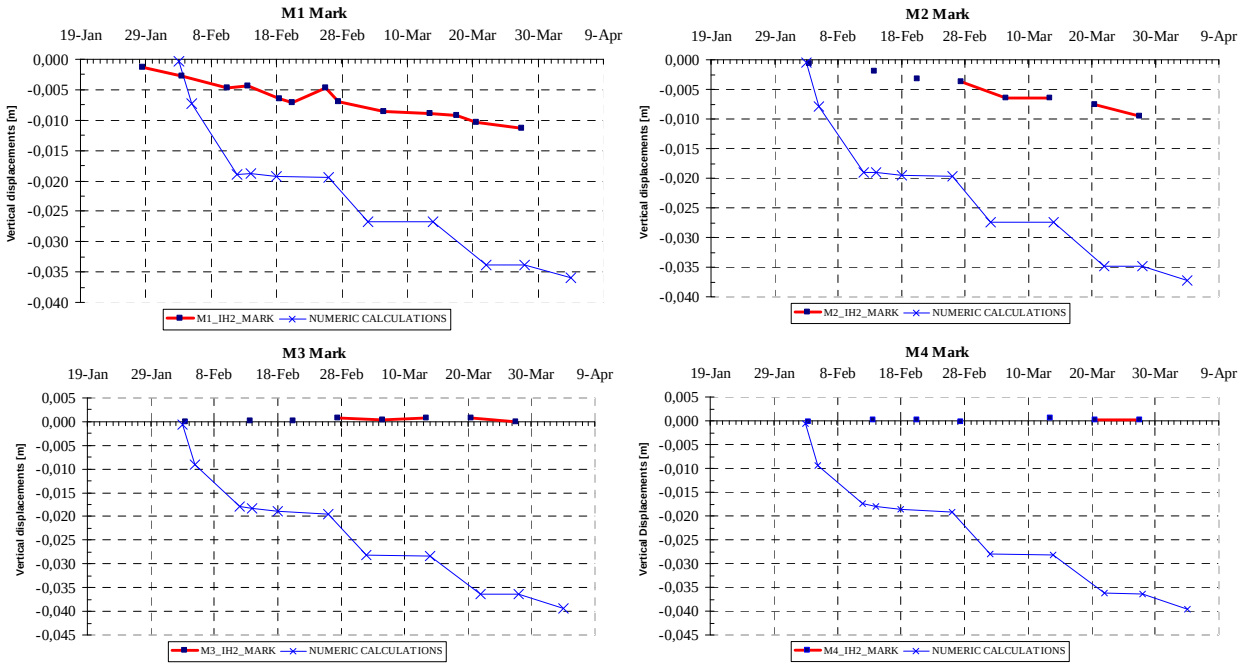


Figure 4 – Comparison between the settlements obtained from Plaxis® and through marks

A mark was placed around the building site, in a distance of 9 m in relation to its limit, to make an additional control for vertical displacements near the excavation along the time. The comparison is given by Figure 5. Although the results obtained in the mark located next to the inclinometer in the beam, are very similar to the numerical simulation, the adjustment for the other brands continue to show some discrepancies because, again, there is an excessive settlement of the entire mass obtained in the numerical simulation. Some considerations on the reason for this difference will be made in what follows.

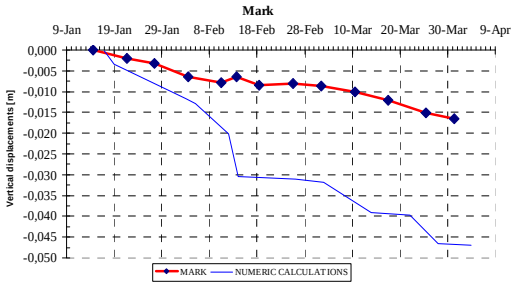


Figure 5 – Comparison between the settlements obtained from the Plaxis® calculations and from a mark

2.4.3 Load Cells

Load cells were placed in several anchors, according to the design. However, only three are relevant to the back analysis, situated on the first, third and fifth level. These cells were slightly away from the studied section for North and South, but sufficiently close to allow its representativeness, enabling to know the real value of pre-stress applied on each of them (3 levels – Figure 6) and the date of its application.

