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Interferences between excavations and tunnels in urban environment – the case of the Quinta dos Candeeiros buildings in Lisbon

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ABSTRACT: In urban environments, excavation works frequently encounter interferences due to the high density of existing underground infrastructure. These conditions often introduce significant complexity into both the design and execution phases, as compromising existing utilities or structures is typically not permitted. This work presents a case study involving the construction of a building at Quinta dos Candeeiros, Moscavide, Lisbon, where the excavation required for two basement levels conflicted with the Red Line tunnel of the Lisbon Metro. This interference forced the development of additional construction measures during the design phase, including the development of an ingenious mechanism to transfer the building loads to the ground without affecting the tunnel. In such complex cases, numerical modelling plays a crucial role in evaluating and optimizing the construction sequence, as well as predicting the interaction between new and existing structures. As presented in this case, the selection of an appropriate framework for describing soil behavior can significantly influence the analysis results, which ultimately may lead to modifications in the construction sequence. Monitoring results confirmed that the adopted construction sequence was effective, as it resulted in minimal ground deformations and ensured the safety of adjacent buildings and the Lisbon Metro tunnel, without compromising the excavation works.

KEYWORDS: Excavations adjacent to tunnels, monitoring, numerical analysis.

1 INTRODUCTION

In urban environments, excavation works are often subject to significant constraints, as much of the underground space is already occupied by existing infrastructure of different types. The presence of these interferences greatly increases the complexity of the construction design and the measures to be adopted during the excavation works, since it is generally not permitted to compromise existing infrastructure. Excavations carried out in close proximity to sensitive infrastructures, such as tunnels, inevitably induce stress redistribution in the ground, leading to soil movements that can potentially damage the tunnel and any adjacent buildings (Ng et al., 2013). Given all the associated constraints, it is impractical to conduct extensive field tests to assess the influence of excavations on tunnels. Consequently, most studies published rely on numerical analysis, using either centrifuge test data as reference (Zheng et al., 2010; Ng et al., 2013, 2015; Huang et al., 2014) or, more frequently, are based on the analysis of real case studies (Lo & Ramsay, 1991; Doležalová, 2001; Sharma et al., 2001; Hu et al, 2003). These numerical studies often involve parametric analyses to evaluate key factors such as the distance between the base of the excavation and the tunnel, as well as the position of the tunnel. All studies consistently indicate that when an excavation is carried out in close proximity to a tunnel, significant ground and structural movements may occur. These movements are highly dependent on the tunnel's relative position to the excavation:

- Tunnels located directly beneath the excavation tend to experience heave, vertical divergence, and horizontal convergence.
- Tunnels positioned beneath the retaining structure exhibit complex behavior, including distortion toward the excavation and asymmetrical displacements.
- Tunnels adjacent to the excavation usually settle, present vertical convergence and horizontal divergence.

It was also observed that the further the tunnel is from the excavation, the smaller the deformations it experiences. Furthermore, an increase in the tunnel lining stiffness was also found to restrain significantly the soil movements. Additionally, due to the stress relief caused by excavation, heave occurs at the base of the excavation and significant tensile strains develop in the tunnel.

Doležalová (2001), when analysing the influence of an excavation carried out above a group of tunnels in Prague, Czech Republic, reported that the region between the tunnel crowns and the base of the excavation yielded due to soil decompression, resulting in significant movements in both the tunnels and the excavation. Lo & Ramsay (1991) demonstrated that the most critical soil parameter for predicting ground movements was the modulus of deformability in extension (unloading). They reported that conservative estimates based on conventional compression triaxial tests can significantly overpredict ground movements, potentially leading to the mistaken conclusion that the project is not feasible. This crucial aspect was also recognized by Ng et al. (2013, 2015), who employed a hypoplastic soil model to simulate the soil behavior during centrifuge tests of an excavation carried out in close proximity to an existing tunnel.

To contribute to the existing knowledge, this paper presents a numerical simulation of the case study of the Quinta dos Candeeiros in Moscavide, Lisbon, where the construction of the buildings' basements was carried out in very close proximity to the Red Line tunnel of the Lisbon Metro, built about 15 years earlier using the NATM method. The results confirm the critical role of the adopted soil model in accurately estimating deformations, demonstrating that the use of an inadequate model may lead to the conclusion that the project is not feasible under the constraints imposed by the Lisbon Metro.

