XVIII EUROPEAN CONFERENCE ON SOIL MECHANICS AND GEOTECHNICAL ENGINEERING

August 26th to August 30th 2024 – Lisbon, Portugal

Session A9
Theme A: Piles

593 Combined pile-raft and raft foundation modelling and design for three distinct office buildings in Lisbon, Portugal
A. Sousa, A. Pinto, N. Silva; Presenting Author: André Sousa

Session A10
Theme A: Modelling; Education

233 Use of BIM methodology in Geotechnical Projects – Construction of underground reservoirs
A. Henriques, N. Silva, M. Lopes; Presenting Author: André Henriques

770 Deep excavation solutions for the construction of a logistic park in Loures (Lisbon)
A. Henriques, R. Tomásio, F. Ramalho, J. Dias; Presenting Author: Rui Tomásio

Session D1
Theme D: Metro tunnels and excavations

545 Lisbon new circular Metro underground line: Centenary buildings underpinning
C. Fartaria, A. Henriques, C. Martins, A. Pinto, R. Tomásio; Presenting Author: Catarina Fartaria

540 Lisbon new circular metro line: interconnection section between new line and existing terminus
P. Marques, C. Martins, C. Fartaria, R. Tomásio, A. Pinto; Presenting Author: Carlos Martins

530 Lisbon new circular Metro line: Buildings underpinning
C. Martins, C. Fartaria, R. Tomásio, A. Pinto; Presenting Author: Carlos de Oliveira Martins

Session D3
Theme D: Ground improvement

457 Ground improvement solutions at the plot 14 of the Northern Lisbon logistic platform
M. Lopes, A. Pinto, A.L. Gonçalves, J. Moreno; Presenting Author: Miriam Lopes

Session D5
Theme D: Excavations; Tunnelling; Monitoring

489 Excavation, retaining wall and deep foundations solutions for an office building at Alcântara
R. Tomásio, F. Veloso, N. Rosa; Presenting Author: Rui Tomásio

987 Solutions of excavations and peripherical earth retaining walls in dense urban área
L. Fernandes, R. Justiniano

Session D10
Theme D: Excavations; Tunnelling; Monitoring

53 Solution of excavation and peripheral earth retaining walls near sensitive structures – O'Living Development, Lisbon
R. Justiniano, A. Pinto

243 Deep excavation solutions in an urban environment - Distrikt residential project, Lisbon
J. Silva, I. Braz, A. Pinto; Presenting Author: Joana Silva

948 Earth retaining solutions and facades underpinning, for the refurbishment an historic building, in Lisbon
I. Braz, A. Pinto, D. Henriques, R. Santos; Presenting Author: Inês Braz

Session E2 Theme E: Flood protection structures

978 Temporary cofferdam at Caniçada Dam, Portugal
A.Pinto, L. Caldeira, F. Cerqueira, N. Plasencia, F. Gomes; Presenting Author: Alexandre Pinto

Poster Session

587 Foundation solutions near Trancão river, in Lisbon
I. Braz, A. Pinto, C. Simões, T. Gomes, C. Caxias

870 Coastal cliff stabilization and access stair reconstruction at Peneco beach, Algarve, Portugal
C. Fartaria, A. Pinto, J. Dinis, P. Nunes, M. Silva; Presenting Author: Catarina Fartaria

904 Ground improvement and special foundations at North Lisbon Logistic Platform, plot 1, Portugal
A. Pinto, M. Lopes, P. Marques, A.L. Gonçalves, A. Perez, J. Moreno, J. Moreno; Presenting Author: Miriam Lopes

972 Special foundations at the cycle-pedestrian bridge over the Trancão river, Portugal
A. Pinto, A. Rodrigues A. Rodrigues, P. Santos, C. Caxias; Presenting Author: Ana Teresa Rodrigues

981 Deep and complex excavation in an urban environment in Miraflores, Oeiras
R. Justiniano, A. Pinto; Presenting Author: Ricardo Justiniano

986 Excavation, earth retaining solutions and facades underpinning of a historic building in Estoril, Portugal
J. Silva, C. Martins, A. Pinto; Presenting Author: Joana Silva

988 Industrial warehouse retaining walls and pavements strengthening at Quinta do Adarse, Alverca, Portugal
T.M.L. Paludeto, M. Lopes, A. Pinto; Presenting Author: Thaís Maria Paludeto

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Combined pile-raft and raft foundation modelling and design for three distinct office buildings in Lisbon, Portugal

Modélisation et conception combinées de fondations sur pieux et sur radiers pour trois différents immeubles de bureaux à Lisbonne, au Portugal

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ABSTRACT: In recent years many projects were designed using the Combined Pile-Raft Foundation (CPRF) concept. Combined Pile-Raft Foundations have a complex soil-structure interaction scheme including the pile-soil interaction, pile-pile interaction, raft-soil interaction, and finally the pile-raft interaction. Consequently, there is a need for 3D numerical models that can study this complex interaction. In this paper, several 3D models are presented and discussed for the foundation design of three office buildings, that were built using the Combined Pile-Raft and Raft Foundations solutions, near the right bank of the Tagus River in Lisbon, Portugal. The study was based on geotechnical information provided by the site investigation and by Static and Dynamic Load Tests on Driven Piles, both performed in several spots evenly distributed at the site. The developed 3D model was able to simulate the behaviour of the piled raft foundation system.

RÉSUMÉ : Ces dernières années, de nombreux projets ont été conçus en utilisant le concept de fondation combinée sur pieux et radeau. Les fondations combinées pieux-radeau ont un schéma d'interaction sol-structure complexe comprenant l'interaction pieu-sol, l'interaction pieu-pieu, l'interaction radeau-sol et enfin l'interaction pieu-radeau. Par conséquent, il existe un besoin de modèles numériques 3D capables d'étudier cette interaction complexe. Dans cet article, plusieurs modèles 3D sont présentés et discutés pour la conception des fondations de trois immeubles de bureaux, qui ont été construits à l'aide des solutions de fondations combinées Pile-Raft et Raft, près de la rive droite du Tage à Lisbonne, au Portugal. L'étude s'est basée sur les informations géotechniques fournies par l'investigation du site et par des essais de charge statique et dynamique sur pieux battus, tous deux effectués à plusieurs endroits uniformément répartis sur le site. Le modèle 3D développé a pu simuler le comportement du système de fondation sur radiers sur pieux.

Keywords: 3D Modelling; Raft Foundation; Combined Pile-Raft Foundation; Finite Element Method

1 INTRODUCTION

The use of piles, strategically located, within a raft foundation has been a commonly accepted practice, which has been used in many projects, to improve both the ultimate load capacity and the settlement and differential settlement performance of the raft.

This paper presents the modelling and geotechnical design of pile-raft and raft foundation solutions for three office buildings built close to the Tagus River, in Lisbon, Portugal (Figure 1). The proximity to the river brought geological and geotechnical restraints that will be further discussed and analysed in the next sections.

The three lots had different constraints regarding the number of basements, and the total number of stories as shown in Table 1:

Table 1. Geometry information table on the 3 lots.

Designation	Area (m2)	Stories	Basements	Excavation Depth (m)
Lot 1	2000	12	3	11
Lot 2	6350	9	2	6
Lot 3	5000	8	2	7

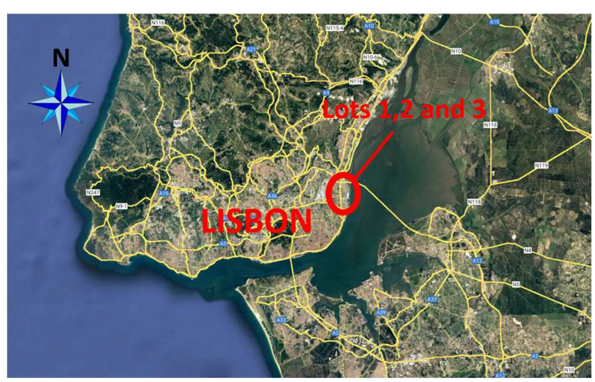

Figure 1. (Google Earth) Satellite location of the 3 lots.

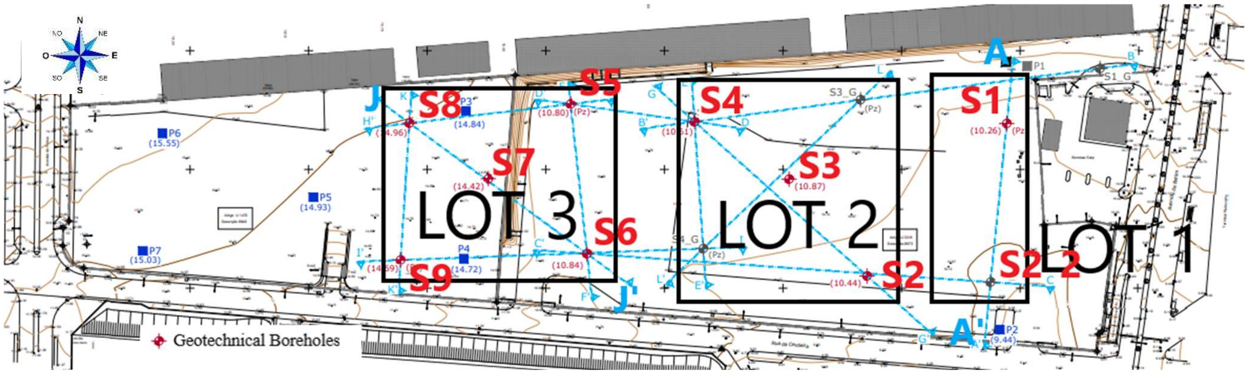

Figure 2. Building lots and boreholes for geotechnical investigation.

2 GEOTECHNICAL CONSTRAINTS

This section presents the main geotechnical constraints that affected the solutions studied.

2.1 Geotechnical investigation

A site investigation campaign consisting of eleven boreholes was conducted with SPT tests. The location of the boreholes is shown in Figure 2.

In general, the area where the lots are located is at a low topography level in an alluvial zone, corresponding to old water courses that formerly flowed into the Tagus River, with a general orientation WNW-ESE (main water course) and SW-NE (secondary water course). Under these alluvial deposits layer is the Miocene layer, characterized by the "Areolas de Braço de Prata" which is formed by dense sands and clays (as illustrated in Figure 3 Figure 4).

Throughout the area landfill deposits were identified, resulting from various anthropic actions developed over time and of diverse nature.

Table 2 shows the several layers detected in the investigation campaign, along with the SPT test results.

Table 2. Prospected layers by the investigation campaign.

Geotechnical Zone	Lithology	NSPT
ZG1	Landfill and other materials that may have been displaced	1-17
	Alluvial Deposits	0-8
ZG2A	Fine silty-clayey sands and sandy silts	10-41
ZG2B	Silty clays, clayey silts and sandy silts	24-60
ZG2C	Fossiliferous lumachelic/calcarenite levels	>60

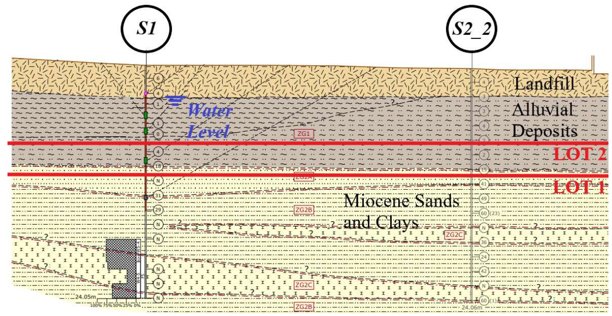

Figure 3. Site investigation geotechnical profile (Section Cut AA' from S1 to S2_2 at lot 1).

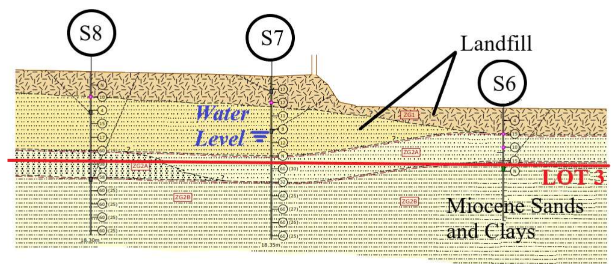

Figure 4. Site investigation geotechnical profile (Section Cut JJ' from S8 to S6 at lot 3).

2.2 Hydrology and permeability

The study of the ground permeability, as well as of the water lines crossing the site area, proved to be particularly important for the design and execution of the foundation solutions.

These permeabilities were evaluated based on laboratory analyses of samples taken during the boreholes drilling.

The Miocene layers in the investigated area are composed of fine silty-clayey sands, sandy silts, and silty clays, sometimes with the intercalation of very fossiliferous levels. In general, this Miocene complex has porosity permeability (primary), but locally it may present fissure permeability in the more carbonate/fossiliferous layers (secondary permeability).

This Miocene complex, due to its fine sandy, silty-sandy, and clayey-sandy composition, has overall moderate to low permeability, providing unfavourable conditions for water percolation, with moderate to not significant flows. The retaining walls for the excavation of all lots reached this layer with lower permeabilities, which drastically reduced the water inflow into the excavation pits. In general, the foundations for the three lots were several meters below the investigated ground water level.

3 FOUNDATION SOLUTIONS

Two types of foundation solutions were adopted for the three lots.

The main criteria for choosing the solution was the control of differential settlements. In cases where the foundation level was directly over the more dense Miocene layer, a raft solution was recommended. When the foundation level intersected alluvial deposits, a combined pile-raft foundation solution (CPRF) was studied to control deformations.

3.1 Raft foundation

The raft foundation solution consists of a continuous slab, connected to the retaining walls by bolts. In the areas where there are higher loads, as for example under the lifts and stairs boxes, greater thicknesses were considered in order to better control differential settlements.

This type of solution allows the hydrostatic vertical uplift pressures, which can be mobilized in the long term, to be accommodated, without having to provide an extensive drainage solution.

3.2 Combined pile-raft foundation

The combined pile-raft foundation (CPRF) consists of a raft foundation (as described before) with soil inclusions in areas with greater concentration of loads. In this case, rectangular barrettes were considered, using the same diaphragm wall technology used for the execution of the retaining walls. At areas under structural columns, thicker areas of the slab were adopted, with driven precast piles resting over the more dense Miocene layer (Miocene sands and clays), in order to reduce settlement in these areas and to homogenise displacements throughout the foundation (Figure 5).

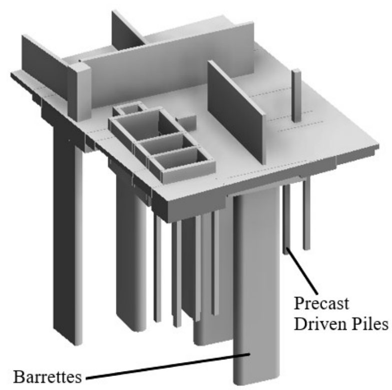

Figure 5. Pile-raft foundation solution at lot 2.

4 STRUCUTRAL DESIGN

4.1 Raft foundation design

The design of a raft foundation consists of the distribution of the loads that come from the structure to the slab, which in turn will transmit them to the ground.

The key issue to this type of design is to minimise the differential settlements derived from the heterogeneous spatial distribution of structural loads.

In this particular case, there is also the action of uplift hydrostatic pressures, which were taken into account in the long-term design of this slab.

To reduce the differential settlements, thicker areas of the foundation slab were adopted under the structural columns and walls.

The design of raft foundation solutions considered the following main design states:

- Maximum total settlements;
- Differential settlements;
- Bending moments and shear forces at the raft.

4.2 Combined pile-raft foundation design

There are several design methods for combined piled-raft foundations, with guidelines from Poulos (2000, 2001), Poulos et al. (2011), Fellenius (2015), among others. The main goal is that the load is transferred to the ground by both the raft and piles. The loads are resisted by the bearing capacity between the raft and soil and by the lateral friction and end bearing capacity of the piles. The design method should comprise 4 phases:

- Preliminary study where the viability of the solution is analysed;
- Mapping of the piles and definition of their general characteristics;
- Detailed study of the optimal number of piles;
- Estimation of settlements, bending moments, and shear forces at the raft, as well as axial loads and bending moments at the piles.

The design of CPRF solutions considers, according to Poulos (2001):

- Ultimate bearing load for axial, shear stresses, and bending moments;
- Maximum total settlement;
- Differential settlement;
- Bending moments and shear forces at the raft;
- Bending moment and axial loads at the piles.

5 IN SITU TESTS – PDA

To estimate the real bearing capacity of the precast driven piles, 9 load dynamic load tests were performed using the “Pile Driving Analyser (PDA)” method. The tests were performed on a square prefabricated reinforced concrete piles of section 300x300mm2 (Figure 6).

The results obtained from the dynamic load test indicate several layers of low to medium strength up to a depth of approximately 2.9 to 13.1 m, where the shaft resistance is lower than 90 kPa. Under this layer, a zone of higher resistance was found, with a shaft resistance ranging from 128 to 524 kPa. The tip resistance for these piles was between 14.6 and 18.2 MPa.

These values (both for the shaft and tip resistance) are an underestimation since low energy was being transferred during these tests to guarantee pile integrity.

Figure 6. Pile Driving Analyser preliminary tests being executed at lot 2.

6 MODELLING

The modelling was performed using software PLAXIS 3D. For the pile-raft foundation of lot 2, PLAXIS 2D axisymmetric analysis were also used for the calibration of the modelled inclusions with empirical methods widely used in the past.

6.1 FE models for structural elements

Different finite element (FE) models were chosen for each of the structural elements to simulate their behaviour in both the short and long term.

6.1.1 Foundations slabs

The foundation slabs were modelled with plate elements (PLAXIS, 2023).

6.1.2 Precast driven piles and barrettes

Both the precast driven piles and the barrettes were simulated using the embedded beam element in PLAXIS 3D. These elements describe the interaction between the piles and the surrounding soil. The interaction at the shaft and at the tip is described by means of embedded interface elements. The pile is considered as a beam which can cross a volume element at any place with any arbitrary orientation. Due to the existence of the beam element three extra nodes are introduced inside the volume element.

6.1.3 Connection between the foundation slab and the retaining wall

The retaining wall and the foundation slab are connected by a linear element which allows shear forces to be transmitted, however rotations are free. This behaviour allows the simulation of the bolted connection between the two elements.

6.1.4 Structural columns and walls

The structural loads for all relevant combinations, are simulated in the model by means of point loads applied at the gravity centre of the structural columns and walls.

The structural wall behaviour was simulated by implementing a rigid beam along the wall centreline, which will redistribute the point load along the walls path, and therefore avoiding any numerical problems that may appear due to the high concentrated loads (applied at the walls gravity centres).

6.1.5 Soil elements

The soil was modelled using the hardening soil model, with volume elements in the 3D models. The Miocene sand layers were set to respond with a drained behaviour and the Alluvial deposits, and the Miocene clay layers were set to respond with an undrained behaviour (effective stress analysis) (Table 3, Table 4 and Table 5).

Both short term and long-term situations were accounted for in the analysis.

Table 3. Material models adopted in the 3D Model.

	Properties	
	Material model	**Drainage type**
ZG1	HS	Undrained (A*)
ZG2A	HS	Drained
ZG2B	HS	Undrained (A*)
ZG2C	HS	Undrained (A*)

**Undrained calculations were performed using effective stress properties. Long-term behaviour was also considered in the model.*

Table 4. Material stiffness in the 3D Model.

	Stiffness		
	E_{50}^{ref} (kPa)	**E_{oed}^{ref} (kPa)**	**E_{ur}^{ref}(kPa)**
ZG1	10000	10000	30000
ZG2A	50000	50000	150000
ZG2B	100000	100000	300000
ZG2C	150000	150000	450000

Table 5. Material strength parameters in the 3D Model.

	Resistance	
	c'^{ref} (kPa)	**φ' (º)**
ZG1	1	28
ZG2A	1	38
ZG2B	40	36
ZG2C	20	39

6.1.6 *PLAXIS 2D axisymmetric modelling for embedded beam behaviour calibration*

The three-dimensional model for lot 2 was calibrated using the axisymmetric mode of PLAXIS 2D, where the response of the embedded beams was compared with the real pile behaviour.

The results (illustrated on Figure 7) show that the loads, for the failure criterion, estimated using the 2D axisymmetric model match the maximum service load calculated using empirical methods (Bustamante and Gianeselli, 1993, 1998).

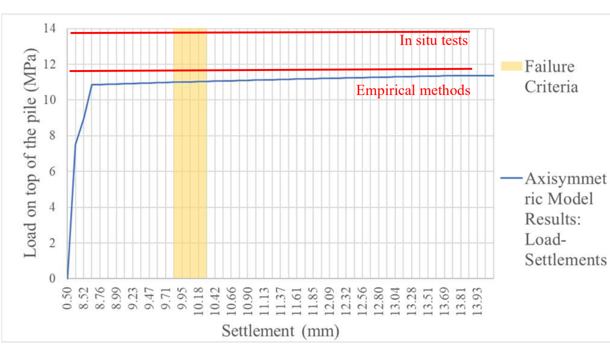

Figure 7. Load-Settlement obtained in the 2D model.

An ultimate load of 990kN was obtained for the failure criteria, for the precast driven piles.

These results were lower than the values obtained in the PDA tests. This shows that the empirical methods used, that are based on a collection of data from numerous field trials, take on assumptions so that the calculated values (using the methods) for the bearing capacity are, in most cases, conservative in relation to their actual capacity.

7 FINAL DESIGN

For the safety verification against relevant limit states, partial safety factors relative to both the actions and the materials, were adopted according to the European standard regulations.

7.1 Ultimate limit states

The final design of the foundation slab and the deep foundation elements was made using the results of the three-dimensional models, duly calibrated.

A load envelope was considered for each structural element, based on the most relevant combinations.

It should be noted that the lots in question are located in a seismic zone, and therefore there are, in all the buildings, sets of stairs and lift boxes and walls that have severe loads that need to be transmitted to the foundations.

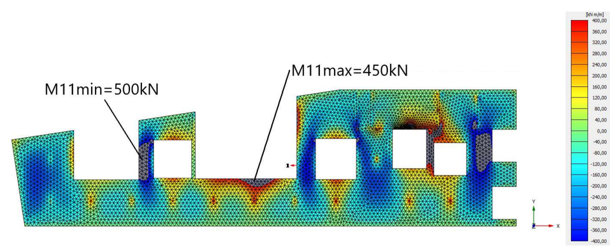

Figure 8. Bending moments M11 at the raft plate for a single combination in the lot 1 model.

Figure 8 shows the distribution of the bending moments in the lowest thickness slab. The distribution of loads on the slab leads to cylindrical bending between the spans without deep foundation elements. The areas over deep foundations will be stiffer, and therefore behave as supports for the hydrostatic uplift pressures.

Both solutions were designed against punching. It's crucial to thoroughly assess the hydrostatic uplift pressures exerted on the bottom of the foundations in these types of solutions, especially considering that the column loading acts in a different direction.

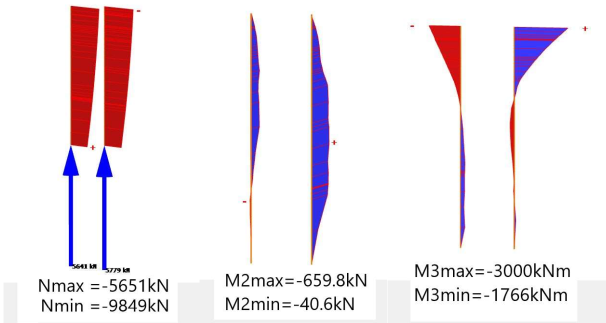

Figure 9. Forces in a pair of barrettes, modelled as embedded beams, for a single combination at lot 2.

All the precast driven piles and the barrettes were checked for combined axial force and biaxial bending, adopting the results from the model (Figure 9).

7.2 *Service limit states*

7.2.1 *Verification of the maximum settlements*

The maximum settlement for each model was accessed and checked, for the relevant combinations. The maximum values of 6.7 mm, 20.3 mm and 16.3 mm were obtained, respectively, for lots 1, 2 and 3.

7.2.2 *Verification of differential settlements*

To check the differential settlements, the maximum rotations over two points in the foundations system were estimated for each model.

Equation (1) shows the formula used to check the rotations at lot 1.

$$\tan(\theta_{rel,max}) = \tan\left(\frac{(\delta_{max}-\delta_{min})\times10^{-3}}{distance}\right) = \tan\left(\frac{(6.7-2)\times10^{-3}}{15}\right) = 5.5\times10^{-5} \leq \frac{1}{2000} = 5\times10^{-4} \quad (1)$$

where $\theta_{rel,max}$ is the rotation (angle) between two points with the settlements δ_{max} and δ_{min}.

7.2.3 *Verification of the global settlements near the façade, due to architectural limitations.*

Due to architectural constraints, it was necessary to verify the differential settlements at the lots peripherical zones. The models provided the ability to quickly access this information, as shown in Figure 10, for lot 2.