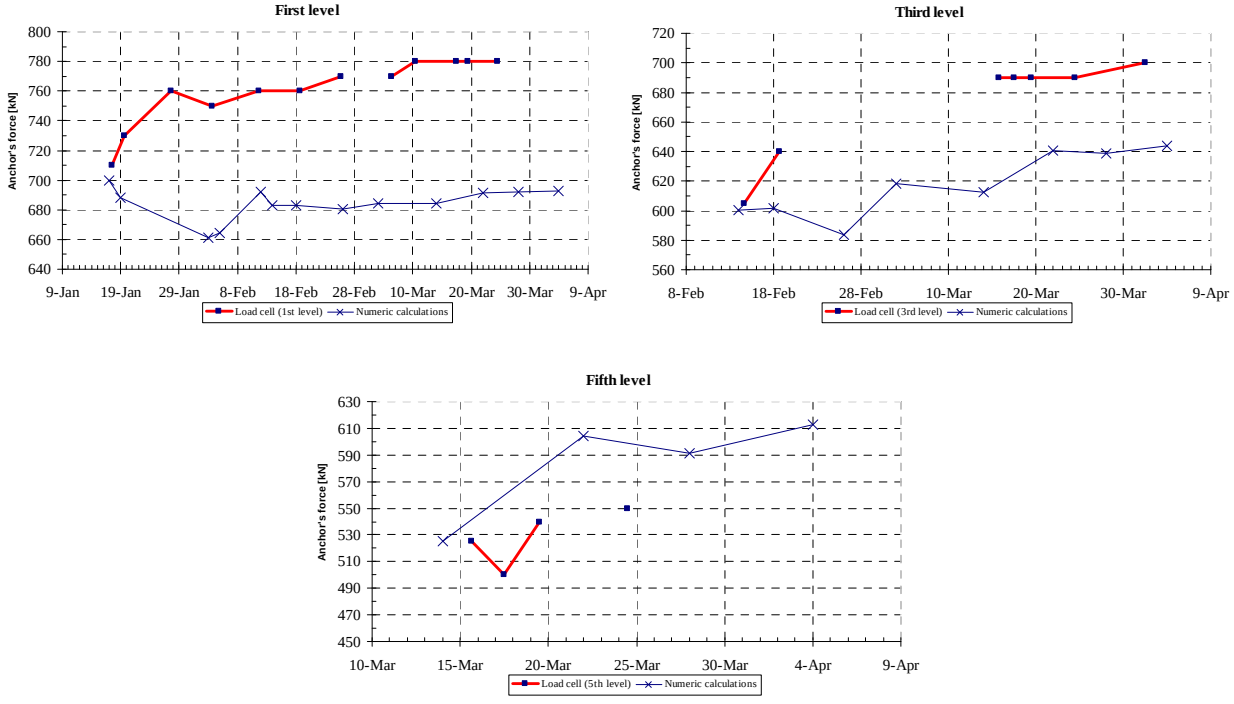


Figure 6 . Values obtained for the 1st (a), 3rd (b) and 5th (c) anchor levels

Along the time, there were registers that could be compared with the value inferred from the numerical situation in each stage, obtained by in each of the elements (“node to node anchor”). Although the numerical results have shown a different behaviour in the first level, is possible to observe a very close convergence in the remaining levels.

3. Comments

The results obtained through numerical simulation of excavation by Plaxis® were very satisfactory, mainly for horizontal displacements of the curtain. However, there were some difficulties that may explain some minor differences and discrepancies between the values obtained by the instrumentation and deferred by numerical modeling. The greatest difficulty was in knowing by certain the exact timing for starting and eding of each and all different phases, as

defined with Plaxis®, in correspondence to the defined in the design. The dates were very important and its record by the photos essential, because the dated instrumentation should match the excavation position, the construction of the supports and the anchors' activation. Another difficulty comes from the fact that only the analyzed vertical inclinometer and the piezometer were situated exactly at the precise vertical profile, for the studied section. The other elements were situated a little further North and South from the right place (eg. 3 anchor levels are not in the same alignment and the horizontal inclinometer is slightly aside to the South).