2 THE CASE OF QUINTA DOS CANDEEIROS BUILDINGS

2.1 *Background*

In January 2020, the company Mexto Portugal S.A. initiated the construction project of two lots at Quinta dos Candeeiros

(Figure 1), located between the Moscavide and Encarnação stations on the Red Line of the Lisbon Metro. To the north of the excavation site lies an abandoned industrial building, to the east residential buildings, and to the south and west is Carlos George Street. The project involved the construction of basements with part of the excavation carried out directly above the Red Line tunnel, as shown in Figure 1.

The building in Lot 1 has the plan dimensions shown in Figure 2 and occupies an area of approximately 1,280 m². It includes two basement levels, requiring an excavation depth of 7.5 m. The north side of the excavation lies directly above the Red Line tunnel, with a minimum distance of approximately 4.0 m between the base of the excavation and the tunnel crown. The building in Lot 2 (also shown in Figure 2) has an area of approximately 1,040 m², and includes three basement levels, requiring an excavation depth of 9.7 m. In this case, the tunnel is located laterally to the excavation, with the minimum distance between the retaining structure and the tunnel crown being approximately 4.5 m. However, due to page limitations, this paper will only focus on the analysis of the excavation works performed for Lot 1, which is the most complex.

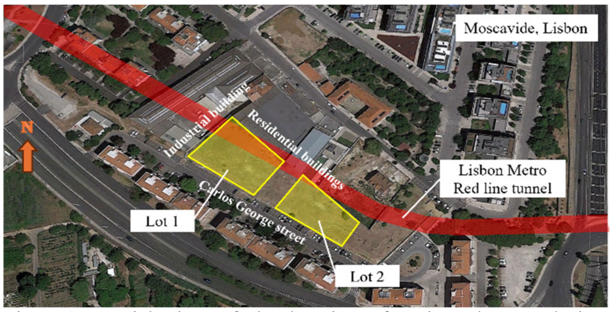

Figure 1. Aerial view of the location of Quinta dos Candeeiros buildings.

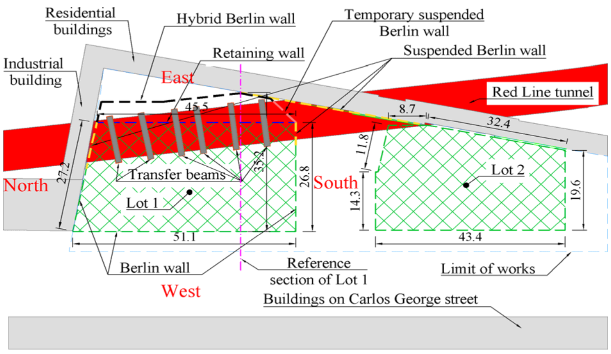

Figure 2. Implantation plant of the buildings of Quinta dos Candeeiros.

2.2 *Ground conditions*

The project is located within the Lower Tagus sedimentation basin, where a Fill layer of anthropogenic origin is commonly found overlying Miocene deposits. In Quinta dos Candeeiros the fill materials are approximately 1.5 m thick and exhibit high variability. They are mainly composed of ceramic debris within a silty-sandy-clayey matrix, characterized by a loose structure, low strength, and high deformability.

Beneath the Fill, three distinct Miocene layers have been identified. The upper layer (ZG3), approximately 6.0 m thick, consists of decompressed sandy silts ("Areolas"), occasionally interbedded with greenish silty clay layers. Below this, an intermediate layer (ZG2) with thicknesses ranging from 6.0 to 7.5 m is composed of compact silty clays, containing shell fragments and minor inclusions of carbonate-sandstone gravel. The lowest and more competent layer (ZG1) comprises very compact silty sands with levels of carbonate-sandstone gravel (Geocontrole, 2020). All layers presented a slight inclination towards southwest.

During the site investigation, the groundwater table was detected very close to the interface between layers ZG1 and ZG2, i.e., below the excavation level and the tunnel crown.