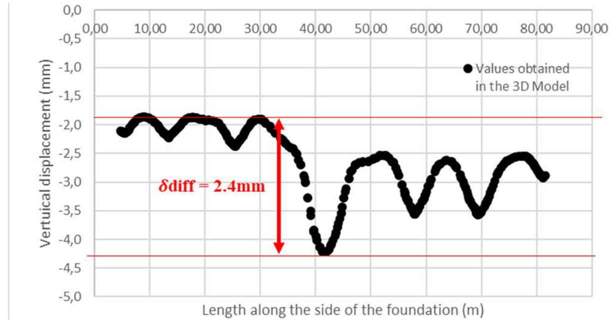

Figure 10. Long-term total settlements from two corners at the lot 2 model.

8 CONCLUSIONS

Higher processing power and access to smaller and greater data storage units are allowing more complex models to be quickly assembled. The described type of model can now be built, calibrated, and ran within most foundation projects time frame and enables the study of crucial issues in this type of project, such as soil-structure interaction, soil permeability, different settlements caused by different foundation stiffness zones, geological and geotechnical changes throughout the area site and much more, to be simulated effectively, henceforth allowing for optimized foundation solutions.

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Use of BIM methodology in Geotechnical Projects – Construction of underground reservoirs

L'utilisation de la méthodologie BIM dans les projets géotechniques - Construction de réservoirs souterrains

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ABSTRACT: Building Information Modelling (BIM) is a digital technology-based methodology that is widely used in the Architecture, Engineering, and Construction (AEC) industry. The application of BIM to geotechnical projects can provide significant benefits. This paper aims to present the use of this methodology in the geotechnical and structural design of two reservoirs inserted in Belo Horizonte's Flood Control System. These reservoirs are comprised of three 40 m diameter secant circular shafts around 35 m deep, in an area of approximately 3500 m^2. The geotechnical and structural design was conducted in a PLAXIS 3D model generated by importing an IFC with geological surfaces defined in GEO5 with the interpolation of boreholes results. Also, a geometrical model was developed in REVIT which allowed a better visualization and interpretation of the construction stage planning, interaction and clashing between different structures before construction and accurate quantities estimation of all elements. All steel rebars were modelled and detailed using REVIT or TEKLA, which improved the accuracy of rebar placement and design, and reduced error and omissions during the construction. This case study allowed us to conclude that the integration of specialized software in a unique collaborative environment can have several advantages leading to a better project and construction outcome.

RÉSUMÉ: La Modélisation des Informations du Bâtiment (BIM) est une méthodologie basée sur la technologie numérique largement utilisée dans l'industrie de l'Architecture, de l'Ingénierie et de la Construction (AEC). L'application du BIM aux projets géotechniques peut offrir des avantages significatifs. Cet article vise à présenter l'utilisation de cette méthodologie dans la conception géotechnique et structurelle de deux réservoirs. Ces réservoirs sont composés de trois puits circulaires sécants d'un diamètre de 40 mètres, d'une profondeur d'environ 35 mètres, sur une superficie d'environ 3500 mètres carrés. La conception géotechnique et structurelle a été réalisée dans un modèle PLAXIS 3D généré en important un fichier IFC avec des surfaces géologiques définies dans GEO5 grâce à l'interpolation des résultats de sondages. De plus, un modèle géométrique a été développé dans REVIT, permettant une meilleure visualisation et interprétation de la planification des étapes de construction, des interactions et des conflits entre différentes structures avant la construction. Toutes les armatures en acier ont été modélisées et détaillées à l'aide de REVIT ou TEKLA, améliorant la précision du placement et réduisant les erreurs et les omissions pendant la construction. Cette étude de cas nous a permis de conclure que l'intégration de logiciels spécialisés dans un environnement collaboratif unique peut présenter plusieurs avantages pour le projet et la construction.

Keywords: BIM methodology, 3d analysis, deep excavations, diaphragm walls

1 INTRODUCTION

In this article, the application of the BIM methodology for the execution and construction of the "Nado 1" and "Vilarinho 2" reservoirs is presented. This project is part of the restructuring of the drainage system of the city of Belo Horizonte, state of Minas Gerais, Brazil (Figure 1).

The city has approximately 90 risk areas, classified according to the number of points per municipality, with the Venda Nova region, where both reservoirs are located, being the region with the highest incidence of flood records, with approximately 20 classified points (Cistini, 2018).

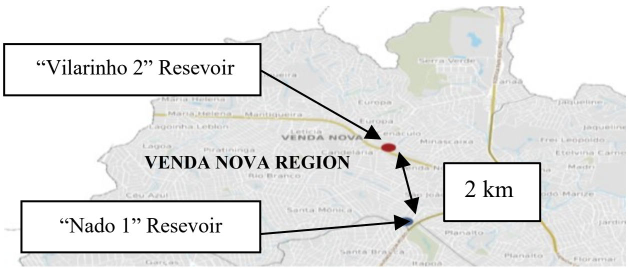

Figure 1 – Site identification.

The construction of two large reservoirs with approximately 3500 m^2 and a considerable depth of about 35 m), situated in a densely populated region of the city of Belo Horizonte, results in a complex process marked by complex soil-structure interaction, influenced by various factors such as structural symmetry, geological-geotechnical asymmetry, flow analysis, stability of the excavation bottom (hydraulic failure) through permeable formations, construction phasing, and demanding control of settlements in the surrounding space. In this context, three-dimensional analyses were conducted to obtain results with greater reliability, capturing the soil-structure interaction appropriately. These analyses were made through the use of the BIM methodology by the creation and integration of geotechnical, numerical, and geometrical models of the reservoir's elements and structures in the Vilarinho area (Figure 2) and Nado area (Figure 3).

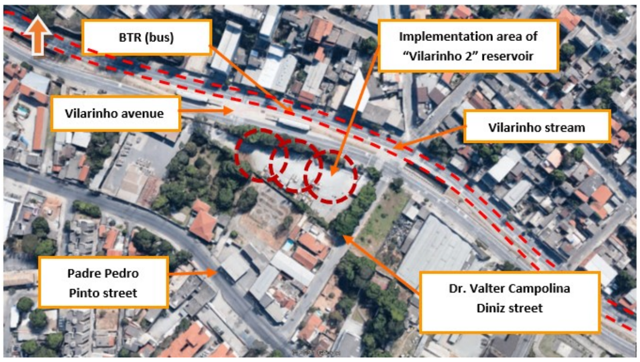

Figure 2 – Vilarinho reservoir.

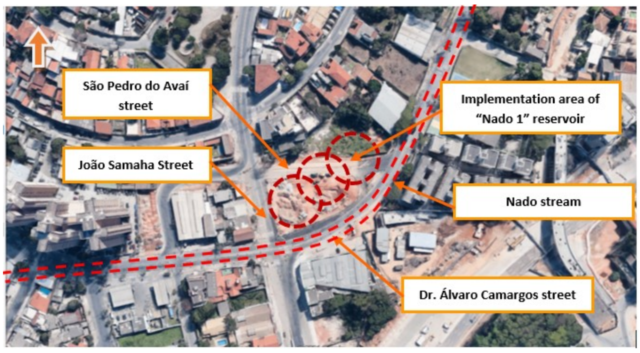

Figure 3 – Nado reservoir.

2 STRUCTURAL SOLUTION

The main concept for the project aimed at controlling deformations in the influence area of the excavation, with the goal of minimizing its impact on existing infrastructures as well as neighboring buildings. Given the substantial depth of the planned excavation and the nature of the geological formations encountered, a solution in diaphragm walls was chosen, materialized by three intersecting circular shafts. At the intersection points, the execution of two reinforced concrete frames is designed, materialized by a set of columns and struts braced at various levels to reduce their respective bending lengths.

To ensure the proper functioning of the retaining structure when existing possible deviations from the verticality of the diaphragm wall panels, various bracing rings were designed along the perimeter of the wall. These rings are responsible for transmitting axial loads to the different levels of struts of the frames (Figure 4).

The regulatory soil pressures and overloads acting on the face of the diaphragm walls are, in turn, transmitted by bending/shear to the rings and, equally, by axial forces from the rings to the two reinforced concrete frames.

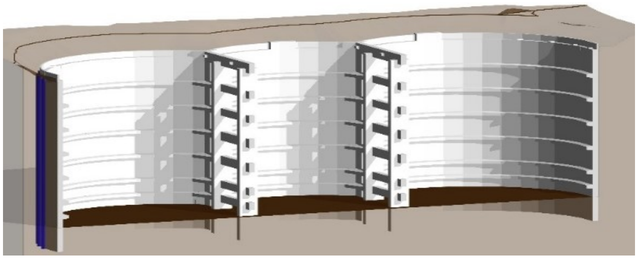

Figure 4 – Excavation structure of the reservoirs.

The structure of the reservoir cover essentially consists of pre-slabs supported by prefabricated beams, which in turn are supported on reinforced concrete columns with a height equal to that of the reservoir (Figure 5).

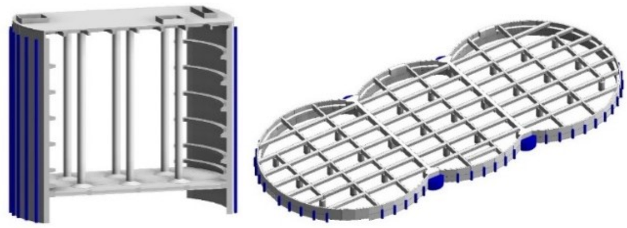

Figure 5 – Cover structure of the reservoirs.

3 USE OF BIM TOOLS

3.1 Introduction

Considering the development and importance of using BIM tools in civil construction, especially in geotechnical projects (Vaníček et al, 2021), the decision was made to carry out this project in a collaborative environment, integrating and coordinating the geometry of the reservoir structure and associated structures with its reinforcement, and ensuring compatibility between the structure and the surrounding geological-geotechnical environment. The development of the project through this methodology also allowed for the identification of various incompatibilities among the different constituent structures of the reservoir, which were timely corrected in the design phase, prior to construction phase.

3.2 Geotechnical modeling

In order to identify the geological formations to be intersected by the excavation, geologic-geotechnical campaigns were conducted at both locations. These campaigns included mechanical drilling, geophysical surveying and in-situ and laboratory tests.

It is observed that the local geological environment generally involves a superficial layer of modern anthropogenic, alluvial, and colluvial materials, referred to as "fill deposits" and "alluvial/colluvial deposits," respectively. Underlying these formations there is a substratum dating from the Archean, representing the lithostratigraphic unit designated as the "Belo Horizonte Complex", composed superficially of residual soils/saprolites covering a gneiss rock mass with a degree of weathering ranging from sound to highly weathered.

In order to make a more accurate assessment of the materials intersected by the diaphragm walls (Tawelian et al, 2016), as well as to determine the various excavation volumes of these materials and the position of the existing water table, a three-dimensional modeling of the geological structure was carried out based on the information provided by the extensive geologic-geotechnical plan.

This model was developed using the Stratigraphy tool in the GEO5 software, which allows, through the input of various point field test data (drillings, wells, CPT, DPT, SPT, DMT, and PMT) and topographic surveying, to interpret and generate a geological model consisting of a set of transition surfaces between the various formations that constitute the terrain in the reservoir areas (Figure 6). The creation of this model was done iteratively, through the interpretation and analysis of the model until it aligned with the geological-geotechnical study results.

All this information was input into the program through georeferencing points with X, Y, and Z coordinates, allowing for subsequent export to the numerical and geometrical modeling program in the universal IFC format.

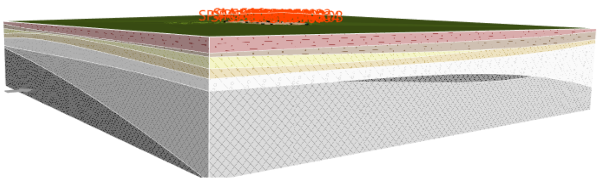

Figure 6 – Stratigraphy generated by GEO5 after inputting geotechnical test data.

3.3 Numerical modeling

In an effort to assess the impact of geological-geotechnical asymmetry, structural asymmetry and the presence of neighboring structures and infrastructure, and thereby estimate the forces and deformations on the structure, three-dimensional finite element analyses were conducted using the PLAXIS 3D software, simulating the key construction phases (Figure 7).

The geometry of the numerical calculation models, particularly concerning the spatial definition of the geological structure, was based on the three-dimensional geological models presented in the previous chapter.

To achieve this, aiming to generate suitable finite element meshes, geotechnical boundary points (generated in the GEO 5 model) were first imported into AutoCAD 3D. Subsequently, Non-Uniform Rational Basis Spline (NURBS) surfaces were created at the transition between the various formations constituting the terrain in the reservoir areas. Finally, these surfaces were imported into the PLAXIS 3D numerical calculation program. This entire process was carried out iteratively until a finite element mesh with sufficient quality for the analysis of all structural elements and neighboring infrastructure was generated.

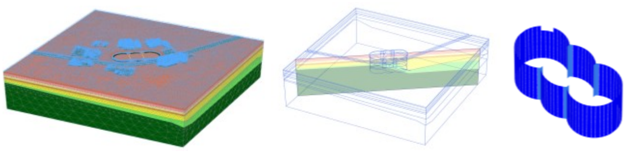

Figure 7 – Geometry and finite element mesh of the three-dimensional calculation model of a reservoir and its structural elements (PLAXIS 3D).

3.4 Geometric modeling

Based on the structural preliminary design drawings created in AutoCAD, the decision was made to do the geometric modeling of the reservoir structure and adjacent structures in REVIT.

To make the project more comprehensive, enhance the understanding of all constituent elements of the models, and facilitate the extraction of material quantities from various components of the reservoir, including formwork, concrete, and reinforcements, various construction phases were created to simulate the reservoir's construction process (Figure 8)

It is also noteworthy that creating the model using these phases was crucial for all stakeholders involved in the construction of the reservoirs (designers, builders, inspectors, and the owner) to have a clear understanding of the planning and execution of the project.

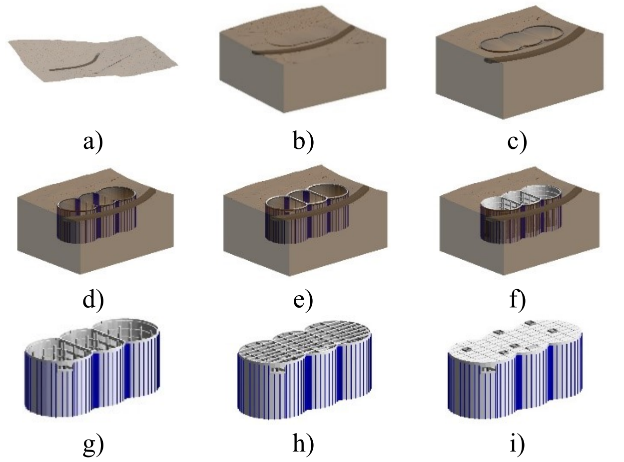

Figure 8 – Phases for the execution of the structure: a) existing; b) excavation; c) guide wall; d) panels and jet grouting; e) first beam; f) rings and struts; g) structural columns; h) cover beams; i) roof slabs.

3.5 Rebar modeling

The reinforcement rebars of the elements of the reservoir were modeled in REVIT and TEKLA software to ensure that, during the construction phase, there were no doubts or errors in the assembly of the reinforcements.

Modeling of simpler and longer-developed elements such as the panels of the diaphragm wall, was carried out in REVIT (Figure 9), while modeling of more complex elements, such as beams and slabs of the cover, was performed in TEKLA (Figure 10).

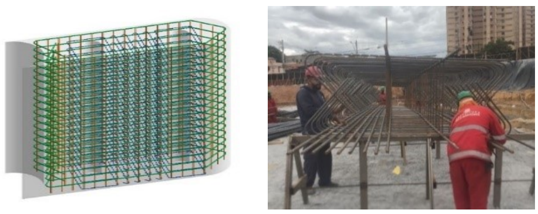

Figure 9 – Modeling the reinforcements' panels and assembling them on-site.

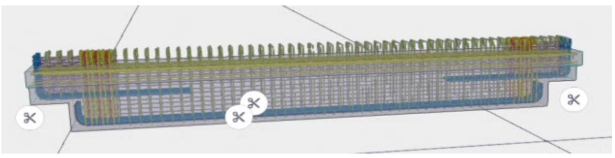

Figure 10 – Modeling the reinforcements of the cover beams.

Despite being time-consuming, this modeling approach offers numerous advantages for geotechnical projects, particularly for designers and builders, including th reduction of steel wastage on the construction site, reduction of time spent on presenting and measuring reinforcements compared to traditional CAD methods, correction of errors and incompatibilities between reinforcements in a preliminary phase and evaluation of all necessary details for proper execution.

4 FINAL REMARKS

The present article addressed the use of the BIM methodology for the design and construction of two underground reservoirs with approximately 35 m of depth, integrated into the flood control system of the city of Belo Horizonte, Brazil.

The geological-geotechnical models, along with their iterative integration with numerical and geometrical models allowed for a better visualization and more comprehensive analysis of geotechnical data. This led to a correct simulation of the soil-structure interaction and, consequently, more realistic and optimized solutions.

The application of the BIM methodology in this geotechnical project facilitated coordination among different disciplines and structures involved. It has provided a better and more effective interpretation of different construction phases from the early design stage, avoiding coordination errors during the execution phase. Thus, this methodology represents a significant evolution in the approach to geotechnical projects and in their execution in a more efficient way.

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XVIII EUROPEAN CONFERENCE
ON SOIL MECHANICS AND
GEOTECHNICAL ENGINEERING

Deep excavation solutions for the construction of a logistic park in Loures (Lisbon)

Solutions de excavation profonde pour la construction d'un parc logistique à Loures (Lisbonne)

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ABSTRACT: This paper aims to describe the deep excavation solutions adopted for the construction of a logistics park with an implantation area of approximately 107 000 m2, located in the municipality of Loures in Lisbon. The use of this site for the construction of a large-scale building with demanding future performance criteria proved to be extremely challenging, particularly because the original place had previously been used as a limestone quarry and subsequently as a landfill for various materials associated with the construction of the nearest main road network. Given the existing constraints, particularly the desired implantation area, the geological-geotechnical study and the neighboring conditions, a deep excavation solution was defined for a maximum height of 20 m, consisting of a bored pile wall with 800 mm diameter and with shotcrete between the piles, braced by permanent anchors and complemented with a concrete Berlin-type wall solutions in areas where the limestone rock, with low degrees of alteration and fracturing, emerged above the excavation bottom. In addition to the adopted design criteria, the main results of the proposed instrumentation and monitoring plan for the retaining solution and neighboring infrastructure are also presented, and a comparative analysis is performed with the calculation models.

RÉSUMÉ: Cet article vise à décrire les solutions de excavation profonde adoptées pour la construction d'un parc logistique d'une superficie d'environ 107 000 m2, situé dans la municipalité de Loures à Lisbonne. L'utilisation de ce site pour la construction d'un bâtiment à grande échelle répondant à des critères de performance futurs exigeants s'est avérée extrêmement difficile, en particulier parce que l'endroit d'origine avait été précédemment utilisé comme carrière de calcaire, puis comme décharge pour divers matériaux associés à la construction du réseau routier principal le plus proche. Compte tenu des contraintes existantes, en particulier de la superficie d'implantation souhaitée, de l'étude géologique-géotechnique et des conditions environnantes, une solution de excavation profonde a été définie pour une hauteur maximale de 20 m, comprenant un mur de pieux forés de 600 mm de diamètre avec du béton projeté entre les pieux, renforcé par des ancres permanentes et complété par des solutions de mur "Berlin" dans les zones où la roche calcaire, avec des degrés d'altération et de fracturation faibles, émergeait au-dessus du fond de excavation. En plus des critères de conception adoptés, les principaux résultats du plan d'instrumentation et de surveillance proposé pour la solution de soutènement et les infrastructures avoisinantes sont également présentés, et une analyse comparative est réalisée avec les modèles de calcul.

Keywords: Deep excavation; Bored pile wall; concrete Berlin-type wall

1 INTRODUCTION

The deep excavation solutions presented in the present article are located in the municipality of Loures (Figure 1) and were developed and designed to enable the construction of a logistic park with an approximately 55,000 m2 footprint.

Figure 1 – Site identification (Google Earth image).

The use of this site for the implementation of a large-scale building with demanding future performance criteria proved to be extremely challenging due to the fact that the original land had previously been utilized as a limestone quarry and subsequently as a dumping ground for various materials. These two situations determined, during excavation, the occurrence of very abrupt geological transitions and completely distinct geotechnical behaviors over very short distances.

After an extensive geological-geotechnical characterization, substantiated by various prospecting campaigns, the best deep excavation and earth support solutions were adapted for both excavation and embankment areas; the latter, however, are not covered in the present article.

For the excavation areas located in the southern elevation, adjacent to the national road EN250, a bored pile wall was adopted, with a maximum height of 20 m, permanently anchored and complemented with a concrete Berlin-type wall in areas where the limestone mass, with low degrees of alteration and fracturing, emerged above the excavation bottom.

On the other side, for the embankment areas, on the western, northern and eastern elevations, considering the existing levels, available space, and the need to ensure the maximum possible storage capacity for surplus soils from the embankment platform, a gabion wall and a reinforced earth wall with facing panels of precast reinforced concrete were constructed.

2 MAIN EXISTING CONDITIONS

2.1 Geological-geotechnical conditions

Four prospecting campaigns were conducted in the area of the logistics park from 2019 to 2021, that involved the execution of around fifty hollow stem auger borings, accompanied by dynamic penetration tests of the SPT type at every 1.5 m, reaching depths of 34 m, twenty-four exploratory pits, using an excavator, with depths ranging from 0.70 to 4.90 m and ten seismic refraction profiles.

The borings from the second and fourth phases were considered particularly representative for the design of the bored pile wall solution due to their location, almost coinciding with the boundary perimeter of the existing quarry. The analysis of these campaigns confirmed the variability of encountered terrains and abrupt geological variations (Figure 2).

During the excavation process, it was also observed that within a few meters, there was a transition from a rocky mass near the surface to a rocky mass below the excavation bottom (Figure 3).

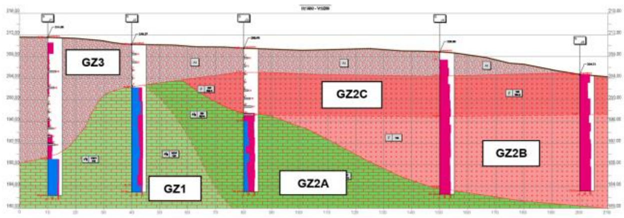

Figure 2 - Geotechnical interpretative profile along the alignment of the bored pile wall.


Figure 3 - View of abrupt transitions between landfill zones and vertical faces of the limestone quarry.

The analysis of the geological-geotechnical campaign allowed the individualization of the 5 geotechnical zones identified below:

GZ3 **-** Corresponds to landfill deposits with soils of poor geotechnical characteristics;

GZ2C - Soils with low geotechnical suitability, corresponding to volcanic breccias and decomposed volcano-sedimentary formations from the Lisbon Volcanic Complex;

GZ2B - Soils with moderately high geotechnical suitability. Correspond to volcanic breccias and decomposed volcano-sedimentary formations from the Lisbon Volcanic Complex;

GZ2A - Massif with high geotechnical characteristics, corresponding to basalts, volcanic breccias, and decomposed volcano-sedimentary formations or rock ranging from highly altered to moderately altered from the Lisbon Volcanic Complex;

GZ1 - Massif with very high geotechnical characteristics, corresponding to whitish crystalline limestones to grayish marly limestones ranging from little to moderately altered.

2.2 Pre-existing conditions

The intervention area is located, in part, over former limestone quarries that were subsequently used as a landfill. In this context, there was a high level of uncertainty about the composition of the fill materials but also about the configuration/inclination of the cut slopes, which were created during the quarrying activities.

2.3 Topographic constrains

The land used for the construction of the logistics park had a slope towards the north, with the area adjacent to national road EN250 being the highest and the area adjacent to a watercourse on the opposite elevation having lower elevations. To achieve a level platform, as previously mentioned, it was necessary to develop a retaining structure through excavation along the northern elevation and part of the western elevation. Additionally, gabion retaining walls and reinforced earth walls were designed on the southern elevation, part of the western elevation, and part of the eastern elevation. As mentioned above, this article will exclusively address the deep excavation solutions in the northern and western elevations.