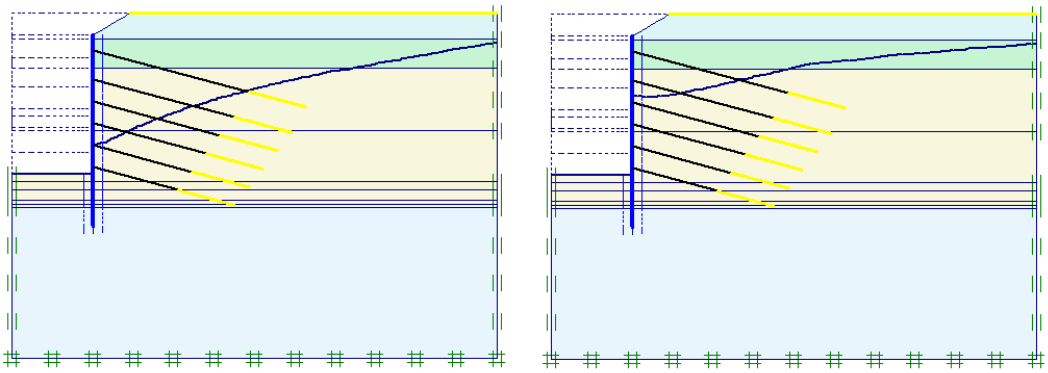
The Plaxis® software presents the results on the end of each stage, but there is no information of the process during each phase. An excavation of 3 m is done sequentially, by parts, for example, of 1 m. Many of the results of instrumentation were taken on dates non coincident with the final phase of Plaxis®, which may explain some of the differences. These reading dates corresponded, often, to a situation in which the work was between two phases in Plaxis® (for example, a given stage was already completed, but the following had not yet been completed). As mentioned, the dates assigned to each of the stages in the numeric calculations started from an estimation based on site photos during the construction and from very few existing records (eg. date on which anchor was pre-stressed). Other difficulties are due to the lack of information on other levels of anchoring and an estimation of the other pre-stressed values in the intermediate levels was assumed. The depth of each horizon, mainly the W5-W4 granite - the one that has greater thickness, was slightly adjusted during converging process.

Analyzing the results, it appears that, for the horizontal displacements observed in the curtain, the fit is clearly good, and the differences of approximately 2 to 3 mm (but never over the 5mm). The comparison at each reading stage follows a general pattern of convergence with the final recording. The vertical displacements don't show as good results (both the values obtained through the horizontal inclinometer as those obtained by the topographic marks). This may be caused by many factors (eg the fact that the inclinometer is a little further south than the studied section) means that it is located in an area where the soil shows more rigidity, both longitudinally and transversely, which may explain in part the slightly difference observed. However, other reasons should be given to explain such difference based on the records of the horizontal inclinometer (IH2): 11 m away from the excavation, it appears that there is a characteristic concavity, typical in this type of excavations. In Plaxis® the concavity only begins in greater distance (although slightly) which implies that, at 11 m, it is not as clear at should be. An increase in mesh size in the horizontal direction is not, by itself, a solution to try to resolve such large settlements in that area, since they don't show a clear trend, if there is an increase in the horizontal dimensions of the mesh, assuring near zero settlements in the limit.

Several hypotheses can then be outlined, but firstly a study was made on the balance between the volumes of earth displaced vertically from the surface and the volume displaced horizontally in the curtain, in the last stage of excavation. In order to have compensation in displacements, the sum of the horizontal movements of the curtain (0.74 m^3/m) would be close to the vertical uplift movements in base surface (2.35 m^3/m), while the volume really displaced in the excavation (0.13 m^3/m) was far from that. This would imply that the soil developed a high volumetric compressibility at the end of excavation, being the more thoroughly tested soil (W4-W5 granite) contributing more to this, since it has the greater thickness. It really confirms the high compressibility obtained in the tests performed in the lab (Viana da Fonseca & Quintela, 2009).

On the balance of displaced volume, it appears that the soil presents a high compressibility which is reflected by the high vertical observed settlements, with special emphasis when a new excavation phase is done. However, the displaced volume due to excavation should be, in part, distributed to the horizontal displacements of the curtain, which is not the case. The reason for such high vertical settlements obtained in numerical simulation was associated with the sequential lowering of the water level, developed numerically by Plaxis®, with generation of new seepage nets. In each excavation stage the water level was reset by taking the information taken in the piezometers installed near the excavation. However, the seepage generated by the program, with the "ground water flow" routine, it provides a smooth transition between the different positions of the water level at the base of the excavation and in the limits (from 55 m).

This attempt to soften the position of water level contradicts the position of the points where the hydraulic head is known, ignoring that. By this and knowing that software establishes a smoothed seepage network it is possible to infer the following conclusion: it is possible that this area would be constituted by kaolinised granite and with a permeability lower than estimated, in about one order of magnitude. By imposing a lower permeability, the water would have greater difficulty in passing through the soil mass creating a much more abrupt transition from the excavation, being the network closer to the reality taken from the records of the piezometers. Imposing a permeability with two orders of magnitude smaller (10^{-7} m/s) for the W5-W4 horizon, simulating the same conditions as those made in the model, it appears that the groundwater level rises significantly as expected (Figure 6). The vertical settlement, for the same phases, is less than the simulation based on the values previously inferred that are expressed in Figure 7.



a) With kx=ky=10-5 m/s b) With kx=ky=10-7 m/s

Figure 6. Position of the water level due to permeability coefficient variation value (Stage 13)