The geotechnical investigation included both field and laboratory tests, from which mechanical parameters were derived for each soil layer. The parameters considered representative of each layer are presented in Table 1.

Table 1. Parameters determined for the soil layers.

Layers	γ (kN/m^3)	c' (kPa)	ϕ' (°)	E (MPa)	ν
Fill	18	2	25	5	0.3
ZG3	20	19	33	40	0.3
ZG2	20	23	33	50	0.3
ZG1	21	5	36	60	0.3

2.3 *Construction sequence*

Given the constraints imposed by the presence of the Red Line tunnel, the design of the retaining structure presented significant challenges. The design was carried out by JETsj (2021), who establish a cautious and well-defined construction sequence. Considering the geotechnical conditions and the displacement limits imposed for both the excavation and adjacent structures, Berlin-type retaining walls were adopted for Lot 1.. On the sides where the retaining structure was not located directly above the tunnel, a permanent Berlin wall was implemented (Figure 2). The construction of the Berlin wall followed the typical sequence, beginning with the installation of the soldier piles, followed by the construction of the capping beam, and then by the alternate excavation of panels and the application of support through anchored reinforced concrete panels. Each excavation level was only initiated after the completion and support of the preceding one.

In the sections directly above the tunnel the construction system was more complex. On the north and south sides, where the retaining structure could not be founded in the ground due to the tunnel's presence, the Berlin wall was suspended from a steel truss beam spanning the tunnel's width. As a result, the wall's weight and the earth pressure were transferred to the upper truss beam rather than to the ground.

On the east side, the building wall was positioned directly above the tunnel, preventing direct load transfer to the ground. To overcome this, the excavation limit was extended away from the tunnel, and a hybrid Berlin wall was installed. In this solution, the upper part consisted of reinforced concrete panels supported by soil nails, while the lower part used wooden planks as lagging and ground anchors. The future building's loads would be diverted to the tunnel sides through a transfer system consisting of six reinforced concrete beams capping inclined micropiles, as illustrated in Figure 3. In order to comply with Lisbon Metro requirements, a minimum distance of 3 m between the micropiles and the tunnel had to be maintained (Figure 3). To reach the desired ground level on the east side a retaining wall was constructed, and the space between it and the excavation limit was filled with backfill.

Figure 3 presents the reference cross-section of Lot 1 at the final stage of the excavation, highlighting the adopted transfer system designed to protect the tunnel from the loads from the building and backfill. Figure 4 shows two photographs of the north and east sides of the excavation during the construction of the retaining wall (a) and backfilling works (b). The images clearly display the top of the transfer beams, the permanent Berlin wall on the north side, and the hybrid Berlin wall on the east side.

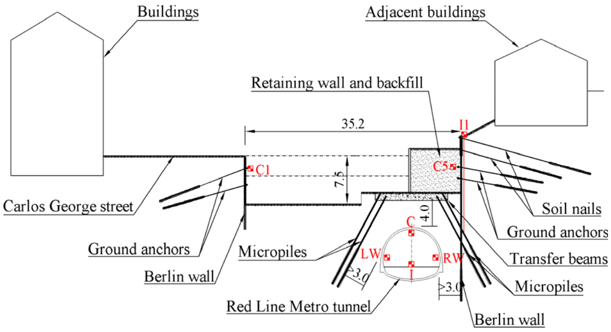

Figure 3. Vertical cross-section of the excavation along the reference section of Lot 1.

Figure 4. Views of the bottom of the excavation performed in Lot 1.

2.4 *Monitoring scheme*

As previously mentioned, Lisbon Metro specifies that no construction should take place within 3 m of its underground structures. For above-ground structures, a minimum safety distance of 8 m must be maintained from the outer face of the Metro structures along their full height. Furthermore, instrumentation of the Metro tunnels and galleries must be carried out in monitoring sections located within the zone of influence, spaced at least every 15 m in straight segments and every 5 m in curved segments. In each section, convergence must be assessed through readings taken at several level marks, performed at least one to two times per week. Movements along cracks should also be monitored using crackmeters, and lateral ground movements should be assessed using inclinometers. According to Lisbon Metro requirements the alert limits for movements are 7 mm in the structures and 3 mm in the rails. The alarm limits correspond to maximum movements of 10 and 5 mm, respectively.