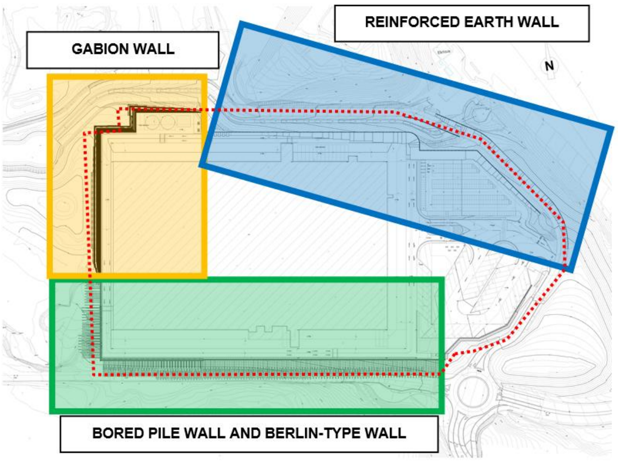

Figure 4 - Identification of the retaining and embankment solutions and lot boundary (highlighted in red).

3 PROPOSED SOLUTIONS

3.1 Bored pile wall

Based on the existing constraints, particularly topographical, geological-geotechnical, neighboring occupancy, and the need for rapid execution, it was adopted a bored pile wall, with a 800 mm diameter and spaced at a center-to-center distance of 1.5 m (Figure 5). This arrangement enabled an excavation of about 20 m in maximum height.

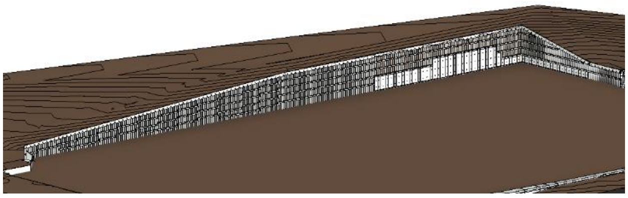

Figure 5 – 3D perspective oof the bored pile wall solution.

Given the geological conditions of the site and the length of the piles, they were entirely executed using a telescopic Kelly bar, occasionally with temporary casing at the borehole entrance. When encountering larger blocks or to ensure anchoring in the rock mass, a tricone bit, rock auger, and coring tool were used.

To enhance efficiency in excavation, the wall was braced using 2 to 5 levels of permanent anchors with planar spacings of 3.0 m, aiming to provide temporary and definitive horizontal balance of the containment against ground impulses, both in static and seismic conditions.

To ensure a more even distribution of forces on the wall and prevent excessive load concentration, reinforced concrete distribution beams were implemented and properly anchored to the piles ().

The anchors were installed at different inclinations to the horizontal (between 15º and 30º) and with variable free lengths (between 9 m and 17 m), with a minimum sealing length of 8 m (Bustamante et al., 1985) and a maximum long-term traction of 900 kN.

Considering the permanent nature of the anchors, the provisions of NP EN 1537 were followed regarding the characteristics of the tendons, injection grout, anchor head protection components, and anti-corrosion protection.

The exposed soil between piles was protected with a sprayed concrete coating applied in two layers and reinforced with metal fibers. The area was properly drained using geodrains between piles.

Figure 6 - Excavation works near the bored pile wall.

3.2 Concrete Berlin-type retaining wall

As indicated in the geological-geotechnical study, an intersection of the piles with slightly altered limestone occurred near the corner of the wall. In this area, the pile wall was underpinned using micro-piles. The remaining excavation was carried out with advances under a concrete Berlin-type retaining wall with rock bolts (Figure 7).

In this type of solution, the retaining wall is constructed in levels, from top to bottom. Each level is created by constructing reinforced concrete panels (first the primary panels and then the secondary ones), supported by spaced micro-piles, on average 3 m apart. These micro-piles are installed before the start of excavation work and function to support the vertical loads to which the retaining wall is subjected. The panels, with a theoretical minimum thickness of 30 cm, are cast against the open vertical face of the limestone mass and connected at the top through a reinforced concrete beam with pre-stressed anchors. This beam simultaneously ensures the underpinning of the base of the bored pile wall and the partial transfer of vertical loads to the micro-piles.

Regarding the horizontal bracing of this wall, the execution of high-strength steel tie rods with a diameter of 32 mm was anticipated.

Figure 7 - Excavation works near the concrete berlin-type retaining wall.

4 DESIGN METHODS

To analyze the behavior of the retaining wall solutions with respect to forces and deformations, finite element models were employed using the PLAXIS 2D software.

The analyses conducted generally involved studying various cross-sections deemed representative of the diverse geometries along the lengths of each of the construction solutions used.

The geomechanical behavior of soils was simulated using Hardening-Soil constitutive models, while for more rocky materials, their behavior was simulated using elasto-plastic Mohr-Coulomb constitutive models, and thus, without hardening.

Structural elements, such as the bored pile wall, concrete Berlin-type retaining wall, and anchors, were modeled based on their elastic properties. The modeling limited their maximum strength to ultimate limit states, allowing for the assessment of the maximum global safety factor.

5 MONITORING PLAN

The implemented monitoring plan aimed to ensure the safe and cost-effective execution of activities related to excavation process and during the lifespan of the logistics park. Additionally, it involved analyzing the surrounding behavior during this phase of construction, leading to adaptations based on the obtained results.

Installed devices allowed for the measurement of the following parameters on a weekly basis:

- Horizontal and vertical displacements of the retaining structures using topographic targets;
- Horizontal displacements of the soil behind the retaining solutions using inclinometers;
- Measurement of tension load in the anchors using load cells.

As of the current date, no significant deviations from the project estimates have been recorded based on the available readings. The maximum horizontal movements on the targets and inclinometers are approximately 15 mm, associated with a maximum excavation height of about 15 m and anchor load increments of less than 10%.

6 CONCLUSIONS

Considering the positive outcomes of the established instrumentation and observation plan, along with the accomplishments during the excavation works, it is concluded that the alternative solutions developed in the context of the design and construction process carried out in collaboration between the designer (JET_{SJ}) and the contractor (Norton EI), have proven to be extremely versatile and well-suited to local constraints.

ACKNOWLEDGEMENTS

The authors express their gratitude to the Client (LIDL) for granting permission for the drafting and publication of this article. It is also important to emphasize that the implemented solutions were the result of a team effort developed in collaboration with the contractor (Norton EI), subcontractors (Ancorpor, MaccaferriI, VSL, among others), and supervision (Duplano).

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Lisbon new circular Metro line: Buildings underpinning

Nouvelle ligne circulaire du métro de Lisbonne: Reprise en sous-œuvre des bâtiments

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ABSTRACT: The new Lisbon circular metro line will crosses a densely urbanized part of the city, connecting Rato Station located at one of the hills of the city and Cais do Sodré Station, at the Tagus River right bank. The underground excavation intersects a wide range of materials, from rock mass to soft soils. Where the construction of the tunnel section is closer to the river, with about 10 m of cover, a Cut&Cover method is used. In this metro the tunnel intersected a pile foundation of two reinforced concrete buildings with 9 upper floors and 1 basement, determining the need to underpin the structures and change permanently its foundation system. The geotechnical and geological conditions present in this metro, associated to highly limited access and working conditions, led to the execution of the retaining walls using jet-grouting technology. Those elements were also used as the building deep foundations, which consists of a reinforced concrete slab (length=50m, width=13m and thicknesses=1.4m and 1.8m), being also responsible for the structure underpinning. In this complex process is defined a controlled load transfer between the structure and the new slab, which was executed using hydraulic jacks, limiting the building differential settlements thought gradual jacks opening and according with monitoring. This paper presents an overall description of the solutions, how they were implemented and the buildings' behaviour during the underground works.

RÉSUMÉ: La nouvelle ligne de métro circulaire de Lisbonne traversera une zone densément urbanisée de la ville, reliant la Station Rato, située sur l'une des collines de la ville, et la Station Cais do Sodré, sur la rive droite du Tage. L'excavation souterraine couvre une large gamme de matériaux, depuis les masses rocheuses jusqu'aux sols meubles. Lorsque la construction du tronçon de tunnel est plus proche de la rivière, avec une profondeur de couverture d'environ 10 m, la méthode de tranchée couverte est utilisée. Dans cette section, la ligne a traversé une fondation sur pieux de deux bâtiments en béton armé de 9 étages supérieurs, ce qui a déterminé la nécessité de soutenir la structure et de modifier de manière permanente son système de fondation. Les conditions géotechniques et géologiques présentes dans ce section, associées à des conditions d'accès et de travail très limitées, ont conduit à la matérialisation de murs de soutènement grâce à la technologie du jet-grouting. Ces éléments ont également servi de nouveau fondations profondes des bâtiments, constitué d'une dalle en béton armé (longueur = 50 m, largeur = 13 m et épaisseurs = 1,4 m et 1,8 m), qui est également chargée de soutenir la structure. Dans ce solution complexe, un transfert de charge contrôlé est défini entre la structure et la nouvelle dalle, qui a été réalisé à l'aide de vérins hydrauliques, limitant les tassements différentiels du bâtiment grâce à l'ouverture progressive des vérins et au contrôle concomitant. Cet article présente une description générale des solutions, de la manière dont elles ont été mises en œuvre et du comportement des bâtiments lors des travaux souterrains.

Keywords: Metro; underpinning; reinforced concrete building; jet-grouting; excavation cut&cover

1 INTRODUCTION

The new Lisbon circular metro line will cross a densely urbanized part of the city, connecting Rato Station, located at one of the hills of the city, and Cais do Sodré Station at the Tagus River right bank.

Where the construction of the tunnel section is closer to the river a Cut&Cover method is used. In this section the line path intersected a pile foundation of reinforced concrete buildings with 9 upper floors and 1 basement, determining the need to underpin these structures.

The geotechnical and geological conditions in this section, associated to highly limited access and working conditions, led to the execution of retaining walls using jet-grouting technology. Those elements were also used as deep foundations of an reinforced concrete slab with 1.4m and 1.8m tick, built for the structure underpinning.

In this complex process was defined a controlled load transfer between the structure and the new slab, which was executed using hydraulic jacks and specific monitoring to control the buildings differential settlements.

2 AFFECTED BUILDINGS

The metro line tunnel intersets buildings nº 42 and nº 44 at Avenida D. Carlos I. Both buildings were built in the XX century, presenting a reinforced concrete structure and pile foundations. The buildings have 9 elevated floors and 1 basement (see *Figure 1*).

Figure 1. Reinforced concrete buildings above de tunnel.

Building nº42 was recently subject to rehabilitation works for conversion into residential use. As a result of the changes needed, was designed a reinforcement of both the structure and foundations, the latter with micropiles. Building nº44, on the other hand, is in its original conditions, used for offices, and showing a good state of conservation. In *Figure 2* is presented the buildings plan superimposed with the tunnel structure layout, showing the need to underpin several columns over intervention area.

Figure 2. Buildings plan view and tunnel structure projection.

3 MAIN CONSTRAINS

3.1 Geological and geotechnical constrains

The geological investigation campaign included the execution of multiple boreholes that allowed the characterization of the ground units along the extension of the Cut&Cover trench over which the buildings to be underpinned are located (see *Figure 3*). It was possible to confirm that in the increasing direction of the mileage of the trench, there is a progressive increase in the thickness of recent materials (landfill and alluvium - essentially sandy type) in parallel with the decrease of the Miocene layer depth overlying the units of the Lisbon Volcanic Complex.

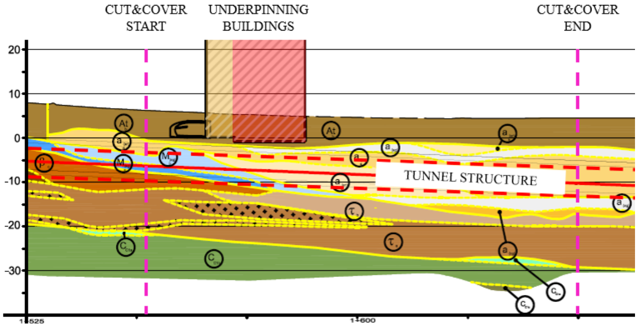

Figure 3. Geological and geotechnical scenario.

3.2 Constructive technologies constrains

The solutions had to respect the local constraints regarding the accessibility of equipment. Considering the need to work inside the building's basements, the solutions had to be compatible with equipment that can operate with a minimal ceiling height of about 3,0m.

3.3 Load transfer procedure

To minimize the occurrence of pathologies in the underpinning buildings, due to differential settlements, the solutions and construction phases had to be compatible with a procedure of controlled load transfer from the buildings to the new foundation system, which was carried out using hydraulic jacks accompanied by monitoring equipment to interpret the behaviour of the buildings and new foundation system.

4 SOLUTION DESCRIPTION

The solution consists of an underpinning system of the buildings columns and pile caps located above the tunnel alignment, which will allow the deactivation of the existing piles and the maintenance of the building's structure functionality and, simultaneously the construction and operation of the new metro tunnel.

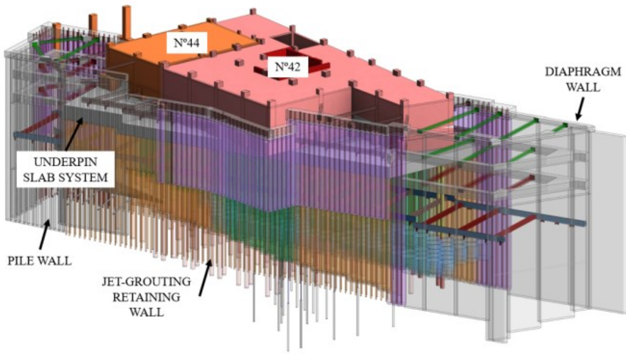

Figure 4. 3D view: Cut&Cover excavation and underpinning solution.

Once the underpinning system had been placed, which is composed of a reinforced concrete slab laterally supported by a double jet-grouting columns curtain wall, the excavation works done inside this curtain and below of the underpinned slab.

4.1 Cut&Cover excavation solution

Since the underpinning solution is connected with the Cut&Cover solution, a brief description is given of, although this is not the focus of this paper.

The Cut&Cover solution was generally composed of a double jet-grouting columns retaining wall with 1000mm diameter columns, spaced 700mm apart, and reinforced with tubular steel profiles, which have a dual function as a foundation of the underpinned slab, as well as retaining the soil and water to allow the excavation required to build the tunnel. In addition to this, and to reach the underpinned slab level, it was executed a third row of jet-grouting columns, also reinforced with steel profiles (*Figure 5*).

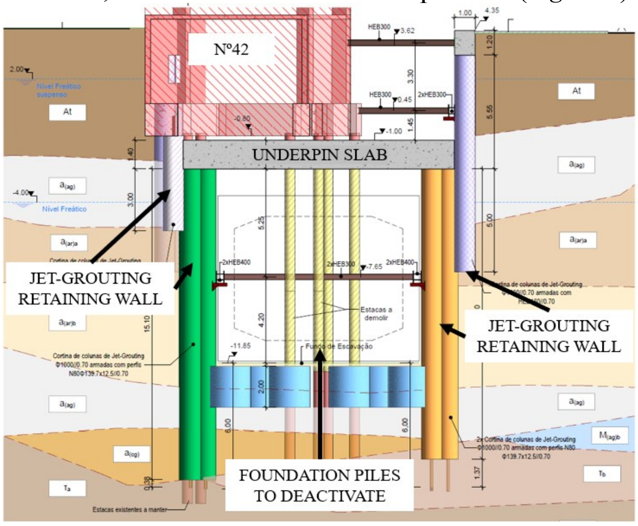

Figure 5. Section: Cut&Cover excavation and underpinning solution.

Finally, the retaining walls were temporary propped each other using four levels of steel struts, 5m apart, resting over steel and reinforced concrete distribution beams. However, in the underpinning area, reinforced concrete slab was used as the third strut level.

4.2 Underpinning solution

The underpinning solution consists of a reinforced concrete slab, not prestressed, supported indirectly on jet-grouting columns, reinforced with steel profiles in order to increase their ductility and stiffness, placed outside the tunnel. The underpinned slab was positioned below the existing pile caps and above the tunnel alignment, with geometry of approximately 50,0m by 13,0m and a variable thickness between 1.40m and 1.80m (see *Figure 6*).

Taken together, these elements make it possible to change the foundation system of the buildings, transferring the loads from the structural columns to the slab which, basically by cylindrical bending, transmits them to the two rows of jet-grouting columns, which finally transmit the reactions to the competent ground layers located below the bottom slab of the tunnel.

This solution makes it possible to deactivate the piles foundation of the buildings existing over the tunnel area, as well as to proceed with the excavation works to build the new tunnel.

4.3 Load transfer procedure

Hydraulic jacks were used in order to ensure a smoother transfer of loads between the buildings columns and the underpinned reinforced concrete slab, and thus allow for greater control of the settlements occurring in this process. Hydraulic jacks positioned between the underpinned slab and the pile caps, will be responsible for relieving the loads of the foundation piles, using the reaction provided by the slab.

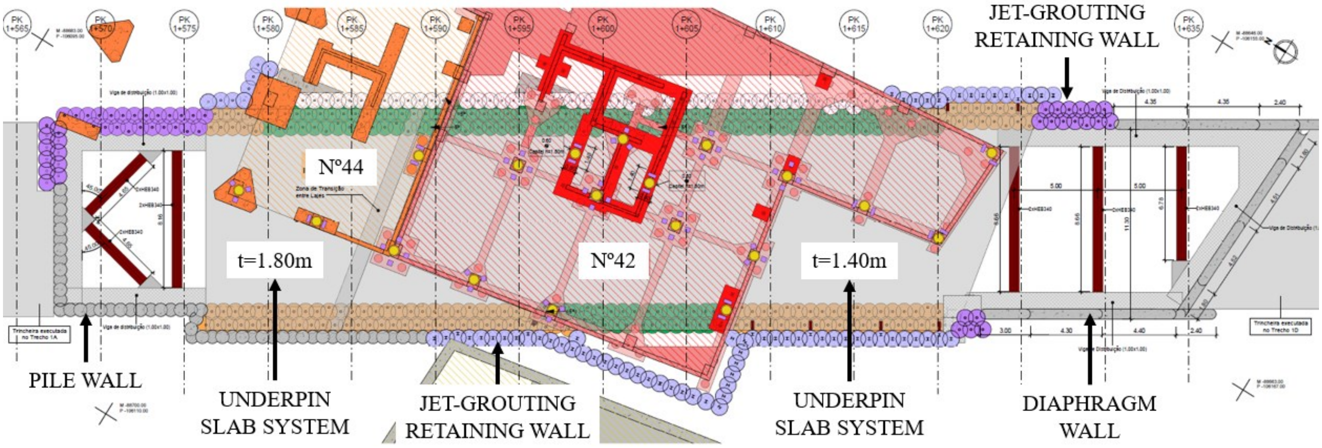

Figure 6. Plan view: Cut&Cover excavation and underpinning solution.

The phases related to the load transfer are summarized below and illustrated in *Figure 7*:

1. Installation of the instrumentation system at the columns;
2. Excavation to the bottom level of the slab;
3. Sheathing of the piles followed by slab construction and hydraulic jacks installation;
4. Excavation under the slab, followed by hydraulic jacks activation and subsequent existing piles demolition, managed by monitoring data analysis.

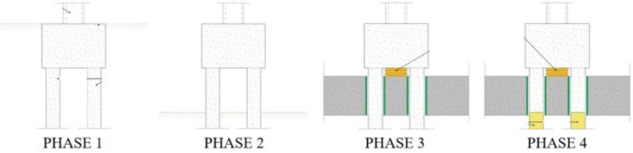

Figure 7. Underpinning solution – Load transfer beams.

5 SOLUTION DESIGN

The underpinning concrete slab was designed using structural models within the *SAP2000* software.

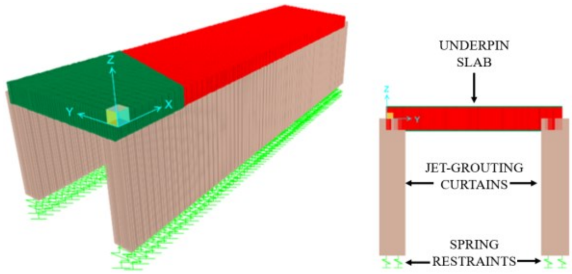

Figure 8. SAP2000 analysis model for underpin slab.

The underpinned slab, as well as the supporting jet-grouting columns, were modelled using shell type elements, and in the case of the latter, these were supported on springs type restraint. This model made it possible to estimate slab structural efforts (see, for example, bending moments in *Figure 9*) as well as its elastic deformation (see *Figure 10*).

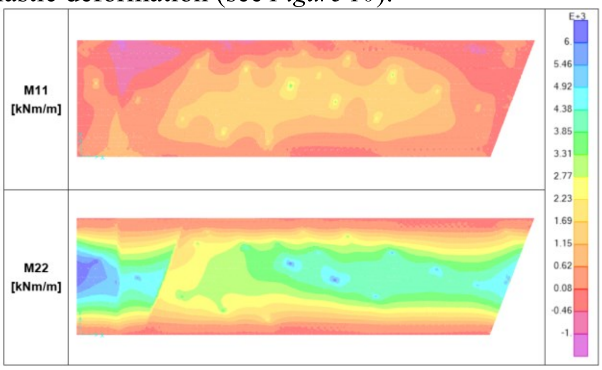

Figure 9. SAP2000 – Underpin slab bending moments.

Considering the support springs on the structural model, was estimated the maximum design axial load on each jet-grouting column, allowing, in parallel analysis with the geotechnical model for Cut&Cover solution, the safety validation of those elements.

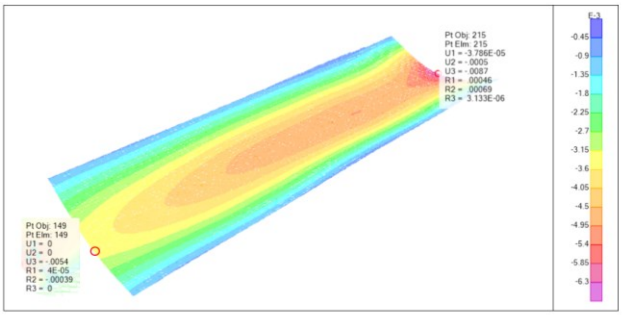

Figure 10. SAP2000 – Underpin slab displacements.

6 MONITORING PLAN

Complementarity, fort the Cut&Cover excavation, a monitoring plan using liquid level sensors and strain gages was installed to measure the settlement behaviour of the slab and columns, as well as the interaction between them (see Figure 11).

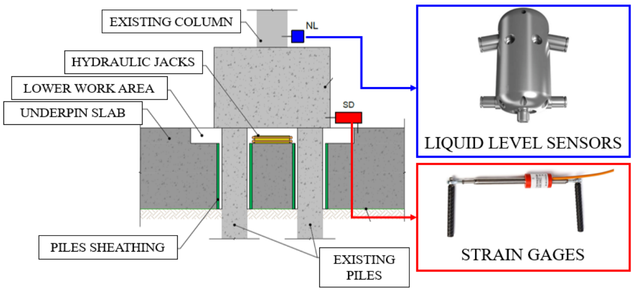

Figure 11. Monitoring devices: liquid level sensors and strain gages

The monitoring plan establishes threshold values for each device. The numerical analyses results were used to determine reference values for measured parameters that were used to set warning values, for 80% of the reference values, and alarm ones, for 120%.

7 FINAL REMARKS

The definition and implementation of a new metro line in a densely urbanized area may lead to interference with existing structures. For scenarios such as the one presented in this paper, an underpinning solution was the most appropriate way to maintain the integrity of the existing structures, although this type of solution often presents multiple restrictions regarding equipment access and operation.

An adequate monitoring and survey plan is essential to manage the load transfer to the underpinning system during the excavation works.

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Lisbon new circular Metro line: Interconnection section between new line and existing terminus

Nouvelle ligne circulaire du métro de Lisbonne: Section d'interconnexion entre la nouvelle ligne et le terminus existant

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ABSTRACT: The new Lisbon metro line will cross a densely urbanized part of the city, connecting Rato station located at one of the hills of the city and Cais do Sodré station at Tagus River right bank. The interconnection between the new line and the existing terminus is located close to the Tagus River bank where the landfill and clayed soft soils rasting over the Miocene bedrock have more than 20m thickness. Given this geotechnical scenario and the constrains related to the densely urbanized area, the cut and cover excavation solutions have designed to minimize the surrounding ground perturbance and surface settlements. Diaphragm walls braced by steel struts were used to hold a 20m depth excavation. The connection was materialized by the existing wall demolition and the simultaneous construction of a new structural system inside the terminus structure. This intervention included the construction of new reinforced concrete beams and columns with a meticulous construction phasing aligned with the demolition works using as well temporary support systems. The complexity of those works was increased by the constrains inside the terminus where the impact at Metro circulation should be minimized. This paper presents an overall description of the adopted solutions, how they were implemented as well as the monitoring plan that allowed the risk management of those complex works.