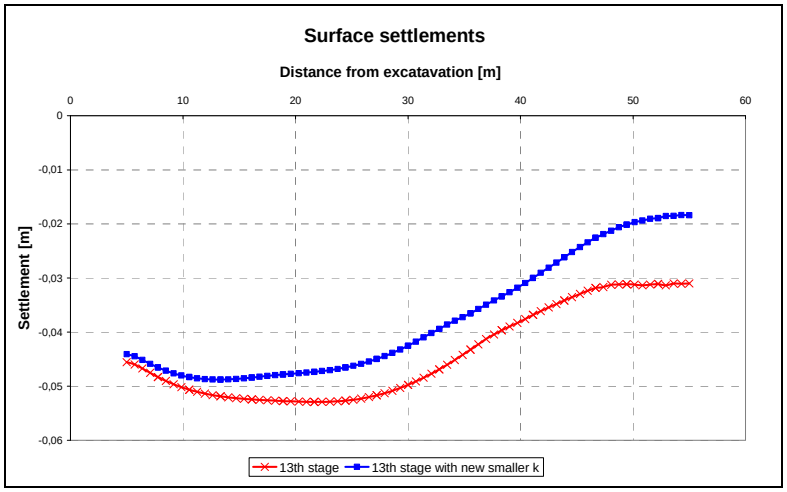


Figure 7. Comparison of the settlements at the surface in stage 13, varying the permeability coefficient

The difference of these settlements can be more than 1 cm, in the furthest area from excavation, keeping the same trend in the later stages. Regarding topographical marks, the situation is similar, which indicates a large vertical surface settlement, which was not observed in the numeric simulation.

Besides the described, there is something that can disturb the credibility of the results obtained through the marks that is the fact of they are supported/fixed to the road surface or on walks, whose rigidity is rather higher than the underlying ground. There is a good and increasing convergence in the mark situated near the building site, supported directly on the soil.

This exception confirms what was mentioned above regarding the difference in vertical settlements. Indeed, looking up the vertical displacements obtained by numerical simulation and

the observation of the target situated on the beam, there is a clear convergence in the results that almost never exceeds 1 mm of difference. That analysis, located at the first beam is not affected by the factors mentioned above, since it is not dependent on changes in groundwater levels and it is not associated to the high rigidity of support of surface marks.

Examining the behaviour of the anchor, whose load values have a different behaviour in each level, there is one that shows a more clear difference in results, the first level (particularly in the initial stages the behaviour is almost symmetric at the beginning). This trend seems to be paradoxical, since in the numerical simulation the load on the anchor is unexpectedly lower after the excavation. This can be explained looking carefully to the directions of the displacement vectors in that area, after the excavation (Stage 3) in Figure 8.