At Quinta dos Candeeiros, a total of 18 monitoring sections were instrumented, spaced 8 m apart. In each section, topographic targets were installed on the tunnel crown (C) and sidewalls (LW and RW), and levelling marks were placed on the tunnel invert (I) as shown in Figure 3. To monitor the excavation works, one inclinometer was installed near the reference cross-section of Lot 1, along with five load cells placed at various positions and depths to monitor the loads on the ground anchors. The groundwater table was monitored using two piezometers, while several survey topographic marks were installed on adjacent buildings to assess potential movements. Figure 3 depicts the relevant instruments installed near the reference cross-section of Lot 1.

3 NUMERICAL MODELLING OF THE EXCAVATION

3.1 *Finite element model*

To assess the influence of the excavation on the Red Line tunnel, 2D numerical analyses were carried out based on the reference cross-section of Lot 1 (Figure 3). The analyses were conducted using RS2 software (Rocscience, 2024), replicating the entire construction sequence defined by JETsj (2021), and evaluating the influence of the constitutive model adopted to simulate the soil behavior (Costa, 2022). A detail of the mesh used in the analysis is shown in Figure 5, consisting of 3375 6-noded triangular elements. A denser mesh was applied in the vicinity of the tunnel and excavation to better capture the expected stress and strain gradients. To minimize boundary effects, the model's lateral limits were set 70 m from the tunnel centerline, which also corresponds to the center of the model. The bottom boundary was placed 15 m below the tunnel invert, aligned with the top of the bedrock layer. Standard boundary conditions were applied in the model: free ground surface, no displacements allowed in the bottom boundary, and only horizontal displacements were permitted along the lateral boundaries. To simulate the loading imposed by the buildings, uniform surface loads of 10 kN/m² per floor were considered. Accordingly, the buildings on Carlos George Street were represented by a load of 60 kN/m², and the adjacent buildings by a load of 40 kN/m².

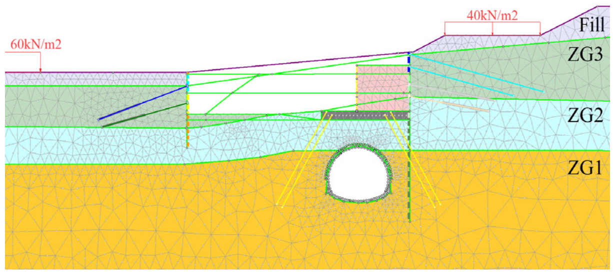

Figure 5. Detail of the finite element mesh at the last stage of the excavation.

The four geotechnical layers were considered in the model based on the previously described geological sequence. Given the sandy nature of the soils and the depth of the groundwater table (located below the tunnel crown) a dry analysis was deemed adequate for representing the site conditions. For the initial analysis, soil behaviour of the four layers was simulated by an elastic-perfectly plastic model with failure governed by Mohr-Coulomb criterium (MC), employing the parameters listed in Table 1. A standard formulation of the model was adopted, in which the loading (E) and unloading stiffnesses (E_u) were considered to be equal. A linear elastic model was used for both the backfill and the concrete in the transfer beams. For the backfill, a Young's modulus (E) of 30 MPa and a Poisson's ratio (ν) of 0.3 were adopted, while for the concrete, values of 32 GPa and 0.2, respectively, were considered.

The retaining structures were modelled as beams, with the stiffness initially corresponding to the soldier piles (Table 2). As the excavation progressed, the model was updated to include the additional stiffness of the concrete panels (E=20 GPa, ν=0.2) in the case of the Berlin wall (0.3 m thick on the Carlos Georges Street side and 0.4 m thick on the adjacent buildings), and of the wooden planks (E=8 GPa, ν=0.2) in the case of the temporary Berlin wall. The retaining wall was also modelled as a concrete beam (E=20 GPa, ν=0.2) with a thick of 0.4 m.

The remainder structural elements – ground anchors, soil nails and micropiles – were modelled as tieback elements with the properties listed in Table 2.