RÉSUMÉ: La nouvelle ligne de métro de Lisbonne traversera une partie densément urbanisée de la ville, reliant la station Rato située sur l'une des collines de la ville et la station Cais do Sodré sur la rive droite du Tage. L'interconnexion entre la nouvelle ligne et le terminus existant est située à proximité de la rive du Tage, où la décharge et les sols meubles argileux reposant sur le substratum rocheux du Miocène ont une épaisseur de plus de 20 mètres. Compte tenu de ce scénario géotechnique et des contraintes liées à la zone densément urbanisée, les solutions d'excavation en tranchée couverte ont été conçues pour minimiser les perturbations du sol environnant et les tassements de surface. Des parois diaphragmes contreventées par des étais en acier ont été utilisées pour maintenir une excavation de 20 m de profondeur. La connexion a été matérialisée par la démolition du mur existant et la construction simultanée d'un nouveau système structurel à l'intérieur de la structure du terminus. Cette intervention comprenait la construction de nouvelles poutres et colonnes en béton armé avec un phasage de construction méticuleux aligné sur les travaux de démolition en utilisant également des systèmes de support temporaires. La complexité de ces travaux a été accrue par les contraintes à l'intérieur du terminus où l'impact sur la circulation du métro devait être minimisé. Cet article présente une description générale des solutions adoptées, la manière dont elles ont été mises en œuvre ainsi que le plan de surveillance qui a permis la gestion des risques de ces travaux complexes.

Keywords: Metro, Terminus, Diaphragm Walls, Deep Excavation

1 INTRODUCTION

The new Lisbon circular metro line will cross a densely urbanized part of the city, connecting Rato Station, located at one of the hills of the city, and Cais do Sodré Station at the Tagus River right bank.
Where the construction of the tunnel section is closer to the river a Cut&Cover method is used.

In the Cais Sodré area, the new metro line intersects the terminus of the existing metro line, and this is one of the points of intersection between the existing and new construction of the new Lisbon metro circular line. This type of connection, associated with geological and geotechnical constraints, existing infrastructures, a dense urban environment, the need for demolition and new construction, required the construction of a diaphragm wall, new concrete portal elements, metal elements, micropile foundations and the use of hydraulic jacks and specific monitoring.

2 AFFECTED BUILDINGS

In the area intercepted by the connection of the new metro line to the existing terminus, the intervention essentially intersects one of Lisbon's busiest streets, Avenida 24 de Julho. Road and rail traffic will be affected, forcing the temporary and phased diversion of these infrastructures (see **Erro! A origem da referência não foi encontrada.**).

Inevitably, this intervention, as already mentioned, intersects with the structure at the terminus of the existing metro line. The structure of the existing terminus appeared to be in good condition, and parts of it could continue to be used in the new structural solution.

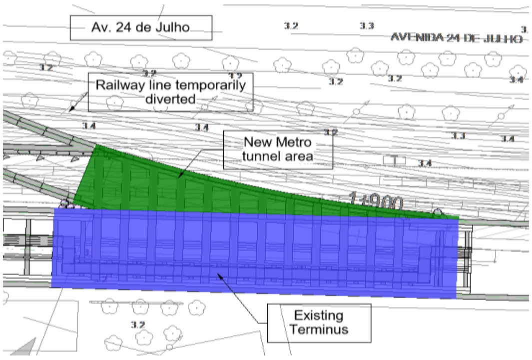

Figure 1. Plan: Layout of the new tunnel and the existing terminus.

3 MAIN CONSTRAINS

3.1 Geological and geotechnical constrains

The geological investigation campaign included the execution of multiple boreholes that allowed the characterization of the ground units along the extension of the Cut&Cover trench on which the intervention is based (see Figure 2). As the line progressed and the Tagus River approached, it was possible to confirm a progressive increase in the thickness of the materials of recent genesis (landfill and alluvium - essentially sandy type), reaching a thickness of around 17.0m, in parallel with the decrease of the Miocene layer depth overlying the units of the Lisbon Volcanic Complex.

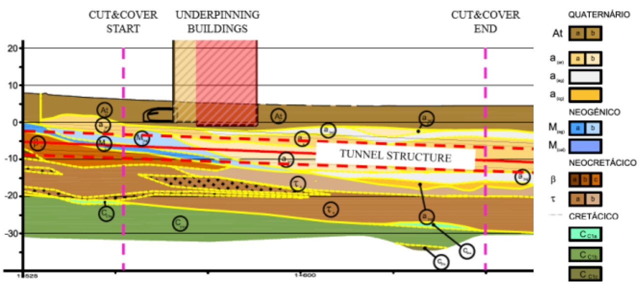

Figure 2. Longitudinal geological section

It is also worth mentioning that, given the proximity of the Tagus River, the groundwater level was considered to be 1,0m below the surface.

3.2 Constructive technologies constrains

The solutions had to respect the local constraints regarding the accessibility of equipment and material, guarantee the partial operational continuity of the terminus throughout the intervention as well as its opening hours and the deadline for completion of the construction work. It should also be mentioned that the intervention ensured that the embankment over the existing terminus cover slab remained in place throughout the intervention, so as not to further restrict transport infrastructure in the area. Considering the need to work inside the existing terminus, the solutions had to be compatible with equipment and material that could initially be transported in an opening with a width of 2.0m and solutions that could be carried out in a short period of time.

3.3 Demolitions and load transfer procedure

The solution had to guarantee the safe partial demolition of elements of the existing terminus, while preserving the existing conditions above the terminus' roof slab. In this way, it was necessary to implement reinforcement solutions that would allow the demolition of one of the side walls of the terminus, duly reconciled with the other constraints described. Since the embankment over the roof slab of the existing structure and its partial functioning were preserved, to minimize the occurrence of pathologies on the roof and surface, due to differential settlements, the solutions and construction phases had to be compatible with a procedure of controlled load transfer from the existing structural solution to the new structural solution, which was carried out using hydraulic jacks accompanied by monitoring equipment to interpret the behaviour of the systems.

4 SOLUTION DESCRIPTION

The solution adopted consists of executing a diaphragm wall to carry out the excavation in the Cut&Cover trench, provisionally restrained by horizontal metal struts. This excavation solution allowed the demolition work to begin and equipment and materials to enter through the side wall of the existing terminus. Inside the terminus for the demolition phase, porticoed structures made of metal elements were defined, using hydraulic jacks and prestressed bars for provisional stability.

For the final phase of the terminus structure, the work generally consisted of building a new porticoed reinforced concrete structure. Locally, metal columns were used to support the concrete beams and the existing foundations were reinforced with micro-piles.

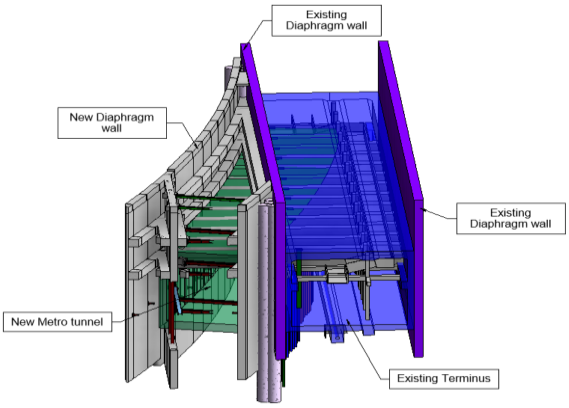

Figure 3. 3D View: Cut&Cover excavation and final structure of the terminus.

4.1 Cut&Cover excavation solution

The Cut&Cover solution was generally composed of an 800mm thick diaphragm wall that was horizontally braced by distribution beams and metal struts, materialised by HEB 400 to 600 profiles. It should also be noted that in this connection area, the existing diaphragm wall was used as a containment for excavation. This existing diaphragm wall had been used 25 years ago to build the structure of the terminus (see Figure 4).

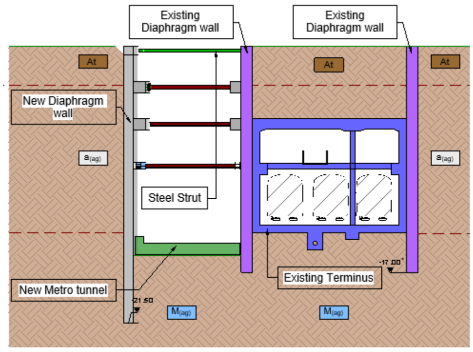

Figure 4. Section: Cut&Cover excavation solution.

4.2 Demolitions and Load transfer procedure

For the demolition phase, it was initially decided to make an opening in the side wall of the existing terminus to bring in the equipment and materials needed to carry out the temporary reinforcement. Initially, the connection between the side wall of the existing terminus and the existing diaphragm wall was guaranteed by means of prestressed bars, and then a metal portal with hydraulic jacks at the top was installed along the entire length of the north side wall of the terminus to be demolished (see Figure 5).

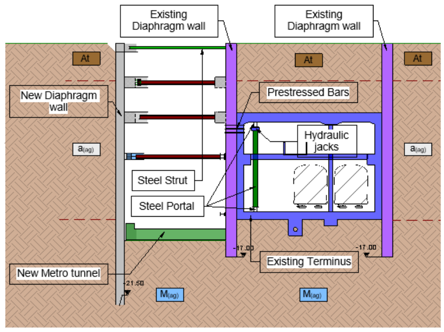

Figure 5. Section: Cut&Cover excavation, demolitions, and load transfer procedure solution.

After some secondary demolition, the existing diaphragm wall and the north side wall of the existing termination were demolished in 4.0 metre strips (see Figure 6).

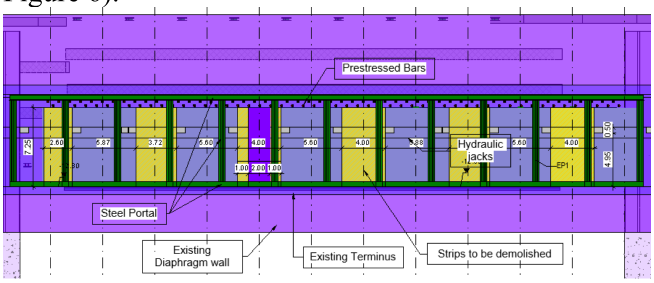

Figure 6. Elevated: Cut&Cover excavation, demolitions, and load transfer procedure solution.

The metal portal was materialised by HEB 500 vertical profiles and two HEB 500 horizontal beams.
Hydraulic jacks were used to ensure a smoother transfer of loads between the existing structure and the new structural solution.

4.3 Final structural solution for terminus

The final structural solution for the terminus consisted of making foundation micropiles on top of the existing floor slab to accommodate the localised load increase of the new solution. A wall beam (VPA) was built over these micropiles to distribute the loads from the new reinforced concrete columns (PB) (see Figure 7 and Figure 8).

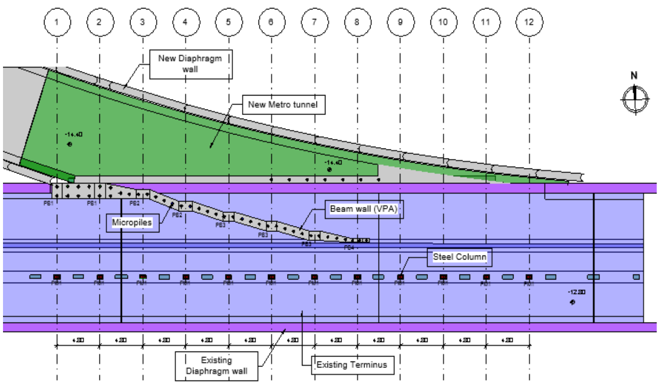

Figure 7. Plan: Foundation of the final solution to the terminus.

Beams (VB1) were built on top of the new columns, running between the south side wall of the existing terminus and the new north side wall of the new line's tunnel. Once the entire new structure was built, demolition of the remaining existing north side wall was completed along the length of the connection between the structures, in order to guarantee the track areas and widths defined in the project.
The structural system described has the capacity to replace the demolished north side wall of the terminus, leaving it with the capacity to support the roof slab and guarantee the correct connection between the new tunnel structure and the existing structure (see Figure 7 and Figure 8).

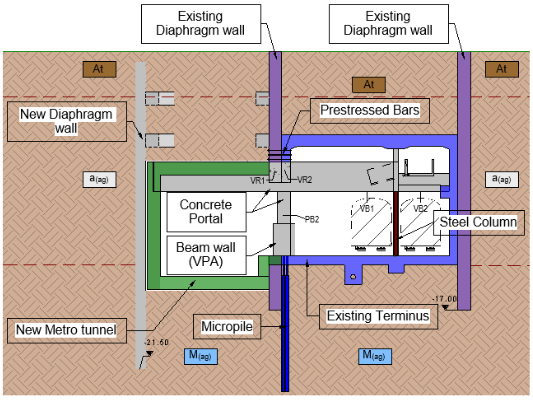

Figure 8 – Section: final structural solution for terminus.

5 SOLUTION DESIGN

The temporary diaphragm wall containment was dimensioned using a two-dimensional model in *Plaxis 2D* software. In this model, the diaphragm wall elements were modelled with plates, and the metal struts by anchor elements.
The final structure of the terminus was designed using a three-dimensional model in *SAP2000* software. All the walls and slabs were modelled using shell elements, and the beams and columns were modelled using frames. The structural model was vertically braced using compression springs to simulate the foundation soil of the floor slab, and horizontal compression springs to simulate the resistance of the ground (see Figure 9).

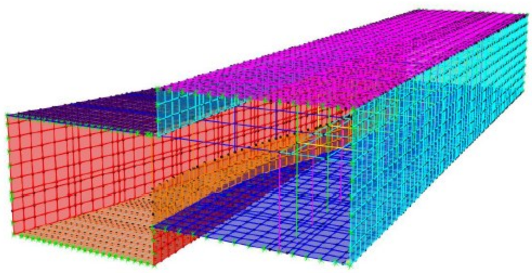
Figure 9. 3D View: Structural model developed in SAP 2000.

This model made it possible to estimate the design forces in the new and existing structural elements.
The model also made it possible to estimate the deformations in the various structural elements of the terminus, considering the new structural solution.

6 MONITORING PLAN

For monitoring the Cut&Cover excavation, traditional elements such as topographic targets, inclinometers and piezometers were used.
In addition to the instruments used in the Cut&Cover excavation, a monitoring plan was installed using liquid level sensors and strain gauges to measure the settlement behaviour of the existing terminus cover slab during the demolition phase.

The monitoring plan establishes alert and alarm values for the limits of each device. With the aim of guaranteeing safety and compliance with the values estimated in the project.

7 FINAL REMARKS

The definition and implementation of a new metro line in a densely urbanised area, on recent soil, with a high water level, with a circular shape and using the structures of the existing metro, leads to areas of connection between new and existing structures, requiring demolition and construction work. For scenarios such as the one presented in this work, the solution for carrying out the excavation using a diaphragm wall restrained by metal struts was the most suitable for maintaining the integrity of the existing structures and infrastructures. The remaining solution adopted for connecting the new structure to the existing one also proved to be adequate, guaranteeing all the regulatory safety criteria.
An adequate monitoring and survey plan was essential to confirm the suitability of the solution during excavation work, and also to manage the transfer of loads between the new structural system and the new structural system. the shoring system during excavation work.

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Lisbon new circular Metro underground line: Centenary buildings underpinning

Nouvelle ligne circulaire du métro de Lisbonne: Reprise en sous-œuvre des bâtiments centenaires

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ABSTRACT: The new Lisbon metro line will cross a densely urbanized part of the city, connecting Rato station located at one of the hills of the city and Cais do Sodré station at the Tagus River right bank. Thus, the underground excavation intersects a wide range of materials, from rock mass to soft soils. The new Santos Station will be located partially beneath XIX century buildings with a cover depth of about 15 m, being these structures highly sensitive to differential settlements given its masonry and timber composition with multiple structural pathologies. Given the building conditions, its heterogeneous soil foundation and the level of surface settlements induced by NATM excavation, an underpinning solution was needed aiming to mitigate the buildings potential damages. Hence, high length micropiles, concrete reinforced beams and walls were executed from the building ground floor aiming to transfer buildings loads to the soils located underneath the tunnel excavation. This paper presents an overall description of the adopted solutions, how they were implemented and the buildings' behaviour during the underground works.

RÉSUMÉ: La nouvelle ligne de métro de Lisbonne traversera une partie densément urbanisée de la cité, reliant la gare de Rato, située sur l'une des collines de la cité, et la gare de Cais do Sodré, sur le Tage fleuve. Ainsi, l'excavation souterraine recoupe une large gamme de matériaux, de la masse rocheuse aux sols meubles. La nouvelle station de Santos sera située en partie sous des bâtiments du XIXe siècle avec une profondeur de couverture d'environ 15 m, ces structures étant très sensibles aux tassements différentiels compte tenu de sa maçonnerie et du bois avec de multiples pathologies structurelles. Compte tenu des conditions du bâtiment, de l'hétérogénéité de la fondation du sol et du niveau de tassement de surface induit par l'excavation NATM, une solution de reprise en sous-œuvre était nécessaire afin d'atténuer les dommages potentiels du bâtiment. Par conséquent, des micropieux de grande longueur, des poutres et des murs renforcés en béton ont été exécutés à partir du rez-de-chaussée du bâtiment dans le but de transférer les charges des bâtiments vers les sols situés sous l'excavation du tunnel. Cet article présente une description globale des solutions retenues, de la manière dont elles ont été mises en œuvre et du comportement des bâtiments lors des travaux souterrains.

Keywords: Metro, Underpinning, Centenary Buildings, Excavation

1 INTRODUCTION

The new Lisbon circular metro line will include a new Station in Santos that will be partially located beneath some XIX century buildings with a cover depth of about 15 m, being these structures highly sensitive to differential settlements given its masonry and timber composition while presenting already multiple structural pathologies.

The underground excavation of this tunnel, using the technology known as New Austrian Tunnelling Method, was analysed with numerical models and surface settlements were estimated considering the geological scenario previously accessed. This information was used to evaluate the building vulnerability to those settlements also considering their structural condition and it was concluded the deficient structural conditions of those structures to support the estimated settlements without major damage. Given this scenario, a buildings underpinning solution was proposed mitigating the impact of the underground excavations on those centenary buildings.

2 AFFECTED BUILDINGS

Considering the vulnerability analysis conducted, two buildings with masonry walls and timber floors were classified with a moderated potential damage induced by the underground excavation. Those XIX century buildings have 3 and 5 upper floors and had a superficial foundation (see Figure 1).

Figure 1. Centenary buildings located above de tunnel.

The buildings are located on a hillside on the area where a monastery collapsed due to a strong magnitude earthquake that took place in 1755. Therefore, is it possible that the superficial foundation is located on top of debris fills which worsen its foundation conditions and can justify the differential settlements that were already visible.

Considering this scenario and the settlements estimation from the underground excavation numerical analysis, using Burland's (1997) classification it was concluded that those building, without reinforcement, would potentially suffer moderate to severe damage.

3 MAIN CONSTRAINS

3.1 Geological-geotechnical constrains

The geological investigation campaign included the execution of multiple boreholes that allowed the characterization of the soil units underneath the buildings (see Figure 2). It was possible to confirm that its superficial foundation was on top a 10m thickness Anthropocene fill layer. Bellow that layer, Miocene and Lisbon Vulcanic Complex materials were found showing satisfactory resistance and deformation characteristics.

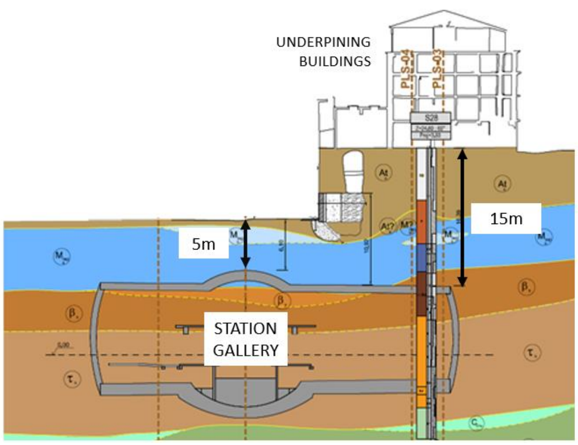

Figure 2. Geological scenario.

3.2 Constructive technologies constrains

The reinforcement solutions must respect the local constraints regarding the accessibility of equipment. Considering the need to work inside the buildings, the solutions must be compatible with equipment that can operate with a minimal ceiling height of about 2,5m.

3.3 Underground works constrains

Underground works consider the excavation using the NATM method which include the use of reinforcement elements such as umbrella soil nails and geodrains that go beyond the tunnel section. The position of those elements must be considered on the underpinning solution to avoid intersection of those elements.

4 UNDERPINNING SOLUTION

To materialize the solution of buildings underpinning, it was proposed the execution of the following elements at the ground floor level:

- Reinforced concrete beams grid, connected to the masonry walls with prestressed threaded bars to promote an effective load transfer do micropiles (see Figure 3) [concrete beams 60cmx80cm | ϕ32mm GEWI bars];

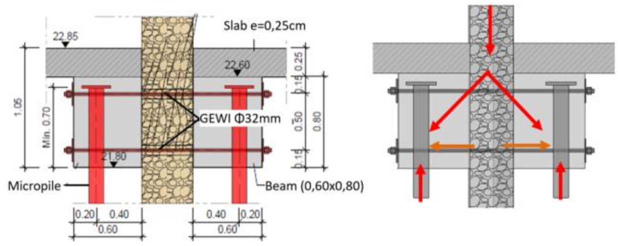

Figure 3. Underpinning solution – Load transfer beams.

- Reinforced concrete lining walls connected to the masonry peripheral walls with stell bolts to promote an effective load transfer [30cm thickness wall];
- Reinforced concrete slab on beam grid for materialization of the ground floor, overall rigidification of the solution and control of torsional forces in the beams [25cm thickness slab];
- Application of sprayable reinforcement mortars reinforced with carbon fibre meshes on internal masonry walls for their confinement and structural reinforcement [S&P ARMO-mesh | S&P ARMO-crete w] (see Figure 4);

Figure 4. S&P ARMO system application.

- Micropiles, vertical and sub-vertical, executed in the alignments of the walls and connected to the beams grid. Its 6m bond length materialized with repetitive selective injection (IRS) is placed outside the excavation influence area, in materials from the Lisbon Volcanic Complex geological unit (see Figure 5) [N80 tube 127x9mm].

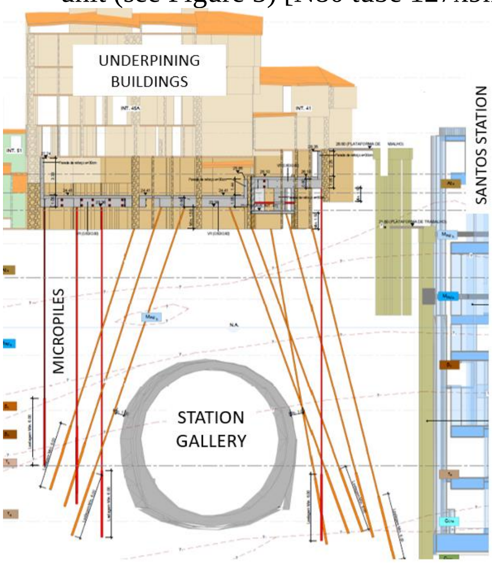

Figure 5. Underpinning solution - Cross section.

The underpinning solution was design to transfer the totality of the buildings load to Lisbon Volcanic Complex competent materials through micropiles with lengths from 20m to 25m, totalling about 4000m of micropiles tubes. That load transfer will occur when the underground works induces a settlement at build foundation level so it is expected that the load transfer can occur gradually with the tunnel excavation progress.

5 SOLUTION DESIGN

The underpinning concrete beam grid was designed using structural models within the SAP2000 software. The grid joint support was placed on micropiles location considering its inclination and using a spring to simulate the axial stiffness of the micropile free length. Those models made it possible to estimate beams structural forces as well as its elastic deformation (see Figure 6).

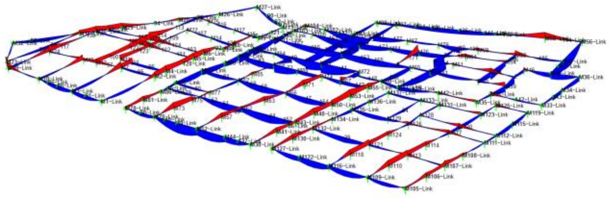

Figure 6. Beams grid SAP2000 analysis – Bending moments.

Considering the joint support springs on the structural model, the maximum design axial load on each micropile was estimated, allowing the safety validation of those steel elements. Since the micropiles free length is placed on influence excavation area it was necessary to account for second order effects so a 20mm as the maximum eccentricity a micropile can be subjected and therefore its design accounted for axial-bending moment verification. A numerical model considering NATM excavation phasing allowed the confirmation of displacements on micropiles to be less than 20mm (see Figure 7).