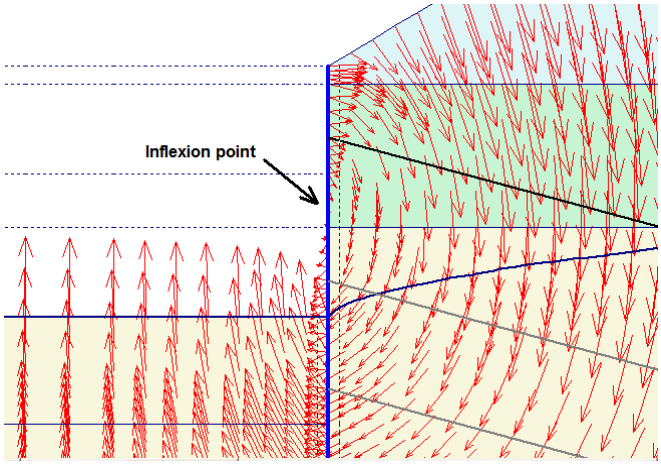


Figure 8. Representation of the displacement vectors in stage 3

The vectors have a different behaviour at the top and at the bottom. The third stage corresponds to an excavation of 4 m (the greatest made in the constructive process), when the curtain is 7 m, without the action of the passive soil contribution. Note that, at this stage, the pre-stress of the second anchor level has not yet been applied. After the excavation, the curtain is subject to the active earth pressure corresponding to 2 and 5 m above and below the anchor's head (which was activated in the previous stage), respectively. Thus, while the passive earth pressure corresponding to the last 5 m, higher than the passive earth pressure corresponding to the first 2 m, there is a tendency for the curtain to move more near the excavation base than at the top, which will create an inflexion point in the intermediate zone. With this inflection point, part of the curtain displaces towards the interior of the excavation and other move in the direction of the soil mass and, being this inflexion point situated below the anchor's head, it moves to the interior of the soil mass lowering its stress (axial load). An inverse situation happens with the records of the cell loads in the first anchor level, where the stress goes up at the excavation but, surprisingly, after performing the pre-stress of the second level there is a relevant increase of the load to 760 kN, never lowering back. As it was mentioned before, the cell load installed were not located exactly in the cross section under study which may, in part, explain the different behaviour of the anchor's load after the first excavation stage. The load cell installed on the first level is really closer to the instrumented and studied section, but it is situated 1 m below the height considered in the numeric modeling for this level (the section used for the modeling is situated just left of the installed load cell). This situation may be the reason why the anchor's load shows a different behaviour to the modeling, being the inflection point situated in a distinct position in the curtain, possibly above the anchor's head. As there is no information about the second level anchor's load, it is probable that it shows some yield behaviour, explaining a higher stress value in the first anchor recorded from the moment of pre-stressing the second anchor. In the third level, although the value inferred by numerical modeling is slightly different from the monitored stresses, the behaviour is similar. On the fifth level, the registration of cell loads has no significant meaning.

4. Conclusive notes

This study about the behaviour of a section in a retaining peripheral structure of the Aliados, in Porto, was developed by performing a numerical analysis based on models well fundament in good characterization tests. The assumptions have been calibrated in order to obtain a good fitting to the observations of the vertical inclinometer, load cells and surface marks. The differences from the observations of horizontal inclinometer were explained from additional analysis of the real induced groundwater flows. The correct definition of the water level from the rest and along the stages, based on information provided by several piezometer installed in the work surroundings, was essential for the successful back-analysis of the position of the water level with the progress of the excavation which allow calibrate the conductivity of the soil mass, based on the results that best fitted the seepage networks, with strong involvement in the quantification of the variation of the effective stress in soil and in the surface settlements. The good and detailed knowledge of the staged timing was crucial for a precise adjustment of the calculation results and observation.

5. Acknowledgments

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6. Bibliography

Brinkgreve, R. B. J., Broere, Waterman, D. (2004). *PLAXIS: Finite Element Code for Soil and Rock Analyses (2D - Version 8)*. Plaxis bv, Delft, Netherlands.

Clough, G.W. e O’Rourke, T.D. (1990). *Construction induced movements of insitu walls.* Design and Performance of Earth Retaining Structures, Geotechnical Special Publication N. 25, Edited by P.C. Lambe and L.A. Hansen, ASCE, pp. 449-470.

Eurocódigo 2 (2004). *Projecto de estruturas de betão*. EN 1992-1-1. Comité Europeu de Normalização, Bruxelas.

NORMETRO (2002a). Memória descritiva e justificativa; cálculos justificativos; Modelo geológico-geotécnico. Estação dos Aliados. Projecto de execução. NORMETRO. Porto.

NORMETRO (2002b). Interpretação dos resultados do levantamento estrutural e avaliação preliminar do risco. Estação dos Aliados. Relatório técnico. NORMETRO. Porto.

Raposo, N.P. (2007). *Pré-dimensionamento de estruturas de contenção ancoradas*. Dissertação apresentada para obtenção do grau de Mestre em Mecânica dos Solos e Engenharia Geotécnica, FEUP. Porto.

Rios Silva, S., Viana da Fonseca, A. & Almeida e Sousa, J. (2008). Modelling of excavation and retaining structure in residual soil from granite. ISSMGE 11th Baltic Sea Geotech. Conf., Sept. Geotechnics in Maritme Engineering ,Vol.1, pp. 765-771. Edit. Mlynarek, Zikora & Dembicki. U.Techn. Gandsk.

Viana da Fonseca, A. (2003). “Characterising and deriving engineering properties of a saprolitic soil from granite, from Porto”. “Characterisation and Engineering Properties of Natural Soils”. Vol.2 pp. 1341-1378. Tan, Phoon, Hight e Leroueil (Ed), Swets & Zeitlinger, Lisse

Viana da Fonseca, A. & Almeida e Sousa, J. (2001). At rest coefficient *of earth pressure in saprolitic soils from granite*. Proc. XIV ICSMFE, Istambul, Vol. 1: 397-400

Viana da Fonseca, A. & Almeida e Sousa, J. (2002). *Hyperbolic model parameters for FEM analysis of a footing load test on a residual soil from granite*. PARAM 2002: Int. Symposium on Identification and determination of soil and rock parameters for geotechnical design. Vol. 1, pp 429-443 Ed. J-P Magnan, Presses L’ENPC , Paris.

Viana da Fonseca, A. e Quintela J. R. (2009). Project and constitutive parameters to model a retaining structure of deep excavation station in Metro do Porto”. First Symposium Méditerranéen de Géo-Engineering, l’USTHB, Alger (In Press).