Structural interfaces were not considered in the model since all structural elements were directly cast against the soil.

Table 2. Parameters of the structural elements used in the analyses.

Type	A (cm^2)	E (GPa)	L_{Total} (m)	L_{Grout} (m)	s (m)	P_0 (kN)
Soldier piles (HEB140):						
- Carl. George Str.	42.96	200	11.5	5.2	2.25	-
- Adj. buildings			25.6	15.4	0.80	
Ground anchors:						
- Carl. George Str. – L1	0.66	200	14.8	7.0	3.00	550
- Carl. George Str. – L2			13.8	6.9		600
- Adj. buildings – L1			13.6	6.8		460
- Adj. buildings – L2			12.5	7.2		400
Soil nails:						
- Slope – L1	0.32	200	21.8	21.8	0.80	-
- Slope – L1			16.8	16.8		
Micropiles:						
- Transfer system	2.48	200	16.0	16.0	5.80	-

Table 3. Construction sequence adopted in the numerical model.

Stages	Description
1	Generation of the initial gravitational stress state;
2	Excavation of the Red Line tunnel using the stress relief method with α=50%;
3	Installation of the tunnel lining and application of the remaining forces (1- α=50%);
4	Installation of the steel soldier piles of the Berlin wall on both sides of the excavation;
5	Partial excavation of the first level (L1) on the Carlos Georges Street side;
6	Installation of the L1 support: concrete panels and pre-stressed ground anchors;
7	Partial excavation of the second level (L2) on the Carlos Georges Street side;
8	Installation of the L2 support: concrete panels and pre-stressed ground anchors;
9	Installation of the first level (L1) of soil nails on the side of the adjacent buildings;
10	Excavation of the slope between the two sides of the excavation;
11	Installation of the support: concrete panels and L2 of soil nails on the side of the adjacent buildings;
12	Completion of L1 excavation;
13	Installation of the L1 support: wooden planks and pre-stressed ground anchors on the side of the adjacent buildings;
14	Completion of L2 excavation;
15	Installation of the L2 support: wooden planks and pre-stressed ground anchors on the side of the adjacent buildings;
16	Excavation for the construction of the transfer beams;
17	Installation of the transfer beams and micropiles;
18	Installation of the retaining wall and placement of backfill up to the final level. Deactivation of the ground anchors on the side of the adjacent buildings.

The construction sequence adopted in the model aimed to replicate the stages carried out on site with the highest possible accuracy. However, some simplifications were necessary, as it is impossible to replicate every step in a 2D model. Therefore, the prior excavation of the tunnel was simplified into only two stages, by applying the stress relief method. During the tunnel excavation stage, a stress relief factor (α) of 0.50 was considered, meaning that only 50% of the load related to the excavation was released. In the subsequent stage, when the tunnel lining was installed, the remaining 50% of the load was released. Since approximately 15 years elapsed between the tunnel construction and the beginning of the Quinta dos Candeeiros project, it was assumed that the stress and strain fields had already stabilized due to the tunnel opening. Consequently, all displacements were reset to zero, and the stages related to the excavation of Lot 1 began with the installation of the soldier piles. In total, the construction sequence was summarized in 18 steps, as described in Table 3.

3.2 *Calibration of the numerical model*

The calibration of the numerical model and the assessment of the excavation behavior were carried out by comparing the numerical results with the data obtained from the instrumentation installed in the tunnel (Costa, 2022).

Figure 6a shows a comparison between the vertical displacements of the survey marks obtained from the numerical model (MC) and those measured during the construction work. It can be seen that, qualitatively, the deformation trends are very similar, with the numerical model successfully simulating the heave caused by the excavation on the side of the adjacent buildings (stage 9 onwards), as well as its reduction during the construction of the transfer system and subsequent backfilling (stages 17 and 18). Similarly to what is observed in the literature, the numerical model also adequately predicts the vertical divergence of the tunnel and its slightly inward distortion towards the center of the excavation. However, the predicted magnitude of the movements is significantly greater than observed, with a maximum heave at the tunnel crown estimated at of 30 mm compared to only 3 mm recorded. According to the numerical analysis, the movement at the rails (invert – I) would have reached 15 mm, well above the limit imposed by the Lisbon Metro, thus rendering the project unfeasible.