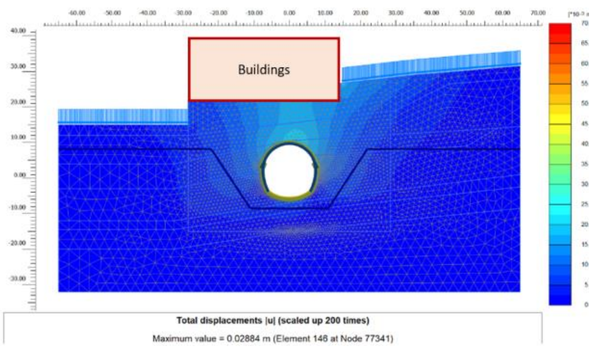

Figure 7. Plaxis 2D – Total displacements.

6 MONITORING AND SURVEY PLAN

Given the highly complexity of the present solution as well as the geotechnical substantial risk scenario a reinforced monitoring plan was implemented. Regardless of the underground excavation typical monitoring plans, at surface inside and outside the underpinning buildings multiple devices were installed such as inclinometers, topographic targets, tiltmeters, crack meters and liquid level sensors. Automated total stations (ATS) were used to access data daily aiming the access of in real-time building behaviour while the underground excavation is implemented and allowing for risk mitigation measures promptly if needed (see Figure 8).

The monitoring and survey plan the establishment of threshold values included for each monitoring device. The numerical analyses results were used to determine reference values for measured parameters that were used to set warning and alarm values, considering warning alarms thresholds as 80% and 120% of the reference values, respectively. Thus, the reach of warning limit means the approximation of the estimated behaviour and alarm means it was surpassed by 30% and therefore the works must stop, and reinforcement measures must be taken.

Figure 8. Automated total stations.

Up to this date, the monitoring survey plan has been implemented with daily measures and the buildings behaviour during the underpinning works and later with underground excavation has been proven as expected, with low settlements reaching a maximum of 3mm (see Figure 9).

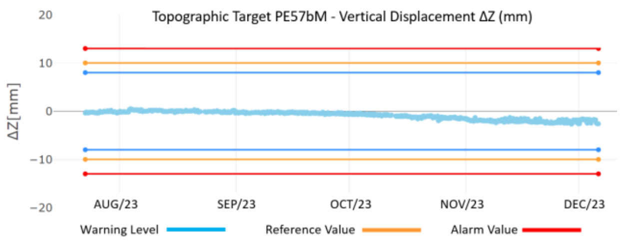

Figure 9. Topographic target - Vertical displacement.

Since the underpinning buildings are located on a hill the mass movements of the soil were also a concern since the station and tunnel underground excavations are taking place at the hill bottom. Thus, inclinometers with 35m long were installed behind those buildings to access deep horizontal soil movements that cloud indicate any global stability issue (see Figure 10).

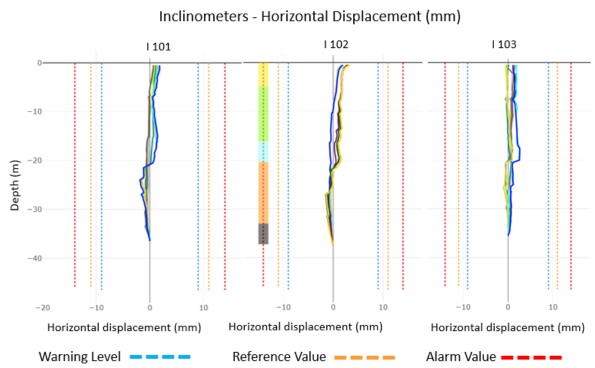

Figure 10. Inclinometers - Horizontal displacement over.

7 CONCLUSIONS

Underground excavations can lead to surface settlements that can impact the structural safety of buildings, particularly centenary buildings were multiply pathologies can already be observed. The use of numerical models for settlement estimation combined with Burland's method to access a buildings potential damage is crucial to determine the necessity of additional reinforcement measures. For extreme scenarios, such as the one presented, an underpinning solution can be most suitable way to maintain the building integrity despite this type of solution often as multiple constrains regarding equipment access and operation (see Figure 11). An adequate monitorization plan is essential to confirm an effective load transfer to the underpinning system while the underground excavation take place.

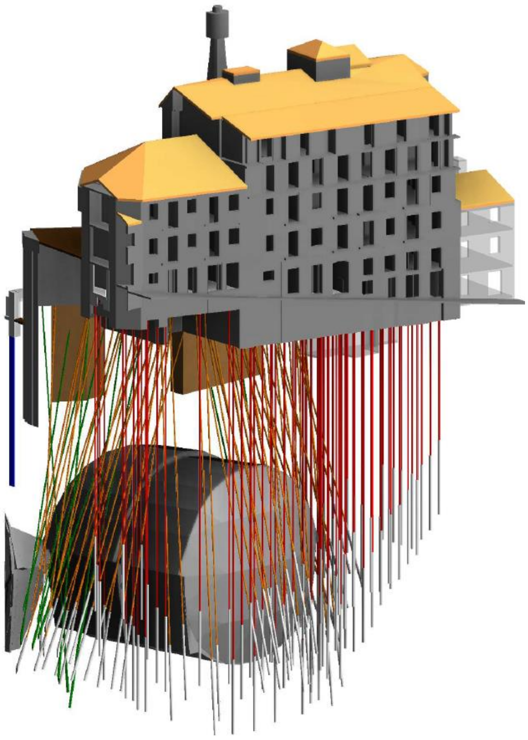

Figure 11. Buildings underpinning view.

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ECSMGE 24
XVIII EUROPEAN CONFERENCE
ON SOIL MECHANICS AND
GEOTECHNICAL ENGINEERING

Ground improvement solutions at the plot 14 of the Northern Lisbon Logistic Platform

Solutions d'amélioration du sol sur le lot 14 de la Plateforme Logistique du Nord de Lisbonne

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ABSTRACT: The present work addresses the ground improvement solutions adopted for the soft soils as foundation of the future industrial warehouse, to be built at the plot 14 of the Northern Lisbon Logistic Platform, at Castanheira do Ribatejo, Vila Franca de Xira, Portugal, comprising an area of 49 813 m^2. A description of the existing conditions is made, particularly of the geological-geotechnical scenario, highlighting the existence of an alluvial layer, with clayed soft soils, with low characteristics of strength and stiffness as well as very low average permeability. The solution adopted consists in the execution of preload embankments associated with (i) stone columns, executed at the area of the warehouse indoor pavements, and (ii) vertical prefabricated geodrains, at the area of the outdoor pavements, in the places designed for car parking and circulation of light and heavy vehicles. This article describes the criteria and methods used at the design of the preload fills, seeking to reduce the hydrodynamic consolidation schedule. The Monitoring and Survey Plan and respective results are also presented.

RÉSUMÉ: Le présent travail traite des solutions d'amélioration du sol adoptées pour les sols meubles comme fondation du futur entrepôt industriel qui sera construit sur le lot 14 de la plate-forme logistique du nord de Lisbonne, à Castanheira do Ribatejo, Vila Franca de Xira, Portugal et qui comprend une superficie de 49 813 m^2. Une description des conditions existantes est faite, en particulier du scénario géologique-géotechnique mettant en évidence l'existence d'une couche alluviale et de sols meubles et argileux, aux faibles caractéristiques de résistance et de rigidité, ainsi qu'une très faible perméabilité moyenne. La solution adoptée consiste à l'exécution de remblais de pré chargement associés à (i) des colonnes ballastées exécutées dans la zone d'entrepôt des trottoirs intérieurs et (ii) des geodrains préfabriqués verticaux au niveau des trottoirs extérieurs aux endroits prévus pour le stationnement et la circulation des véhicules légers et lourds. Cet article décrit les critères et méthodes utilisés lors de la conception des remblais de pré chargement cherchant à réduire le temps de consolidation hydrodynamique. Le plan d'observation et de surveillance et les résultats respectifs sont également présentés.

Keywords: Ground improvement; soft soils; preloading embankments

1 INTRODUCTION

The Northern Lisbon Logistic Platform (NLLP) is located at Vila Franca de Xira, Portugal, in an area of approximately 1 000 000 m^2. This platform was built with the intend of creating an interconnection point for international, national, and regional logistical flows within the Lisbon and Tagus Valley region.

On plot 14 of the stated platform, the construction of a new industrial building is expected, with an area of approximately 32 968 m^2. In the aerial view in Figure 1, it is possible to identify the target area for intervention, as well as the limits of the NLLP.

2 GEOLOGICAL AND GEOTECHNICAL CONDITIONS

The area is located on the right bank of the Tagus River, dominated by the alluvial soils of the Lower Tagus, ancient deposits of river terraces and, in depth, by soils dating from the Miocene.

Figure 1. Aerial view of the construction site (image taken from Google Earth).

According to the information obtained during the geotechnical investigations, it is confirmed that the local geological-geotechnical scenario is characterized, superficially, by a layer of landfills, developing to a depth variable between 1.1 m and 3.4 m. According to the available sampling, it is a heterogeneous layer made up of silts with calcareous gravel.

Immediately below the layer of landfills, to a maximum depth of 20,0 m, there are alluvial deposits, essentially made up of soft silts and very soft high plasticity clays, with a high content of organic matter and shell fragments.

Underlying the alluvial layer, colluvial formations can be identified, composed of sandy clays of medium consistency with rounded heterometric gravel of an essentially quartzite nature.

The position of the water table is dependent on the water level in the Tagus River, being influenced not only by seasonal variations, but also by the daily tidal cycles felt in the estuary. During the prospecting geotechnical works, the presence of the water table was detected at a depth varying between 1.3 m and 3.4 m, generally occurring at the level of the interface between the natural terrain and the overlying landfills.

3 GROUND IMPROVENT SOLUTIONS

Given the presence of soft clayey soils, with low strength and stiffness, as well as reduced support capacity, and considering the extended period available for the execution of the works, it was decided to implement ground improvement solutions with the aim of increasing the geomechanical characteristics of the foundation soils, thus providing favourable conditions to obtaining total and differentials settlements compatible with the good performance of the pavements during the service lifetime of the new industrial building.

In this context, a consolidation ground improvement solution was adopted using preload embankments.

To accelerate the process of hydrodynamic consolidation of the compressible materials (soft clayey soils), the introduction of drainage elements was considered, namely, stone columns, at the area of the warehouse indoor pavements, and prefabricated vertical drains (PVD), at the area of the outdoor pavements, in the places designed for car parking and circulation of light and heavy vehicles. It should be noted that the stone columns were installed in a previous work in 2010, while the PVD were installed more recently in 2022.

Considering the magnitude of the predicted service load in the indoor and outdoor pavements (50 kN/m^2 and 20 kN/m^2, respectively), it was proposed to build 8.0 m and 4.0 m high embankments, with slopes H=1.5; V=1.0.

The stone columns were arranged in a square array with 3.0 m centre-to-centre spacing. The PVD were installed in a triangular arrangement with 1.2 m spacing. Both these elements were executed up to an estimated average depth of 20.0 m, in relation to the work platform elevation.

In Figure 2 the typical cross section of the consolidation ground improvement solution is presented.

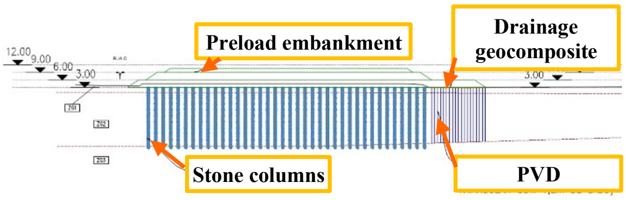

Figure 2. Typical cross section of the consolidation ground improvement solution (not to scale).

Given the presence of a surface layer of landfills, the execution of the PVD required pre-drilling works (Figure 3).

Figure 3. Installation of the prefabricated vertical drains.

After the installation of the PVD, and prior to the execution of the pre-load embankments, a drainage geocomposite responsible for directing the inflowing water to the vertical geodrains was applied (Figure 4).

Figure 4. Aerial photograph of the drainage geocomposite installation.

Figure 5 shows an aerial photograph of the preload embankments.

Figure 5. Aerial photograph of the preload embankments.

4 DESIGN

4.1 Settlements

The total settlement of the compressible soil as a result of the increase in vertical stresses can be calculated by adding the portions corresponding to (i) the immediate settlement, produced during construction in undrained conditions, therefore without volume variation; (ii) primary settlement, resulting from the expulsion of water due to the dissipation of excess pore pressure, during the development of the primary consolidation process; (iii) and secondary settlement, corresponding to the continuation of soil deformation after conclusion of primary consolidation, corresponding to secondary consolidation or creep.

For the problem under study, immediate and primary consolidation settlements were considered.

The value of immediate settlement was estimated, adopting, for the foundation soil, an isotropic linear elastic behaviour at constant volume and carefully choosing the deformability modulus in undrained conditions.

The methodology for calculating settlements by primary consolidation included the application of Terzaghi's theory of one-dimensional consolidation, associated with a two-dimensional distribution of loads transmitted to the ground.

For this purpose, the settlement by primary consolidation was estimated based on the parameters obtained in the oedometer laboratory tests, discretizing the compressible stratum into layers, and estimating the total settlement considering the variation in effective stresses in depth.

The stress increments for the stated calculation and, as such, for definition of the height of the preload embankments, were determined, considering the self-weight of the embankments (temporary and permanent), the self-weight of the pavement slab and the different service loads. It should be noted that in this estimate second-order effects associated with the expected settlement of landfills were considered.

The estimate of the duration of the primary consolidation process involved calculating the estimated vertical and horizontal consolidation times and subsequently the total value resulting from these two mechanisms.

Having knowledge of the total settlement value associated with a given embankment height, as well as the law that governs its evolution over time, it was possible to estimate the settlement evolution curves over time shown in Figure 6.

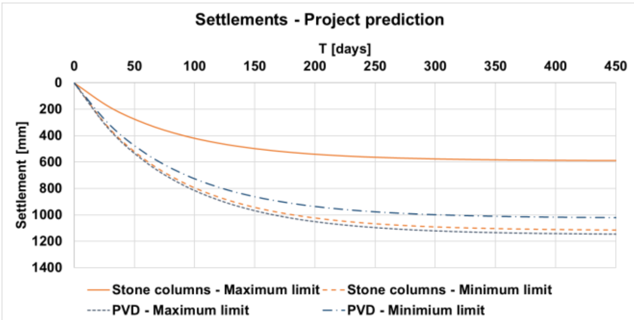

Figure 6. Primary consolidation settlements prediction at design stage.

4.2 Slope stability

As part of the verification of the global stability of the pre-load embankment slopes and the bearing capacity of the foundation, including the quantification of safety factors, an automatic calculation program designed for this purpose was used: SLIDE (V6.0) which allows carrying out analyses in a limit equilibrium regime. This software made it possible to carry out a stability analysis, using circular failure surfaces based on the Bishop-Simplified Method. Two-dimensional finite element analysis was also carried out in parallel using the PLAXIS 2D software.

5 MONITORING AND SURVEY PLAN

To guarantee the carrying out, in safe conditions, of the ground improvement works, mainly the stability of the embankments/foundation and the evolution of settlements/degree of consolidation, a Monitoring and Survey Plan was employed.

In this context, it was considered essential to monitor the following quantities:

a) Water levels installed on the ground, through the installation of 72 vibrating wire piezometers, arranged in a total of 18 locations, at 4 different depths.

b) Horizontal movements of the ground measured using 6 inclinometers, arranged on the periphery of the embankments.

c) Surface settlements of the natural terrain and embankments/platforms, through the measurement of topographic marks supported on 52 settlement plates.

Figures 7 and 8 show, respectively, the vertical displacements measured in the settlement plates, recorded until March 9, 2023, in the area where the stone columns and PVD were built.

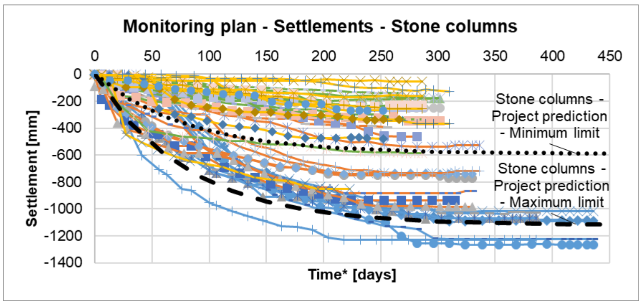

Figure 7. Observed settlements – stone columns.

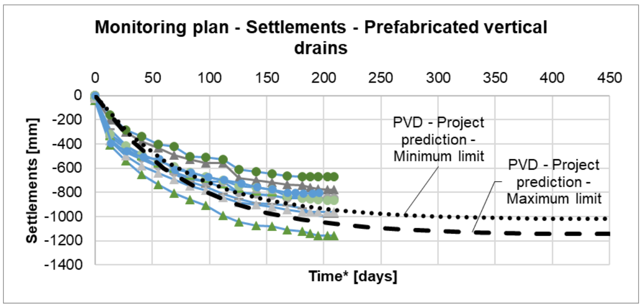

Figure 8. Observed settlements – prefabricated vertical drains.

Regarding measurements of water levels and horizontal displacements, it is important to note that the late installation of piezometers and inclinometers did not allow for a complete record of these quantities to be obtained from the beginning of the construction of the pre-load embankments. Nevertheless, the evolution recorded is in accordance with the project prediction, particularly regarding the reduction, over time, of pore water pressures tending towards values that are close to the hydrostatic distribution. Regarding the recording of horizontal displacements, the mobilization of displacements of greater magnitude was observed at the depths corresponding to the existence of the soft clayey stratum, with a higher rate of evolution during the construction period, which stabilized after the end of the construction of the embankments.

6 FINAL REMARKS

The framework of the work described, including the available time, determined the need to develop innovative ground improvement solutions aiming to provide favourable conditions for the future employment of economic foundation solutions, compatible with the structure serviceability, not only on plot 14, but also on adjacent plots, taking profit of both soil behaviour knowledge (from monitoring results) and existent preloading embankment.

In this context, it should be noted that the adopted solutions make it possible to mitigate, during the life of the future industrial structure, high magnitude settlements, resulting from the primary consolidation process, developed at the level of soft clayey alluvial layers.

The importance of the Monitoring and Survey Plan in managing the behaviour of the work is highlighted, allowing the interpretation of the observed settlements and the establishment, with greater accuracy, of the time necessary to obtain the specified degree of consolidation. As such, it is an indispensable tool in a geotechnical work, with the characteristics of this one.

It is noted that the magnitude of settlements estimated at the project stage, in general, was in line with the values measured on site. The exception to this statement were the settlements recorded in the NW area of the plot, where the stone columns were installed, and where settlements of a magnitude considerably lower than the values recorded and estimated for the SE area were observed. This difference is attributed to the effect of the prolonged presence of previously constructed pre-load embankments, as well as the effect of reinforcing the ground provided by the stone columns.

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Excavation, retaining wall and deep foundations solutions for an office building at Alcântara – Lisbon

Excavation, mur de soutènement et fondations profondes pour un immeuble de bureaux à Alcântara - Lisbonne

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ABSTRACT: This paper presents the proposed solutions for the excavation works, retaining wall and deep foundations necessary for the construction of 4 underground floors of two office buildings at Alcântara, adjacent to Avenida da Índia, in Lisbon. The main existing constraints were archeological, geological, hydrogeological, geotechnical and the neighbourhood conditions, which determined the choice of the retaining wall solution and respective horizontal support. Diaphragm wall technology was used for the retaining wall combined with a top-down sequence to assure the horizontal support. The solutions adopted for the deep foundations are also described, namely the use of high-capacity base grouted bored piles. Particular emphasis is given to archaeological aspects, which strongly determined the top-down solution and construction sequence. Finally, the main results of the monitoring devices are presented as well as the changes implemented during the works determined by differences between the measurements and the design estimations.

RÉSUMÉ: Cet article présente les solutions proposées pour les travaux d'excavation, de mur de soutènement et de fondations profondes nécessaires à la construction de 4 étages souterrains de deux bâtiments de bureaux à Alcântara, adjacents à l'Avenida da Índia, à Lisbonne. Les principales contraintes existantes étaient les conditions archéologiques, géologiques, hydrogéologiques, géotechniques et de voisinage, qui ont déterminé le choix de la solution du mur de soutènement et son support horizontal. La technologie des parois moulées a été utilisée pour le mur de soutènement, combinée à une séquence descendante pour assurer le support horizontal. Les solutions adoptées pour les fondations profondes sont également décrites, à savoir l'utilisation de pieux forés de grande capacité avec injection the coulis de ciment dans la base. Un accent particulier est accordé aux aspects archéologiques, qui ont fortement déterminé la solution et la séquence constructive. Enfin, les principaux résultats des dispositifs de suivi sont présentés ainsi que les changements mis en œuvre au cours des travaux déterminés par les différences entre les mesures et les estimations de conception.

Keywords: Diaphragm wall; bored piles; top-down; excavation; ground anchor.

1 INTRODUCTION

The foundation, excavation and retaining wall solutions developed for the construction of the underground levels of the Alcântara Riverside project, located on Avenida da Índia in Alcântara, Lisbon, were defined to enable the construction of 4 underground levels under safe conditions and to guarantee the minimisation of the impact on the existing constraints in the surroundings (Figure 1).

Considering the location of the building, there was a set of extremely important natural and anthropogenic constraints that strongly determined the conceptual design of the works, namely: the proximity to the River Tagus, which determined the existence of a high water table; the existence of very permeable sandy alluvium which, together with the high water table, recommended that ground anchors should (prudently) not be used below the water table; and the existence of important archaeological findings scattered throughout the intervention area, which needed to be preserved and/or recorded before they suffered any affect resulting from the works.

To take into account all the existing constraints, a diaphragm wall solution was defined, anchored at the top level (at the capping beam) and propped at the level of floors -1 and -3 by slab strips and metal struts.

Figure 1 – Site location.

The foundation solution consisted of Ø1000mm and Ø1200mm high-capacity base grouted bored piles, embedded at the basalt rock and injected through the cross-hole tubes at a depth of 3.0m below the base.

2 MAIN CONSTRAINS

2.1 Geological and geotechnical scenario

An in-depth analysis of the geological conditions revealed sedimentary materials overlying volcanic formations, requiring specialized geotechnical solutions (Figure 2).

A first step in comprehending the geological and geotechnical intricacies scenario of the Alcântara region involved deploying advanced subsurface exploration techniques. The deployment of Cone Penetration Tests (CPT), borehole drilling, and geophysical surveys played a central role in mapping the subsurface layers and soils distribution. This section delves into the methodologies employed, emphasizing the importance of obtaining accurate and high-resolution data to support subsequent design phases.

The Alcântara site exhibited a diverse range of sedimentary layers, ranging from loose soils to compacted clays. Understanding these layers was fundamental to devising foundation strategies. The paper expounds on the challenges posed by the variability in sediment types and how these challenges were addressed in the selection of foundation elements. The stability of basalt formations, identified at deeper levels, provided a robust material for foundational elements.

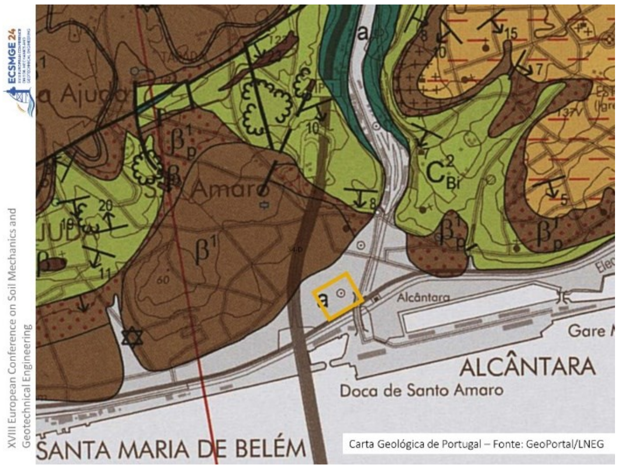

Figure 2 – Portuguese geological map.

2.2 Hydrogeological scenario

The altered water flow resulting from human interventions demanded a thorough evaluation of hydrogeological conditions. In the present case, to assess the impact of the 4 basements on the underground flow regime, a global hydrogeological study was developed by LNEC (2018), in accordance with the provisions of PP Alcântara Poente (Figure 3). The result of the study indicates that the impact on the ground water table was neglectable and that no mitigation measures were needed.

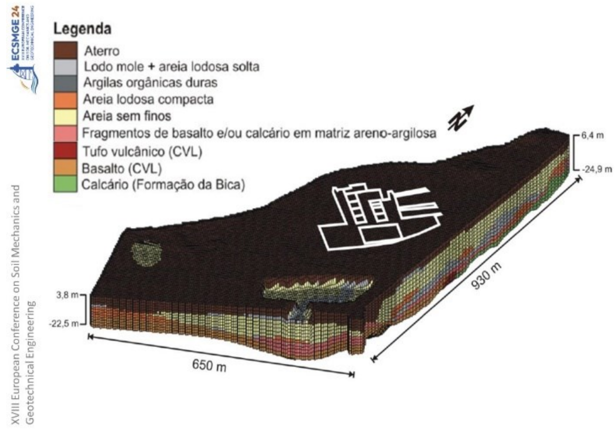

Figure 3 – Hydrogeological study – Global model (LNEC 2018).