To improve the prediction, a second analysis (MCU) was conducted using a non-standard formulation of the elastic-perfectly plastic model, which allows the loading and unloading stiffnesses to be distinguished. Following the results obtained by Lo & Ramsay (1991), the unloading stiffness of all four soil layers was increased by a factor of 3, while the loading stiffness and all other soil parameters were kept constant. The results obtained, presented in Figure 6b, show that while the general trend remains consistent, there is a significant reduction in movements, with a maximum heave of approximately 12 mm at the tunnel crown and about 4 mm at the invert. These results clearly highlight the importance of adequately selecting the unloading stiffness modulus in excavation works. However, despite the significant improvement in predictions, the movements estimated in the MCU analysis are still higher than those measured in the tunnel.

A third analysis (HS) was carried out using the Hardening soil model (Schanz et al., 1999), which not only distinguishes loading and unloading stiffness but also incorporates stress-dependency, thereby offering a more realistic representation of soil behavior. Based on the laboratory tests results and supported by past experience in similar cases, the default soil parameter relationships were adopted in this analysis, with E=E_{50}=E_{oed} and E_u=E_{ur}=3E. The predicted results are presented in Figure 6c, where a further considerable improvement can be observed. In this analysis, the maximum heave predicted at the tunnel crown is approximately 9 mm, and about 3 mm at the invert, values which compare more reasonably with the measured in the tunnel. Nevertheless, these results suggest that conservative parameters were derived from the results of the geotechnical investigation. While this reflects a safer and common practice in Geotechnical Engineering, in scenarios such as this one, it can pose a problem, as Metro Lisbon imposes strict limitations on allowable movements and may consider the project unfeasible based on these predicted values.

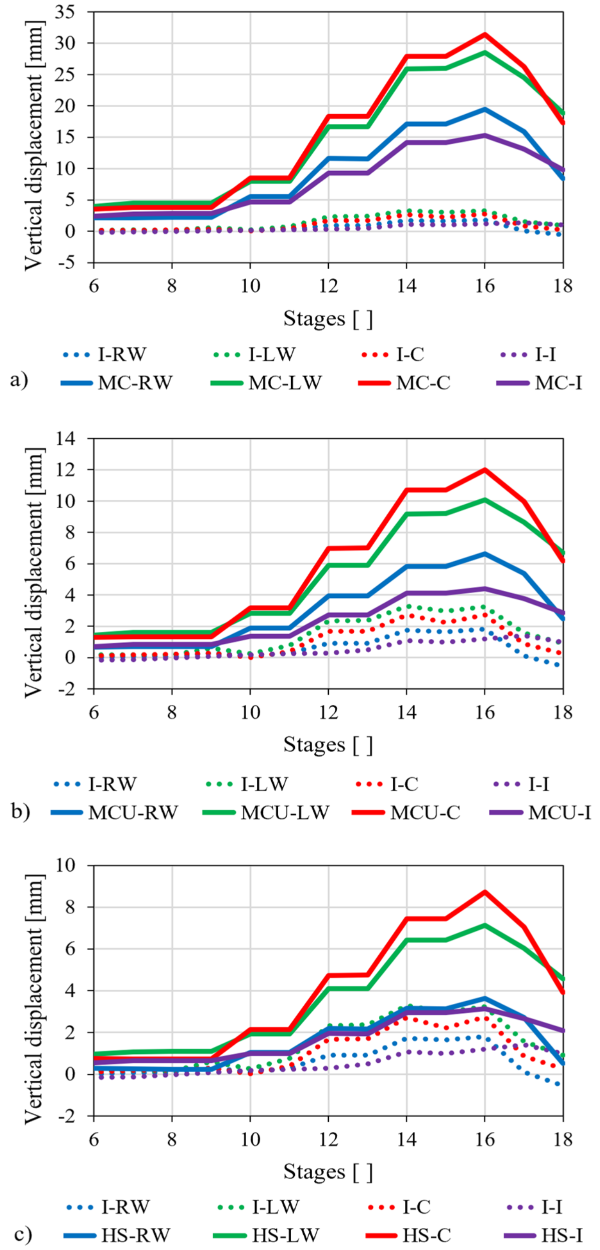

Figure 6. Influence of the constitutive model adopted on the prediction of the tunnel vertical displacements during excavation.

4 EXCAVATION BEHAVIOR

In this section, the main results of the HS analysis are presented, as it provided the best estimate of both the behavior and the magnitude of the movements that occurred during the excavation.

As expected, the excavation influences the movements in the tunnel lining, leading to vertical divergence and slight distortions toward the center of the excavation. The total displacements predicted for stages 16 (final excavation) and 18 (backfilling) are presented in Figure 7. In stage 16 the upward movements and vertical divergence reach their highest values, which tend to decrease during the backfilling stage (stage 18). In contrast, the asymmetric loading introduced by the backfill increases the distortions of the tunnel lining toward the center of the excavation, resulting in a slightly more asymmetric deformed tunnel shape.

The plastic zones developed in the ground throughout the construction sequence are shown in Figure 8. The upper image corresponds to Stage 16, where more significant plasticization can be observed in the area of the anchors located beneath Rua Carlos George, behind the retaining structures, and between the tunnel crown and the base of the excavation, where a yielded soil wedge is formed. In the lower image, which represents the final stage (Stage 18), the extent of plasticization remains largely unchanged, despite the recompression induced by the backfill on the soil above the tunnel crown.

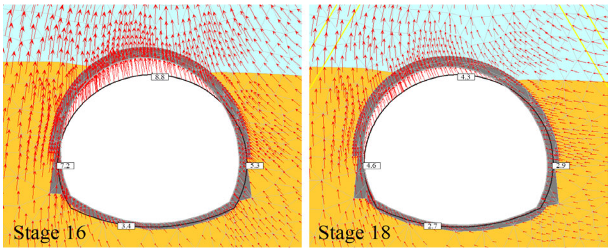

Figure 7. Deformations predicted for the tunnel lining.

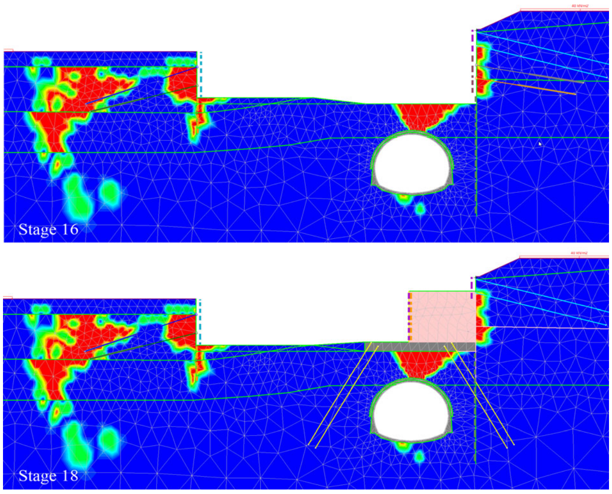

Figure 8. Plastic zones around the excavation pit.

A comparison between the horizontal displacements measured by inclinometer I1 and those predicted by the HS model is presented in Figure 9 for stages 13, 14 and 17. It should be noted that, in the instrumentation survey, the displacements at the base of the inclinometer were assumed to be zero. However, given the short length of the instrument, it is very likely that ground movements were occurring at that level, as indicated by the numerical analyses. By zeroing the horizontal displacements of the numerical models at the base of the inclinometer, it can be seen that the model reasonably predicts the increase in displacements associated with the excavation progress. In this case as well, the model tends to overestimate the lateral movements observed on site, although their magnitude remains quite small, highlighting the effectiveness of the adopted construction sequence in controlling deformations.