2.3 Archaeological factors

The existence of archaeological remnants, up to a depth of at least 6,0m, including several well-preserved quay walls, added an extra layer of complexity to the project (Figure 4).

Of all the structures identified, it was considered that the existing quay wall along the entire southeast alignment should be preserved in situ, with the remaining archaeological structures being demolished after proper registration.

In this context, the peripheral retaining wall, combined with temporary ground anchors allowed an unconditioned archaeological excavation to a general depth of 6.0m and an additional excavation up to 9.0m deep in the central area of the excavation.

Figure 4 – Archaeological remains.

uniformly and mitigate the impact of hydrostatic pressure.

In this context, the foundation piles were designed for two extreme scenarios: one in which the water table could reach 1.20m above the ground surface (in the long term due to sea level rise); and another in which the water table could not determine the installation of significant water pressure under the slab (in the short term and dependent on the permeability of the soil involved).

Figure 5 – Bored piles drilling and base grouting.

3 SOLUTIONS

This section provides a detailed insight of the chosen geotechnical solutions. It includes the rationale behind opting for reinforced concrete bored piles, the structural considerations in designing a substantial raft foundation, and the application of diaphragm walls supported by temporary anchors and slab bands. The flexibility and adaptability of these solutions to address unforeseen challenges are emphasized.

3.1 Deep foundations

The selection of reinforced concrete bored piles as foundation elements was supported in a meticulous understanding of the geological strata. The Ø1000mm and Ø1200mm piles were designed to pass through the heterogeneous sedimentary layers and anchor firmly into the stable basalt formations. Detailed geotechnical analysis, combined with high pressure grouting of the pile bases, guided the precise placement of these piles, and allowed to maximize their load-bearing capacity (Figure 5).

The design of the raft foundation emerged as a critical aspect, considering the hydrogeological challenges inherent in the project site. A robust prestressed raft was designed to distribute loads

3.2 Retaining wall and support system

The implementation of diaphragm walls as earth retaining solution was imperative for this project since it ensures high quality results, minimizing the number of joints between panels, reducing the vertical deviation of the wall, and decreasing the risk of water inflow into the excavation area. These aspects are quite important due to the height of the wall (up to 32,0m), the presence of high permeability sandy alluvium soils, and the existence of several important and settlement-sensitive city infrastructures (Figure 6).

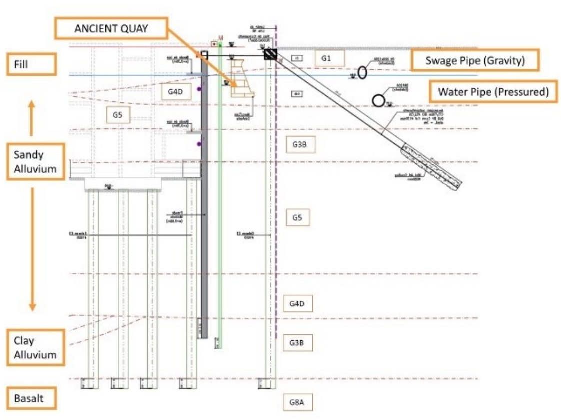

Figure 6 – Cross Section of the retaining wall.

The design of the horizontal support of the wall was defined to use temporary ground anchors at the capping beam and two levels of slab bands (floor -1 and floor -3). The self-drilling ground anchors were made of tubular profiles ANP 52/26 with 10m to 17m of free lengh and 7.0m of pressure grouted lengh.

The execution of these anchors allowed the full excavation until the depth of 6.0 m and the necessary registry of all archaeological artefacts, ensuring compliance with the archaeological heritage preservation critiria. After that, the foundation piles were executed, including the teporary collumns to support the slab bands (Figure 7).

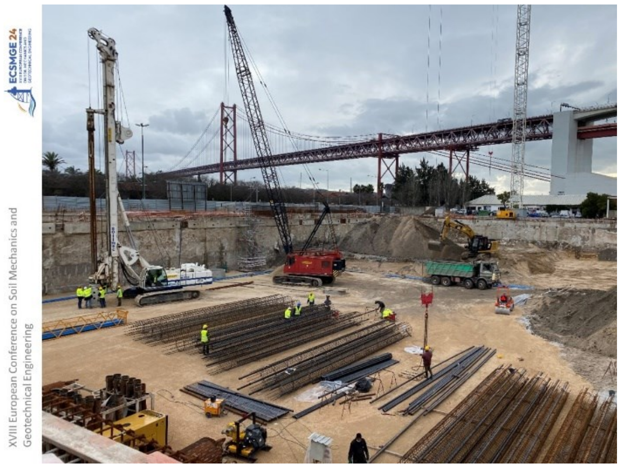

Figure 7 – Excavation works at 6.0m depth.

The general design of the slab bands account not only the resistance and stiffness necessary to guarantee safety conditions of the retaining wall, but also the associated construction process, both the excavation phase and the construction phase. In this sense, the geometry of the slab bands was defined in such a way as to interfere as little as possible with the execution of the vertical elements of the structure of the building.

During the excavation phase, the slab bands were supported by the retaining wall at the external perimeter and HEB200 steel collumns sealed into the foundation piles of the definitive columns (Figure 8).

Figure 8 – Execution of slab bands.

4 NUMERICAL MODELING

Advanced numerical modelling tools, such as PLAXIS 2D (Figure 9) and SAP2000 (Figure 10) were used to analyse the structural and geotechnical behaviour of the earth retaining system. The behaviour of the soil was simulated using the Hardening Soil constitutive model. The retaining walls and metallic profiles were modelled using "Plate" beam-type elements, with elastic behaviour. The anchors were modelled with spring-type elements, namely "node-to-node anchor", while the slab bands were modelled using "Fixed-end anchor" spring-type elements. Finally, the foundation piles were modelled using "Embedded Beam Row" elements.

Regarding the high water table and the high permeability of several aluvium soil layers the Plaxis analysis were performed considering a steady state water flow and the flow rate was estimated to design the temporary water pumps.

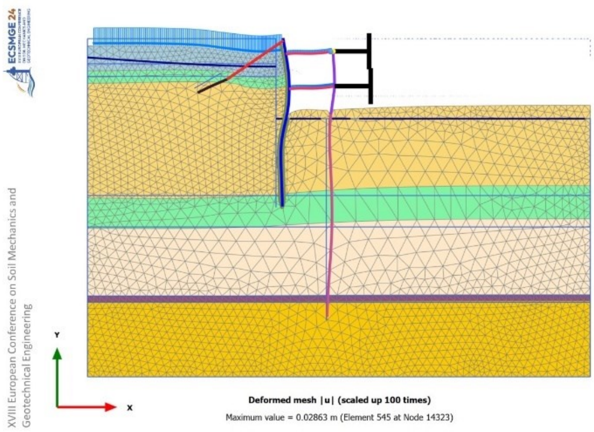

Figure 9 – Plaxis 2D model. Final phase of excavation.

The slab strips on floors -1 and -3 and the additional steel struts, were analysed using the automatic structure calculation program SAP2000. The actions acting on the slab strips were determined in the PLAXIS 2D model and their stiffness was also incorporated in that model.

5 MONOTORING AND OBSERVATION PLAN

Figure 10 presents the monitoring plan implemented during construction, which involved various instruments to track displacements (inclinometers), rotations (tiltmeters), settlements (settlement marks), and groundwater levels (piezometers).

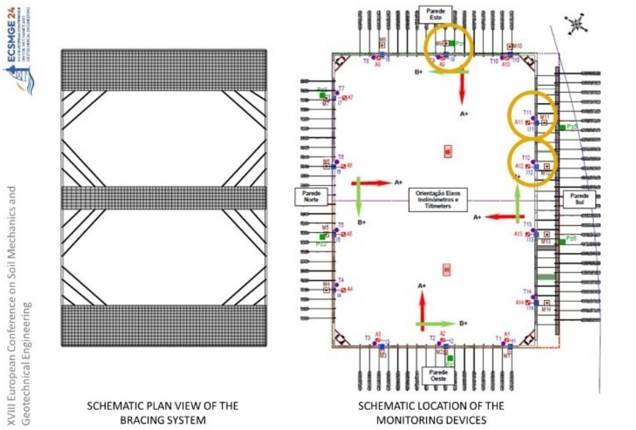

Figure 10 – Monitoring devices (Inclinometer I9, I11 and I12).

During the progress of the excavation works, there was a general tendency to measure horizontal displacements of the wall greater than those foreseen in the design, particularly along the elevation supported by steel struts which are more deformable than the strip slabs.

The inclinometer graphs on Figure 11 allows the identification of much higher deformations (75mm) on I11 and I12 which are placed in a location where the diaphragm wall is supported by steel struts, than on I9 (20mm) which was placed in a location where the diaphragm wall is supported by a strip of the floor slab.

The estimated horizontal deformation of the wall for both section was 22mm and 27mm, respectively.

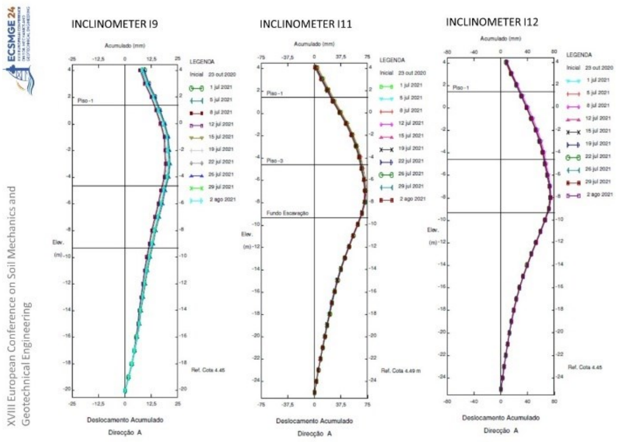

Figure 11 – Inclinometer graphs.

The comparison of the wall deformation presented in Figure 9 with the real deformation measured by the inclinometers (Figure 11), and the retroanalysis performed during the construction works, indicated that the most probable cause of the higher deformation was a combination for two aspects:

- Overestimation of the steel struts' stiffness due to an underestimation of the slab strips stiffness. The smaller deformation of the slab strips decreases the induced deformation of the corner struts which reduces the efficiency of the struts on the perpendicular elevation;
- Underestimation of the stiffness reduction of the clean sand alluvial formations located at the passive zone of the wall.

6 CONSTRUCTION CHALLENGES AND ADAPTATIONS

Unforeseen challenges, including higher-than-expected deformations and anomalies in foundation pile tests, are addressed in this section.

Several steel struts were encased in reinforced concrete to increase their stiffness and, indirectly, to minimise the horizontal deformations of the retaining wall.

During the execution of the cross-hole integrity tests, planned for all foundation piles, the existence of some anomalies was detected, namely lack of covering, enlargements of the section, concrete segregation, and intercalation of soil pockets (Figure 12).

Figure 12 – Foundation piles anomalies.

Figure 13 – Bottom drainage gravel layer.

In some situations, where the pathologies were relevant for the future performance of the piles, due to their position on the building layout or the expect stress level, some reinforcement measures were implemented. In the most demanding situations, namely in a structural alignment where there was only one pile under each column, it was necessary to enhance the load capacity through the execution of additional large capacity self-drilling micropiles of 3200kN ultimate load.

During the excavation works, through the quantification of the flow rates pumped daily, it was possible to confirm that the inflow of water into the excavation area was extremely low when compared to the estimates from the design phase (conditioned by the various permeability tests carried out during the initial surveys). Based on these results and previous experience (Pita, 2012), the planed raft foundation was replaced by a drainage mattress and individual pile caps connected by foundation beams (Figure 13).

7 CONCLUSION

The success of the project is attributed to the careful integration of geological, hydrogeological, and archaeological considerations.

The adaptability of the proposed solutions, supported by systematic monitoring of the retaining wall was determinant in the timely implementation of corrective measures and cost optimization decisions.

ACKNOWLEDGEMENTS

The authors would like to thank the Owner of the building for authorising the writing and publication of this paper. They also would like to emphasise that the success of the solutions implemented was the result of teamwork of the following companies: JETsj Geotecnia as designer, Rockbuilding as project and supervision manager, Alves Ribeiro / HCI as general contractor, and Teixeira Duarte as geotechnical subcontractor.

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Solutions of excavations and peripherical earth retaining walls in dense urban area

Solutions d'excavations et de murs de soutènement périphériques en zone urbaine dense

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ABSTRACT: This paper describes the main design and execution criteria for peripheral earth retaining solutions required to construct a multi-family residential building on Avenida Conde Valbom in Lisbon. The building covers approximately 200m², encompassing three underground and seven elevated floors. The excavation area is bordered by a 10-floor building with one underground level to the north and an 8-floor building with five underground stories to the south. Geological findings indicate a groundwater level of around 4 meters. Soil investigations reveal clayey fill materials, rocky fragments, and other elements atop dark gray clays, followed by Miocene formations named "argilas e cálcarios dos prazeres" and "Formação de Benfica" from the Eocene-Oligocene era. For excavation and containment, the proposed method involves utilizing the "king post wall" technique horizontally supported by a slab strip. This approach includes phased construction of vertically reinforced concrete panels, supported by micro-piles. The slab strip provides load-bearing support and incorporates HEB140 double shorings where necessary. This solution permits a thinner wall (0.30m) and simultaneous execution during excavation. To ensure stability against soil pressures during excavation, concrete pouring of panels directly into the ground is planned, with support from the slab strip and metallic shorings at existing floor levels. Monitoring and survey plan aims to execute excavation and containment structures safely and economically while assessing the behavior of neighbouring structures during construction. It addresses identified constraints and quantifies risks associated with the construction phase.

RÉSUMÉ: Le document décrit les principaux critères de conception et d'exécution des solutions périphériques de retenue des terres nécessaires à la construction d'un immeuble résidentiel multifamilial sur l'Avenida Conde Valbom à Lisbonne, couvrant une superficie d'environ 200m² comprenant trois étages souterrains et sept étages surélevés. La zone de fouille est délimitée par un bâtiment de 10 étages avec un sous-sol au nord et un bâtiment de 8 étages avec cinq sous-sols au sud. Les résultats géologiques indiquent un niveau d'eau souterraine d'environ 4 mètres, tandis que les investigations du sol révèlent des matériaux de remplissage argileux, des fragments rocheux et d'autres éléments sur des argiles gris foncé, suivis par des formations du Miocène appelées "argilas e cálcarios dos prazeres" et "Formação de Benfica" de l'ère de l'Éocène-Oligocène. Pour l'excavation et le support, l'approche proposée implique l'utilisation de la méthode "king post wall" soutenue horizontalement par une bande de dalle. Cette méthode comprend une construction par phases de panneaux en béton armé verticalement soutenus par des micro-pieux. La bande de dalle assure le support de charge et intègre des étais doubles HEB140 là où nécessaire. Cette solution permet un mur plus mince (0,30m) et une exécution simultanée pendant l'excavation. Pour assurer la stabilité contre les pressions du sol pendant l'excavation, la coulée de béton des panneaux directement dans le sol est prévue, avec le soutien de la bande de dalle et des étais métalliques aux niveaux de sol existants. Le plan d'instrumentation et d'observation vise à exécuter les structures d'excavation et de retenue en toute sécurité et de manière économique, tout en évaluant le comportement des structures environnantes pendant la construction. Il aborde les contraintes identifiées et quantifie les risques associés à la phase de construction.

Keywords: Earth retaining wall; slab strip; king post wall; monitoring and survey plan

1 INTRODUCTION

The present article describes the solutions adopted for the excavation and peripheral soil containment, necessary for the construction of the 3 underground floors of a multi-family residential building located on Avenida Conde Valbom nº21-25, in Lisbon. In Figure 1, the location of the construction site is presented, highlighting the following boundaries for the excavation area:

- Conde Valbom Avenue side: The terrain is approximately at the level of floor 0.
- Northern side: Existing building with 10 floors and 1 basement floor.
- Southern side: Existing building with 8 elevated floors and 5 basement floors.
- Rear side: Boundary with the existing backyard.

Figure 1- Aerial view of the intervention site (images taken from Google Earth)

The building to be constructed has a footprint area of approximately 200 square meters, comprising three underground floors and 7 elevated floors plus a rooftop.

2 MAIN CONSTRAINTS

2.1 Neighbourhood conditions

The excavation and peripheral containment solutions must be compatible with preserving the proper functionality of all structures and infrastructures located near the excavation area. The Figure 2 illustrates the plan view of the neighbourhood main constrains.

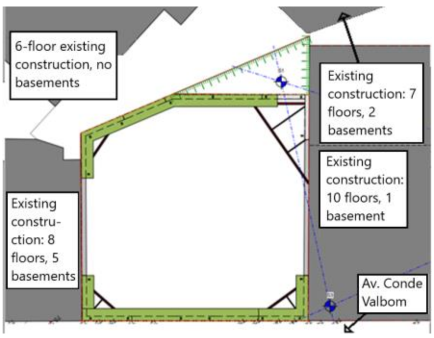

Figure 2 –Plan view of the neighbourhood constrains.

2.2 Geological and geotechnical conditions

The geological-geotechnical survey encompassed three boreholes for SPT tests and sample collection. Reported groundwater levels approximate 4 meters. Survey results reveal primarily brown clayey fill materials to a depth of about 3.0 meters. Subsequently, Miocene formations display whitish-gray silts, and lower layers exhibit clayey and silty sands, attributed to the Eocene-Oligocene.

The findings categorized three geotechnical zones: ZG1 (NSPT 7-15) for recent cover deposits, ZG2 (NSPT 18-45) for uncompressed in situ masses, and ZG3 (NSPT 45-60) representing the more robust "Formação de Benfica".

3 EXCAVATION AND PERIPHERAL EARTH RETAINING SOLUTIONS

In the design of the adopted peripheral earth retaining solutions, the objective was to maintain crucial principles such as: (a) Manage soil deformation and safeguard neighbouring structures and infrastructures against excavation-related impacts; (b) Minimize disruptions to adjacent structures and infrastructures as much as possible.

Considering the existence of various conditioning factors, the excavation and peripheral containment were executed using the "king post wall" technology, coupled with the installation of a slab strip as illustrated on Figure 3.

. The construction approach for the "king post wall" type containment wall involves a phased assembly, from top to bottom, of reinforced concrete panels supported by vertical micro-piles. The profiles are inserted into holes and sealed along the sealing bulb length using the IGU injection system in competent terrain. The slab strip integrates a section of the final slab to bear soil loads, while double shorings are deployed in areas where the slab strip isn't possible to execute. Provided the assumptions hold, panels will be directly concreted into the ground to stabilize the containment wall against soil pressures during excavation, supported by the slab strip and metallic shorings at existing floor levels (0, -1, and -2). The following figures represents the plan view of the -1, -2 and -3 floor. To minimize soil decompression, strict adherence to the proposed construction phasing is important, notably ensuring a 24-hour maximum gap between excavation and concreting. In the definitive phase, floor slabs at levels 0, -1, and -2 will themselves ensure the containment wall's stability, transitioning temporary props to permanent upon completing the mentioned structure.

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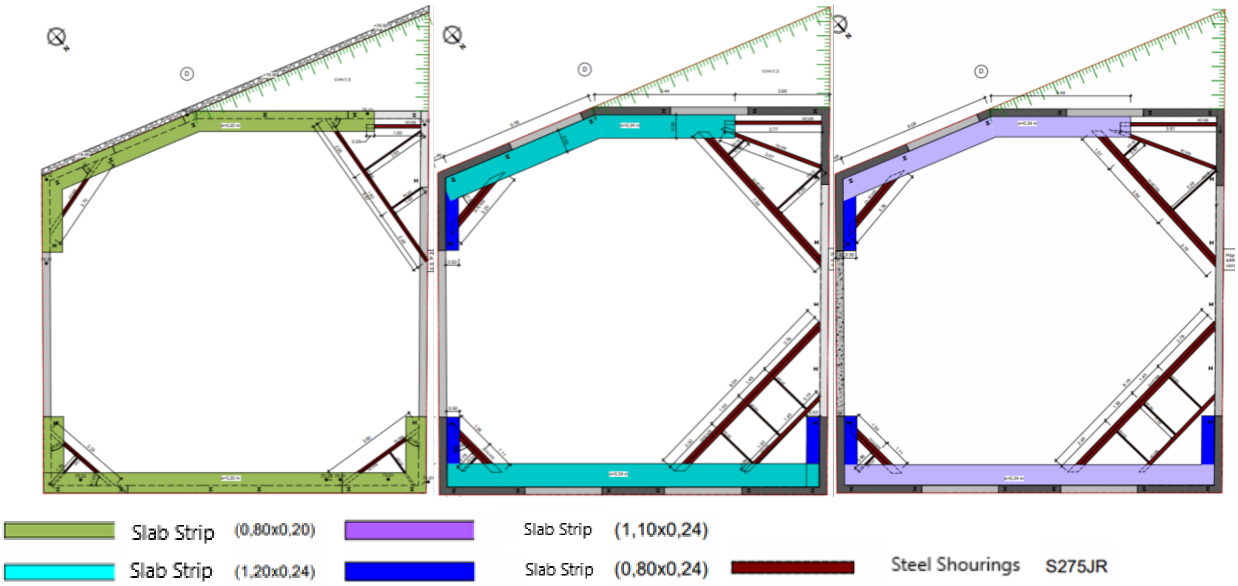


Figure 3 – Design plan view of the peripheral earth retaining solution at levels 0, -1 and -2

4 GEOTECHNICAL AND STRUCTURAL DESIGN

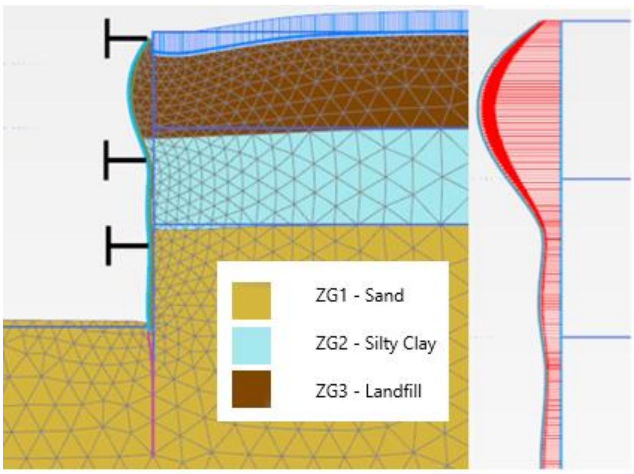
The evaluation of the proposed methods for the peripheral earth retaining solutions in terms of forces and deformations encompassed a comprehensive analysis across construction phases. This assessment utilized dedicated finite element program PLAXIS 2D, purpose-built for this specific task. A constitutive model was developed to mimic the behaviour of "Hardening Soil," considering nonlinear constitutive relationships and variations in soil stiffness under differing stress states. These outlined parameters from Table 1 were then implemented in PLAXIS 2D to accurately model the behaviour of the soils.

Table 1 - Geotechnical soil parameters.

Soil Type	ZG3	ZG2	ZG1
N_{SPT}	7-15	18-45	45-60
Υ [kN/m^3]	18	18	20
Φ [º]	27	30	38
E_{50}^{ref} [MPa]	10	40	90
E_{ur}^{ref} [MPa]	30	120	270

ZG_1 – *Loosely compact clay fill;* ZG_2 – *moderately compact silt soil and clayey sandy soil;* ZG_3 – *compact clayey sand soil.*

In these models, the "king post wall" and the vertical micropile were represented using "plate" elements exhibiting elastic behavior. Meanwhile, the slab strip and the steel struts were simulated as "anchors." The behaviour of the peripheral retaining structure underwent analysis, evaluating design aspects such as containment structure forces, deformations, stress conditions, and soil stability. Figure 4 showcase specific displacement results derived from these developed models.



u	*16,34 mm*	u_x	*15,82 mm*

Figure 4 – Expected maximum total displacements (u) and maximum horizontal displacements (ux).

5 MONITORING AND SURVEY PLAN

Based on the complex framework of the site, a Monitoring and Survey Plan (MSP) was established to ensure safety and optimal functionality during excavation and the construction of geotechnical structures, as well as for neighbouring structures and infrastructures. Within this plan, the following instruments were installed: (a) 20 topographic targets positioned along the earth retaining solution and placed along neighbouring infrastructure; (b) 1 inclinometer placed behind the peripheral containment wall.

Alert and alarm criteria were established for all instruments and monitored structures based on the conducted modelling. Overall, the readings consistently remained below the defined alert and alarm criteria outlined in the project. Figure 5 and Figure 7 illustrates the schematic locations of the instrumentation devices along with some of the most noteworthy results.

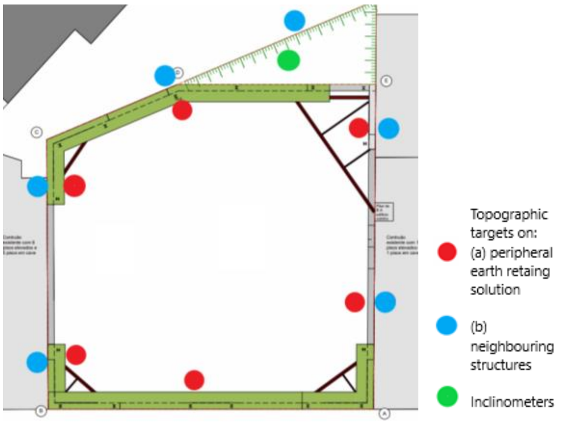


Figure 5- Instrumentation and monitoring plan

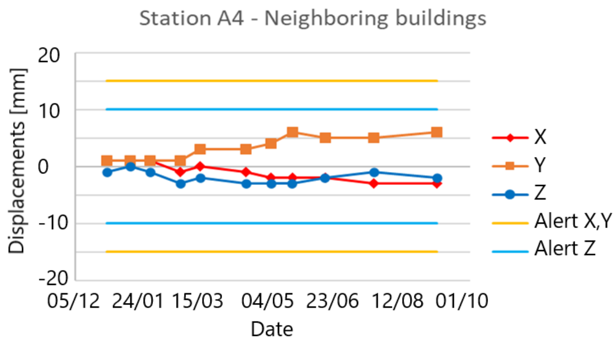


Figure 6 - Maximum displacements on neighbouring structure (station A4)

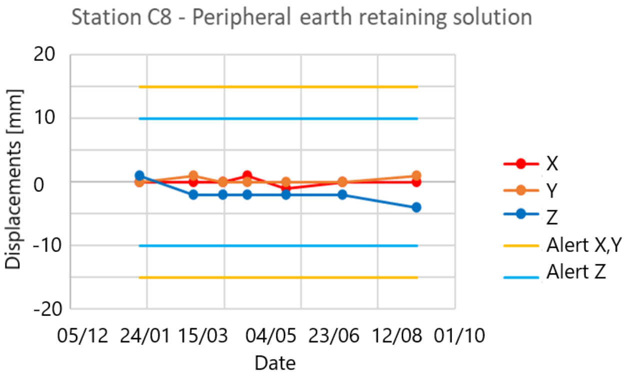


Figure 7 – Maximum displacements on peripheral earth retaining solution (station C8)

In general, maximum displacements of about 6mm were recorded for neighbouring structures, and approximately 3mm for the peripheral containment solution, values well below the alert and alarm criteria set at the beginning of the excavation, which were 15mm and 30mm, respectively. As for vertical displacements, a maximum value of around 4mm was registered, equally well below the alert and alarm criteria of 10mm and 20mm, respectively. The apparent discrepancy between estimated and observed deformations can be attributed to several factors, notably the geomechanical attributes of the soils involved. Specifically, the Miocenic formations exhibit superior characteristics compared to those initially considered in the design calculations.

6 CONCLUSIONS

In the scope of this article, it was possible to demonstrate the technical efficiency of king post wall braced by various structural elements such as struts, and slab stripes. The peripheral earth retaining solution presented actual displacements lower than those estimated in project and displayed a highly stable behaviour during the excavation works. Additionally, the use of slab bands not only overcame the encountered limitations due to the excavation´s site proximity to the technical gallery and the future underground parking lot, but also incorporated elements of the final structure, enhancing the economic efficiency of the solutions. One of the most daunting hurdles encountered throughout the project was the limited space available for the construction site. Situated within a densely urbanized environment, the work site posed significant challenges, primarily due to its restricted area. This spatial constraint significantly impacted various aspects of the construction process, from material storage to site infrastructure and the removal of excavated soil.

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Solution of excavation and peripheral earth retaining walls near sensitive structures – O'Living Development, Lisbon

Solutions d'excavation et murs de soutènement périphériques à proximité des structures sensibles – Développement O'Living, Lisbonne

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ABSTRACT: This paper refers to the excavation and retaining wall solutions for a residential development located in Lisbon, where part of the basement structure is located 4m above the Lisbon Metro (ML) tunnel crest. The constraints related to the execution of peripheral earth retaining walls are discussed, using the technology of Munich Walls, Berlin Walls, and Mixed Walls, duly adapted to the existing constraints. Finally, the results of the Monitoring and Survey Plan are pointed out, confirming its importance as risk management tool in a work of high geotechnical complexity and located very close to very sensitive structures, as it was ML tunnel.

RÉSUMÉ: Cet article fait référence aux solutions d'excavation et de mur de soutènement pour un développement résidentiel situé à Lisbonne, où une partie de la structure du sous-sol est située à 4 m au-dessus de la crête du tunnel du métro de Lisbonne (ML). Les contraintes liées à l'exécution des murs de soutènement périphériques sont discutées, en utilisant la technologie des murs de Munich, des murs de Berlin et des murs mixtes, dûment adaptée aux contraintes existantes. Enfin, les résultats du plan de surveillance et d'arpentage sont soulignés, confirmant son importance en tant qu'outil de gestion des risques dans un ouvrage de grande complexité géotechnique et situé très près d'ouvrages très sensibles, comme il s'agissait d'un tunnel ML.

Keywords: Munich Walls, Berlin Walls, Mixed Walls, Deep Excavations

1 INTRODUCTION

This paper describes the excavation and peripheral retaining walls solutions, designed and executed in a residential development, in Lisbon, whose excavation bottom level was about 4m from the vault of the Lisbon Metro (ML) tunnel, corresponding to the section of the Red Line, located between the stations of Moscavide and Encarnação, executed using NATM technology. The work studied includes two buildings (Plot 1 and Plot 2), with Plot 1 consisting of 8 upper floors and 2 underground basements and Plot 2 consisting of 8 upper floors and 3 buried basements. Figure 1 shows an aerial view of the location of the site.

This article aims to address in more detail the solutions of Mixed Berlin (peripheric wall where the upper side is as munich walls (concrete) and lower side is as Berlin walls (timber lagging), with soldiers piles connecting both type of wall), and Suspended Munich walls (supported by adjacent peripheric walls), conditioned by the proximity of sensitive structures, such as the ML gallery.

Figure 1 - Site Plan and ML path.

There was a need to resort to studies of non-conventional solutions considering the regulations and standards of the ML, which does not allow the execution of structural and foundation elements within 3m of the ML.

2 CONSTRAINTS

Given the location of the ML tunnel, about 4m below the entire North peripheral retaining wall of Plot 1, it was not possible to execute the traditional Munich Walls (as Figure 2). Likewise, in Plot 2, in an extension of about 20m, it was not possible to execute piles or micropiles, due to the need to respect the precautionary distance of 3m to the ML tunnel.

According to ML documents (Metropolitano de Lisboa 2010a), in addition to the need to respect the precautionary distance of 3m, during the excavation and peripheral containment works, the following criteria of maximum deformation were also imposed, to be proven by the instrumentation:

- The structure of the ML tunnel could not suffer displacements greater than 7 mm vertically and/or horizontally;
- On the ML rails, it was not possible to verify a variation of relative displacement, between sections of 8m distance, greater than 3mm.

These rules are applied to any excavation that takes place within 25m of the ML tunnel, which was the case (Metropolitano de Lisboa 2010b).

3 CONCEPTUAL SOLUTIONS

3.1 PLOT 1

The conceptual solution for Plot 1 North area was with the execution of a Mixed Berlin Wall (Figure 2 and 3). In this solution, a Munich wall was executed in the top area of the retaining wall and a Berlin wall was executed in the bottom area, because the space between the mixed berlin walls and the definitive wall of the building was to be fille (as Figure 3).

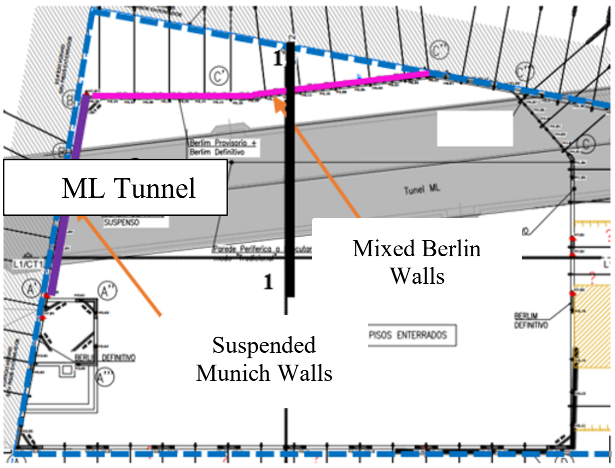


Figure 2 - Plot 1 – Conceptual solution.

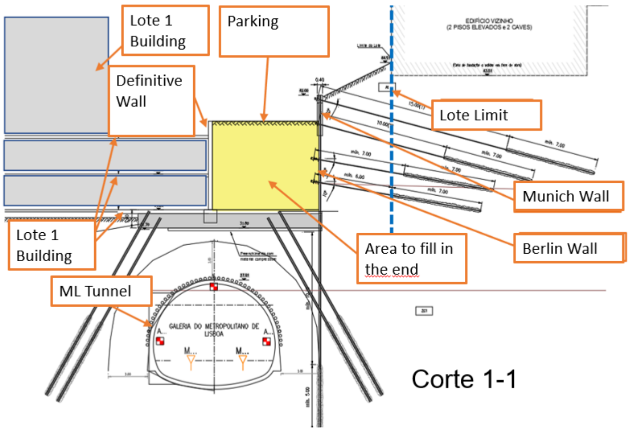


Figure 3 - Plot 1 – section 1.

3.2 PLOT 2

The conceptual solution for Plot 2 North area was with the execution of a Suspended Munich Wall (Figures 4 and 5). In this solution, the micropiles in the suspended Munich wall are in tension, to transmit the vertical load from the wall to the top beam (with 2m height). Due to the suspended load, reinforcement of the micropiles in the end of Munich wall is required, to hold all vertical load from the suspended Munich wall.

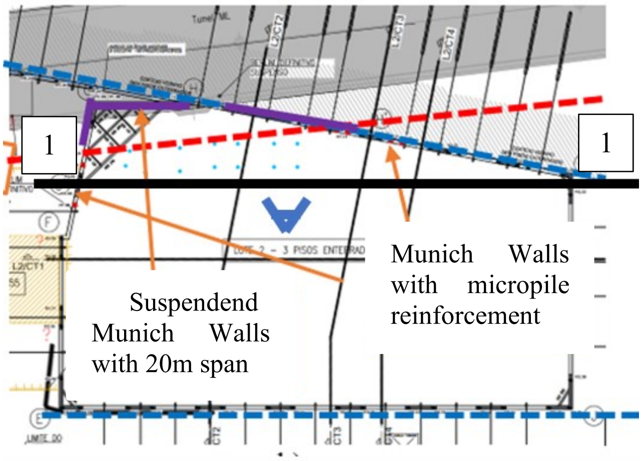


Figure 4 - Plot 2 – Conceptual solution.

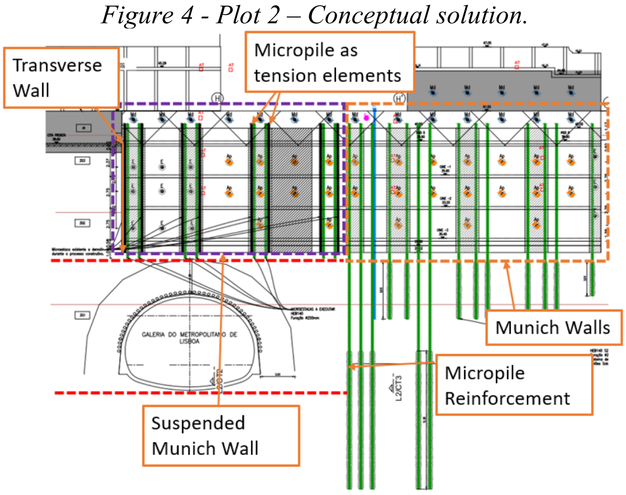


Figure 5 - Plot 2 – section 1.

4 EXECUTION

4.1 PLOT 1

Figures 6 and 7 show the site at completion of the excavation. Figure 7 shows the beginning of execution of the permanent wall of the building and the space (min 3m) to the Mixed Berlin Wall.

Figure 6 - Plot 1 – Mixed Berlin Wall.

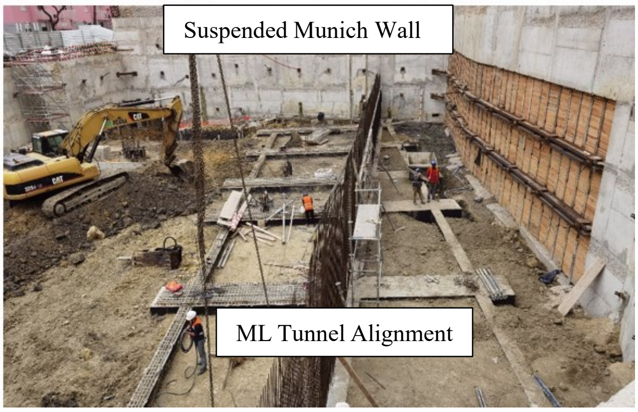


Figure 7 - Plot 1 – Suspended Munich Walls.

4.2 PLOT 2

Figure 8 show the suspended Munich wall solution at bottom excavation. To reduce vertical load, 3 levels of props were adopted.

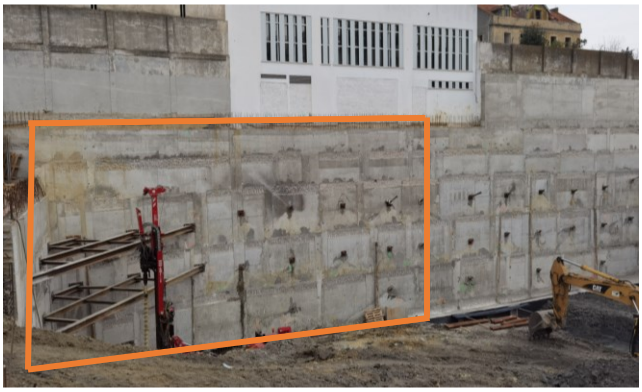


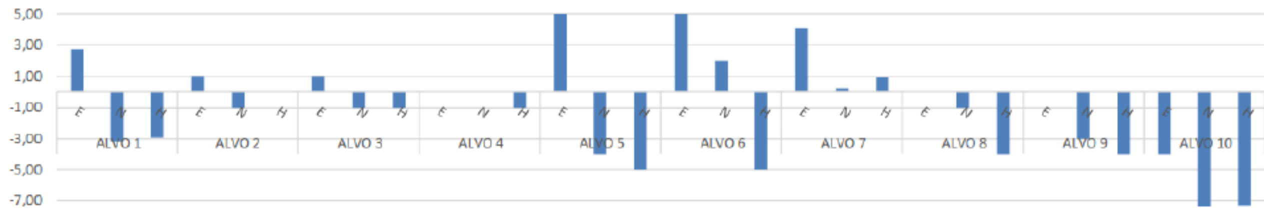
Figure 8 - Plot 2 – Suspended Munich Walls.

5 MONITORING AND SURVEY PLAN

The instrumentation adopted in situ, consisted of a wide range of devices, as the following:

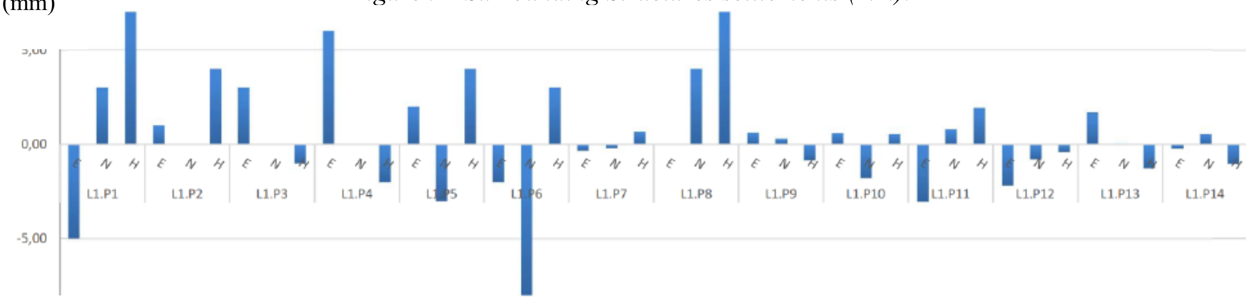
- Topographic targets in the peripheral retaining structures, placed during their execution;
- Topographic targets in the surrounding structures, installed before the start of excavation work;
- Piezometers, installed to control the position of the water table;
- Inclinometers, installed to control the evolution of horizontal displacements of the ground;
- Load cells, used to evaluate loads on temporary anchors;

Topographic targets and topographic marks installed inside the ML gallery to control the relative (convergences) and absolute deformations of the vault and rails, respectively. Figures 9 to 13 show the results.



(mm)

Figure 9 - Surrounding Structures settlements (mm).



(mm)

Figure 10 - Plot 1 Peripheric wall settlements (mm).

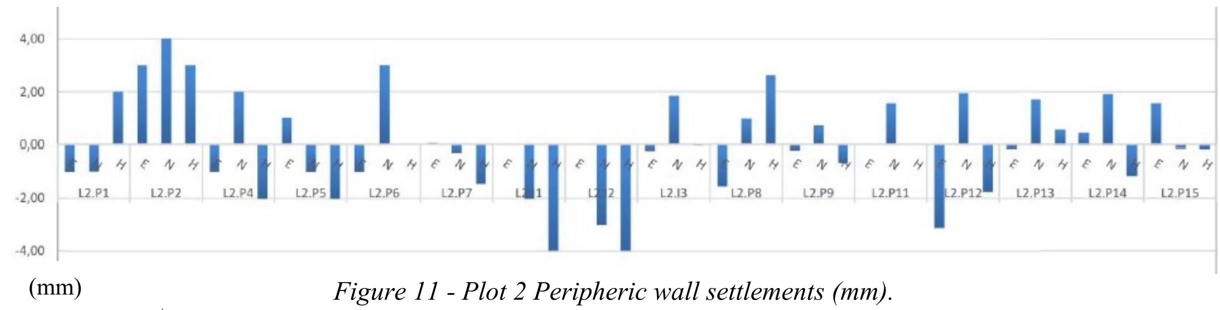


Figure 11 - Plot 2 Peripheric wall settlements (mm).

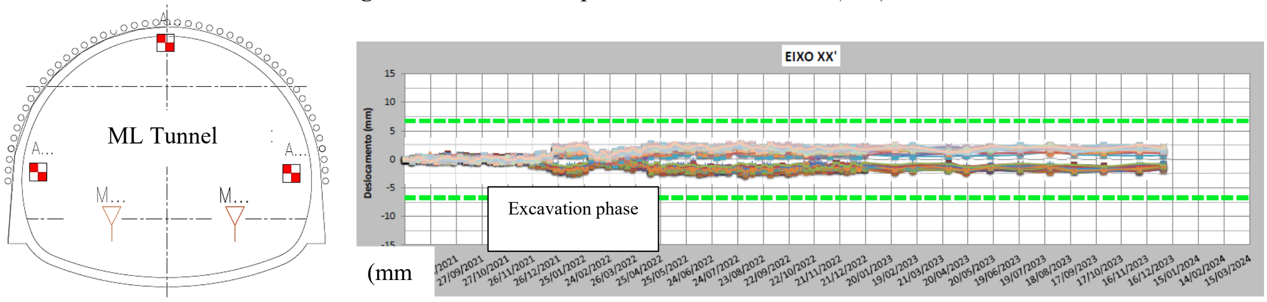


Figure 12 - ML Tunnel – Topographic longitudinal settlements of the vault (mm).

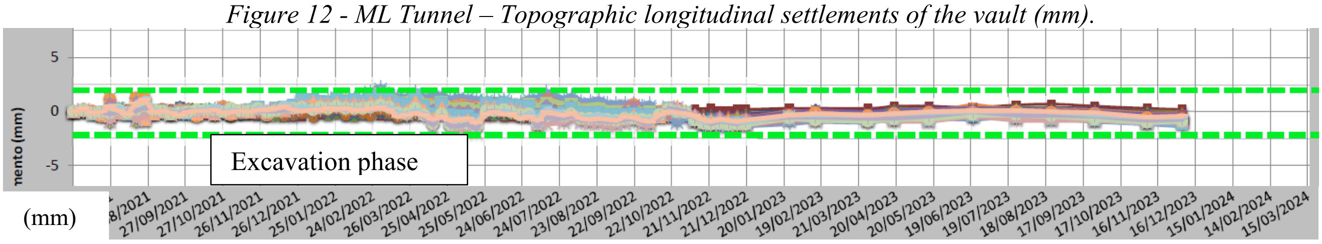


Figure 13 - ML Tunnel – Rails vertical settlements (mm).

Settlement result of the peripheric retaining wall suggest good correlation with the model considered in the design. Settlement result of surrounding structures were compatible with the profile of those structures. ML tunnel settlement was lower than max acceptable value. Rails settlements were at the max acceptable value (3mm) during excavation phase.

6 CONCLUSIONS

This article highlights the main solutions of peripheric earth retaining walls in a sensitive geotechnical area, due the vicinity of the ML Tunnel. Risk management, soil behaviour with good correlation with design models and assessment of good behavior of the structures were only possible with the execution of a Monitoring and Survey Plan.

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Deep Excavation Solutions in an Urban Environment - Distrikt Residential Project, Lisbon

Solutions de Excavation Profond en Milieu Urbain – Projet Résidentiel Distrikt, Lisbonne

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ABSTRACT: This paper describes the main design and execution criteria adopted for the peripheral earth retaining solutions, required for the construction of three underground floors for the Distrikt Residential Buildings, located in the south of Parque das Nações, in Lisbon. The excavation, with 11 m of maximum depth, mainly intersected landfill layers and Lisbon Miocene soils (Areolas do Cabo Ruivo e Areolas do Braço de Prata). The excavation site is confined by several underground infrastructures, which are mostly inside a technical gallery, close to the excavation pit boundaries. It is also noteworthy the proximity to two buildings with several basements, as well as the possibility that the excavation can coexist with the construction of an underground car park, under Rua Mário Botas. To minimize the ground disturbance, as well as the deformations of the mentioned structures, a reinforced concrete bored pile wall with 600mm diameter was defined, braced in general by steel props and temporary ground anchors. In the elevations where the neighbourhood conditions didn't allow the execution of ground anchors, the retaining wall was braced by concrete slab's strips, which were connected to pile buttresses and supported by HEB200 micropiles. The main results of the monitoring plan are drawn and presented, including the analyses and comparison with the estimated values from the design phase.

RÉSUMÉ: Cet article décrit les principaux critères de conception et d'exécution adoptés pour les solutions de retenue périphérique du sol nécessaires à la construction de trois étages souterrains pour les bâtiments résidentiels Distrikt, situés au sud du Parque das Nações, à Lisbonne. L'excavation, d'une profondeur maximale de 11 mètres, a principalement traversé des couches de remblai et des sols miocènes de Lisbonne (Areolas do Cabo Ruivo et Areolas do Braço de Prata). Le site d'excavation est confiné par plusieurs infrastructures souterraines, principalement à l'intérieur d'une galerie technique, à proximité des limites de la fosse d'excavation. Il est également à noter la proximité de deux bâtiments avec plusieurs sous-sols, ainsi que la possibilité que l'excavation puisse coexister avec la construction d'un parking souterrain, sous la rue Mário Botas. Pour minimiser la perturbation du sol, ainsi que les déformations des structures mentionnées, un mur de palplanches en béton armé de 600 mm de diamètre a été défini, renforcé en général par des étais en acier et des ancrages temporaires. Aux endroits où les conditions du quartier ne permettaient pas l'exécution d'ancrages au sol, le mur de soutènement était renforcé par des bandes de dalle en béton, reliées à des contreforts de pieux et supportées par des micropieux HEB200. Les principaux résultats du plan de surveillance sont présentés, y compris les analyses et la comparaison avec les valeurs estimées de la phase de conception.

Keywords: Earth retaining walls; Pile walls; Slab bands; Buttress; Temporary ground anchors.

1 INTRODUCTION

The present article describes the solutions adopted for the excavation and peripheral soil containment, necessary for the construction of the 3 underground floors of the Distrikt enterprise. The enterprise is located on plot 3.22, located between Rua do Adeus Português, Av. Fernando Pessoa, and Rua dos Argonautas, in the southern area of Parque das Nações.

The project envisages the construction of 4 towers with 13 elevated floors and 3 common underground floors, which occupy the total area of the plot and are expected to be used for car parking and technical areas.

Figure 1 displays an aerial perspective of the intervention site.

Figure 1 – Virtual view of the future residential building´s.