The loads in ground anchors C1 and C5 recorded during monitoring are compared with those predicted by the numerical model in Figure 10. It can be observed that the load in ground anchor C1 showed a slight decrease in the prestressing load (Table 2) throughout the construction sequence. In contrast, the HS model predicts a significant drop of about 100 kN at the stage when the L2 ground anchor is installed and prestressed (stage 8), followed by a slight reduction similar to that observed on site. The behavior of ground anchor C5 is even more contrasting, with only a minimal reduction in the prestressing load (Table 2) recorded at site, whereas the HS model predicts a substantial increase in load immediately after installation, caused by the excavation of L2, which then tends to stabilize over the remaining construction stages. These differences, although significant, can likely be attributed to corner effects, since the south side of the retaining structure is close to the locations of the monitored ground anchors – particularly ground anchor C5, where the largest discrepancies in behavior are observed.

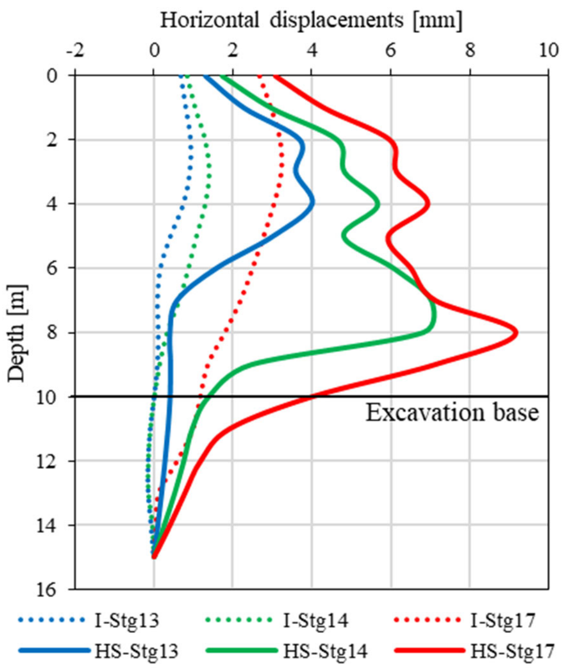

Figure 9. Comparison of the horizontal movements in inclinometer I1.

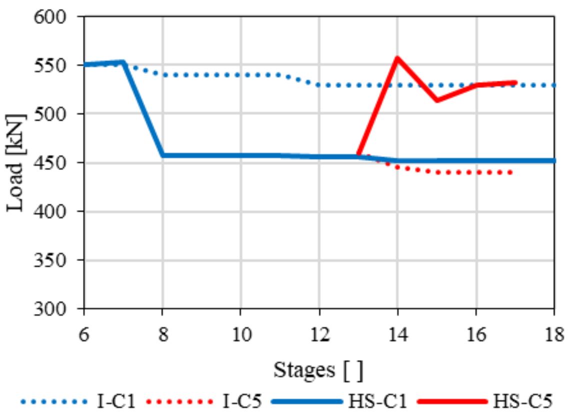

Figure 10. Comparison of the load in ground anchors C1 and C5.

5 CONCLUSIONS

The excavation at Quinta dos Candeeiros, carried out in close proximity to the Red Line Lisbon Metro tunnel, presented significant challenges due to strict deformation limits imposed by Lisbon Metro. The adopted construction sequence proved to be reliable and effective, successfully limiting ground movements and maintaining tunnel performance within the allowable thresholds. Instrumentation data confirmed that the design strategy, including the use of transfer systems and staged excavation, was appropriate and well implemented.

The numerical analyses performed highlighted the critical importance of selecting suitable constitutive models to simulate soil behavior. Initial simulations using Mohr-Coulomb models predicted deformations that would render the project unfeasible. However, by introducing more advanced models – namely, the Hardening Soil model – predictions aligned much more closely with field measurements, allowing for a more realistic interpretation of the behavior of the excavation and interaction with the tunnel. Naturally, the use of such models relies on accurately estimating the necessary parameters, which can only be ensured through a comprehensive geotechnical survey complemented by appropriate laboratory testing.

These findings demonstrate that the apparent risk associated with deep excavations near sensitive infrastructure can be mitigated through a combination of advanced modeling techniques and a meticulously planned construction sequence. Moreover, they emphasize that overly conservative assumptions in geotechnical design may lead to misleading conclusions about the feasibility of otherwise viable engineering solutions.

6 ACKNOWLEDGEMENTS

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