Following similar work carried out on excavations in urban environments (Carvalho and Pinto (2019), Tomásio and Pinto (2019) and Pinto et al. (2017), the importance of adopting excavation and peripheral soil containment solutions duly compatible with the project´s framework and various existing constraints, without compromising the safety and proper functionality of the project and neighbouring structures and infrastructures is empathized, as it will be presented in the following chapters.

2 MAIN CONSTRAINTS

2.1 Geological and geotechnical conditions

To characterize the geotechnical behaviour, associated with the soils present at site, a prospecting campaign was conducted involving the execution of 16 mechanical boreholes accompanied by Standard Penetration Tests (SPT) and the collection of samples for macroscopic classification. Additionally, 5 hydraulic piezometers were installed in some of the boreholes. According to the results obtained, the geological structure of the site is characterized by the presence of Miocene substrates corresponding to Areolas do Cabo Ruivo (M_{CR}) and Areolas do Braço de Prata (M_{BP}), overlaid by modern anthropogenic materials, Landfill Deposits (At), with a thickness ranging from 1.5 m to 7.9 m. The Miocene formations are mainly composed of fine, silty sands with carbonate nodules or fossil remains. This units have a considerable thickness, reaching the end of the boreholes. Bio-calcarenite levels (pebble beds) with variable thicknesses ranging from 0.1 m to 2.0 m were also identified.

In the sample collection, it was identified the contamination of the uppermost layers of fossiliferous limestone (pebble beds), revealing the presence of hydrocarbons in 6 of the boreholes conducted, all located on the north of the plot.

In terms of hydrogeology, the Miocene layers function as a multilayer system, consisting of alternating permeable layers (sands, sandstones, and some limestones) with other semi-permeable layers, such as clays and siltstones. Therefore, the higher the percentage of fines (silt and clay), the lower the permeability. On the other hand, bio-calcarenite layers are often highly permeable due to their high porosity and can constitute more productive layers.

2.2 Neighbourhood conditions

The excavation site, bordered by roads, infrastructures, and buried networks, is notably close to the Parque das Nações technical gallery, where diverse service infrastructures are established. The integrity and operability of these infrastructures must be maintained during construction work, thus influenced the design of peripheral earth retainment solutions.

During the project´s development, excavation works could potentially occur simultaneously with the construction of the underground levels of the parking lot to be developed immediately adjacent to the plot. In this context, and for this elevation, it had to be defined interior bracing solutions that did not require occupying neighbouring plots. In the following figure it is possible to observe the main boundaries of the excavation site.

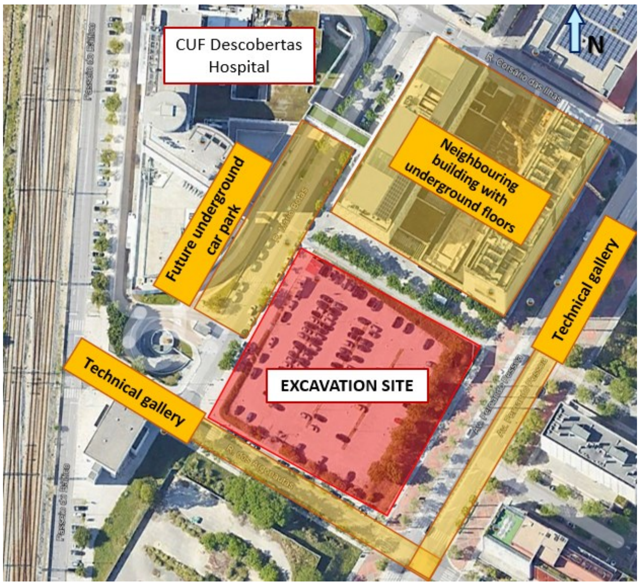


Figure 2 – Main boundaries of the excavation site.

3 EXCAVATION AND PERIPHRAL EARTH RETAINING SOLUTIONS

In the design of the adopted peripheral earth retaining solutions, the following fundamental principles were sought to be respected, beyond the necessary horizontal retainment of the excavated grounds:

- Control soil deformations and surrounding structures and infrastructures to the excavation, allowing the adaptation to possible geological and geotechnical singularities;
- Ensure minimal interference with all adjacent structures and infrastructures.

Considering the existence of several conditioning factors, a reinforced concrete bored pile wall with 600 mm diameter piles and spaced 1.20 m, was defined. The bored pile walls were constructed using a telescopic Kelly bar technology, with temporary casing employed only at the top of the terrain to penetrate through the fill layers, when necessary.

During the excavation phase, the exposed soil between bored pile walls was generally protected with sprayed concrete with a minimum thickness of 8 cm, reinforced with metallic fibers, applied in two layers and properly drained through drainage pipes. These devices were served to collect and conduct water to the building's drainage system through vertical drainage pipes installed at the back of the wall and between the bored piles. In the regions where there was a higher probability of increasing locally the permeability and/or contamination of the soil, it was constructed a permanent reinforced concrete wall with 25 cm thickness, properly integrated to the bored pile walls.

The reinforced concrete bored pile wall was braced, in general, by multiple levels of steel props and pre-stressed ground anchors, to ensure the horizontal balance of the provisional earth retaining structures.

To ensure a better distribution of the forces along the curtain wall, as well as to prevent the excessive concentration of loads, the temporary ground anchors and steel props were connected to distribution concrete beams and crown beams. In the definitive phase, the stability of the curtain wall is provided by concrete slab strips, which will integrate the definitive concrete slabs of the underground floors, and the temporary ground anchors and steel props are deactivated after the completion of the definitive structure.

In elevations where the presence of neighbouring infrastructures or structures constrained the execution of ground-anchors, a bracing system was defined, consisting in reinforced concrete slab bands, which function has horizontal beams (with the section of the future underground floors). The slab bands offer high stiffness to the earth retaining wall, resisting and transmitting the impulses acting on the curtain wall directly to the buttresses, which were also formed by bored piles. Considering the excavation depth, bracings were implemented at levels -1 and -2, with the initial excavation section carried out under a curtain wall that will be functioning as a cantilever. The design of the structural elements considered not only the necessary strength and stiffness to ensure safety and proper functionality but also the construction process associated with both the excavation and the execution of the underground floors. Figure 3 and 4 shows 2 plan views of the peripheral earth retaining structures and the respective shoring elements at each underground floor level.

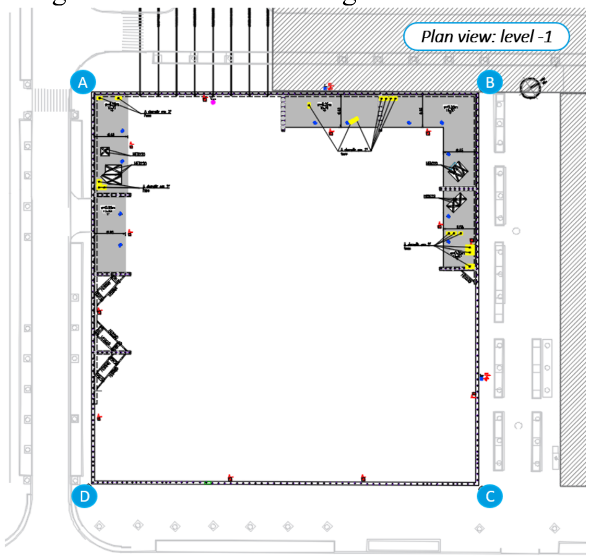


Figure 3 - Design plan view of the peripheral earth retaining solution at level -1.

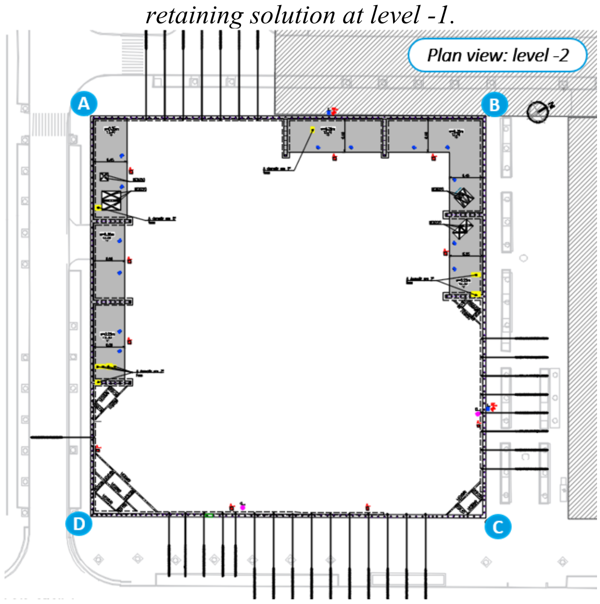


Figure 4 - Design plan view of the peripheral earth retaining solution at level -2.

The ground anchors comprised 6 and 7 strands, each with a diameter of 0.6 inches, spaced at intervals of 3.60 m. A tensile load ranging from 600 kN to 800 kN was applied to these anchors, which varied in both length and inclination. The inclinations of the anchors were specifically determined to prevent potential intersections with existing infrastructures and structures. Additionally, these inclinations facilitated the execution of the primary grouted bulbs in geologically stable soil concerning the excavation´s geometry (Figure 5). The bonded lengths were accomplished using the IRS system, incorporating double obturators and non-return valves, with a minimum hole diameter of 200 mm.

Figure 5 - Bored pile wall shored temporarily by ground-anchors.

The configuration of the slab sections was defined with the aim of minimizing interference with the construction of the pillars and other structural elements within the ground floors. The slab bands exhibited an average width and thickness of 6.50 m and 25 cm, respectively. During the excavation phase, the slab bands were supported on the retaining walls and on HEB200 profiles, spaced approximately 6.50 m apart (Figure 6). These profiles were bonded for a length of 2.0 m within sections of Ø600 mm piles, reaching a depth of 4.0 m below the excavation´s bottom. The execution of the slab strips took place during the excavation phase, involving concreting against the ground over a layer of levelling concrete and plastic film.

Figure 6 - Concrete slab strips supported by HEB200 micropiles and buttresses.

To ensure the bored pile wall essential´s drainage conditions, sub-horizontal drainage pipes were installed between the curtain´s piles, measuring 4 m in length and 50 mm in diameter. These pipes were made from high density polyethylene (HDPE), covered in a geotextile with a density of 150 g/m^2. It´s primary function was to facilitate internal drainage within the massif, thereby averting potential hydrostatic impulses resulting from rainwater infiltration. To guarantee the gravitational flow of collected water, these drainage elements were positioned with an upward slope of 10º from the horizontal and with a horizontal spacing of 3.60 m.

The water collected by drainage pipes was directed into the peripheral rain gutter, which ran parallel to the retaining wall. Below the excavation bottom level, there was no interference with the hydrogeological system, as the piles, being discrete and spaced elements, did not form a barrier to the water flow.

4 GEOTECHNICAL AND STRUCTURAL DESIGN

The assessment of the presented peripheral earth retaining solutions in terms of forces and deformations was conducted for all major construction phases using finite element programs such as PLAXIS 2D and 3D, specifically designed for this purpose. A constitutive model was developed to simulate the soil behaviour characteristics of a "Hardening Soil", considering a nonlinear constitutive relationship and variations in the soil's stiffness under applied stress states. The parameters outlined in Table 1 were applied in PLAXIS to model the soils.

Table 1 - Geotechnical soil parameters.

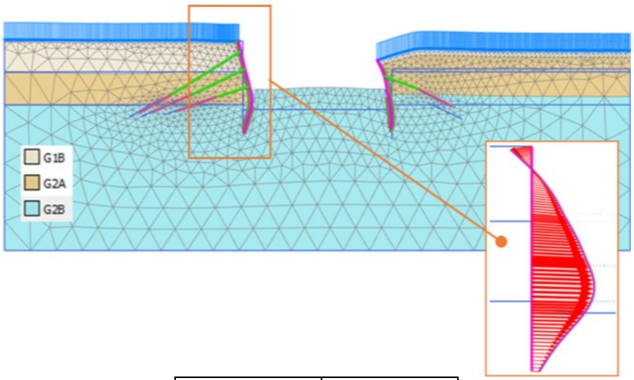
Soil Type	ZG_{1A}	ZG_{1B}	ZG_{2A}	ZG_{2B}	ZG_{2C}
N_{SPT}	<10	10-30	10-30	30-60	60
Υ [kN/m^3]	18	19	19	20	21
Φ [º]	27	31	32	33	36
c´ [KPa]	-	-	-	-	-
E_{50}^{ref} [MPa]	8	20	40	50	200
E_{ur}^{ref} [MPa]	24	60	120	150	600
m [-]	0,5	0,5	1	1	1
R_f [-]	0,9	0,9	0,9	0,9	0,9

ZG_{1A} – *Loosely compact sandy-clay fill*; ZG_{1B} – *moderately compact sandy-clay fill;* ZG_{2A}– *moderately compact sandy-silt soil;* ZG_{2B} – *compact sandy-silt soil;* ZG_{2C} – very *compact sandy-silt soil.*

The study of the reinforced concrete bored pile wall sections braced by slab bands, steel struts or temporary ground anchors was conducted through the development of a Plaxis 2D model. In contrast, due to the distance of the buttresses and the interaction of the slab bands with these elements, an analysis of this solution was performed using a Plaxis 3D model.

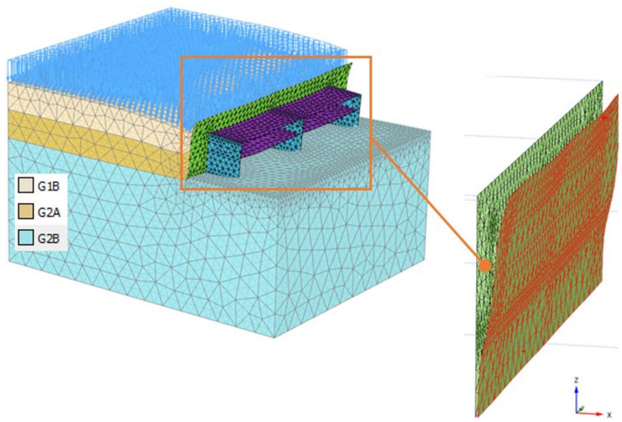
In the developed models, the curtain wall´s, slab bands and buttresses were represented using "plate" elements with elastic behaviour, while the ground anchors were modelled as "node-to-node anchor" elements, with the respective bond lengths modelled using "embedded beam row" elements.

The behaviour of the peripheral retainment structure was analysed for the main excavation phases, assessing crucial design parameters such as forces of the containment structures, deformations, stress states and stability of the soils to be contained. Figures 7 and 8 present some results from the developed models in terms of displacements.



u	u_x
18,40 mm	*8,68 mm*

Figure 7 – Total displacements (u) and horizontal displacements (u_x), expected at the excavation´s base level of the curtain wall, braced by temporary pre-stressed ground anchors.



u	u_x
56,34 mm	*55,54 mm*

Figure 8 -Total displacements (u) and horizontal displacements (ux), expected at the excavation´s base level of the curtain wall, braced by slab bands and buttresses.

5 MONITORING AND SURVEY PLAN

Based on the complex framework of the site, an Monitoring and Survey Plan (MSP) was established with the aim of ensuring safety conditions and optimal functionality during the excavation and the execution of the geotechnical structures, as well as neighbouring structures and infrastructures. In this context, the following instruments were installed:

- Topographic targets distributed along the bored pile walls and edges of the slab bands;
- 4 load cells to measure the pre-stressed load applied to the temporary ground anchors;
- 2 inclinometers positioned behind the peripheral containment walls;
- 2 piezometers to measure underground water pressure;
- 1 seismograph for vibration control during the excavation of areas with higher resistance.

Based on the conducted modelling, alert and alarm criteria was established for all instruments and monitored structures. Overall, readings consistently remained below the alert and alarm criteria defined in the project. In Figure 9, the schematic location of the instrumentation devices and some of the most notable results are presented.

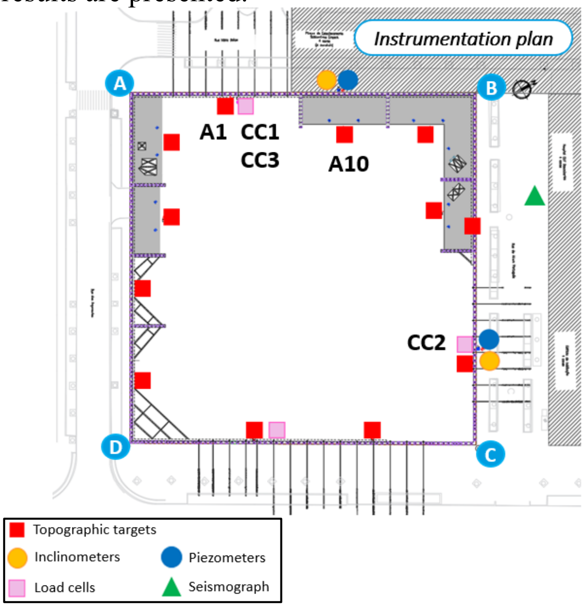
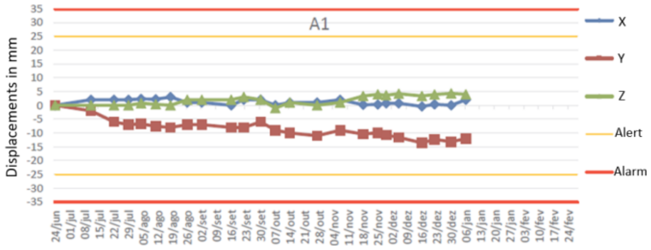
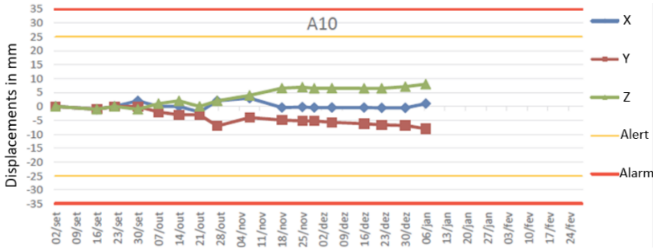


Figure 9 – Instrumentation and monitoring plan.

As observed in the examples below, deformation values consistently remained below the alert criteria outlined in the project (yellow line in the graph), as shown in Figure 10. Regarding the monitoring of load cells, a minor adjustment to the initial load applied to the anchor tendons was noted, facilitated by a hydraulic jack. Subsequently, the tension values remained nearly constant throughout the construction process, confirming the satisfactory performance of the project (Figure 11). These conclusions are applicable to the remaining instruments.



(a)



(b)

Figure 10 – Displacement results on station A1(a) and A10(b).

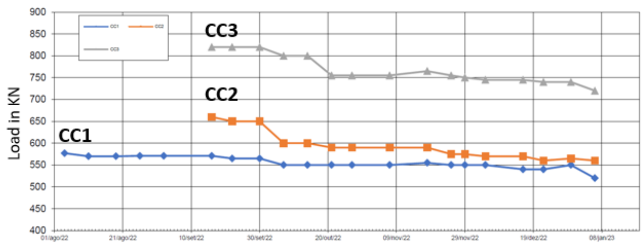


Figure 11 – Load evolution on ground-anchors CC1, CC2 and CC3.

Considering the above, as of the present date with the conclusion of excavation and peripheral containment works, the instrumentation results serve to validate the appropriateness of the implemented solutions and the geomechanical parameters meticulously considered in the modelling. This paves the way for prospective optimization of similar geological and geotechnical solutions in the future.

6 CONCLUSIONS

In the scope of this article, it was possible to demonstrate the technical efficiency of reinforced concrete bored piles braced by various structural elements such as struts, temporary ground anchors and slab bands. The peripheral earth retaining solution presented actual displacements lower than those estimated in project and displayed a highly stable behaviour during the excavation works. Additionally, the use of slab bands not only overcame the encountered limitations due to the excavation´s site proximity to the technical gallery and the future underground parking lot, but also incorporated elements of the final structure, enhancing the economic efficiency of the solutions.

However, there were some on-site challenges that led to adjustments to the initial project solutions. The main challenge arose from an increased water flow into the excavation, given the intense rainfall experienced in December 2022 towards the end of the excavation works. To address this unforeseen circumstance and to ensure the integrity of the sprayed concrete between the piles, in response to the rising water flow into the excavation area, the number of drainage pipes was increased, and temporary water collection zones were established. In the definitive phase, the water collected by the pipes will be directed to the building's general drainage system.

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ECSMGE 24
XVIII EUROPEAN CONFERENCE ON SOIL MECHANICS AND GEOTECHNICAL ENGINEERING

Earth retaining solutions and facades underpinning, for the refurbishment an historic building, in Lisbon

Solutions de soutènement et de repris en sous-œuvre de façades, pour la rénovation d'un bâtiment historique, à Lisbonne

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ABSTRACT: This article presents the earth retaining, underpinning and facades retaining solutions as part of the refurbishments works of the 260 years old Pedrosas (CTT) Palace, in Lisbon. To allow the construction of new basements, under the original foundations of the masonry building, it was necessary to design earth retaining and facades underpinning compatible solutions.The earth retaining solution consists of a reinforced concrete wall, built using the King Post Wall technique, temporary braced by ground anchors and steel struts. The underpinning and the temporary structures of the foundation were carried out using tubular micropiles. The main design criteria are presented, as well as the main monitoring and survey results.

RÉSUMÉ: Cet article présente les solutions de soutènement du terrain, de reprise en sous-œuvre et de maintien des façades dans le cadre des travaux de rénovation du Palais Pedrosas (CTT), avec 260 ans, à Lisbonne. Pour permettre la construction de nouveaux sous-sols sous les fondations d'origine du bâtiment en maçonnerie, il a été nécessaire de concevoir des solutions compatibles de soutènement du terrain et de sous-bassement des façades. La solution de soutènement consiste en une paroi avec la technique de King Post Wall, en béton armé, temporairement stabilisé par des ancrages au sol et des tirants en acier. Les travaux de fondation, de reprise en sous-œuvre et des structures temporaires ont été réalisés à l'aide de micropieux tubulaires.Les principaux critères de conception sont présentés, ainsi que les principaux résultats de surveillance et d'étude.

Keywords: earth retaining wall; underpinning; façade retaining

1 INTRODUCTION

In the scope of the redevelopment of Palácio das Pedrosas (CTT), located on São José Street in Lisbon, it was necessary to carry out a project involving excavation and peripheral earth retaining with facade underpinning. According to the aerial view presented in Figure 1, it is possible to identify the approximate layout of the construction, as well as some of the most relevant boundaries and constraints. The referenced area has boundaries to the north with an existing building that needs to be preserved.

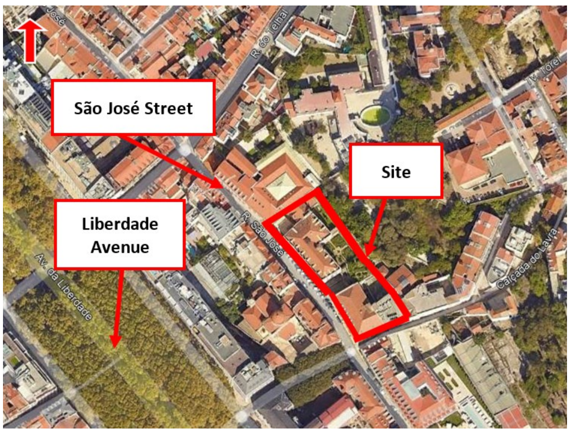

Figure 1 - Site location.

The following figure shows the areas to be excavated beneath the existing structure.

Figure 2 - Areas to be excavated under the existing structure.

As an old building, the solution to be adopted was conditioned by various factors, making this project quite challenging.

2 MAIN CONSTRAINTS

2.1 Constraints related to the building's boundaries

In this intervention, several underground floors will be executed, being adjacent to neighbouring buildings and roads. The proposed solutions must, therefore, be compatible with preserving the integrity of all structures and infrastructure located outside the perimeter of the construction site, ensuring their functionality conditions.

2.2 Constraints related to the need to preserve facades and structural walls.

Given the necessity to maintain the integrity of the majority of the main building facades, it is considered essential that the intervention be carried out in a way that minimizes its impact on their stability and appearance. Equally important was the adoption of construction solutions compatible with the use of equipment suitable for the available spaces and access, allowing the execution of the works while limiting the occurrence of vibrations and noise.

2.3 Geological and geotechnical constraints

According to the data collected during the geotechnical characterization campaign and considering the geology described in the geological map, the following lithological strata were identified on-site:

- Landfills: Detected in the superficial zone of all boreholes, with thickness ranging from approximately 0.20 to 0.50m. They are heterogeneous landfills with masonry fragments, primarily silty-clayey, and brownish.

- Prazeres Clays and Limestones: The Miocene levels are represented by alternating silty and clayey levels. The silty levels sometimes have transitions with fossil fragments and/or marly materials, with greenish to greyish tones. The more clayey levels often appear slightly micaceous and with darker tones. Generally, they exhibit plastic behaviour.

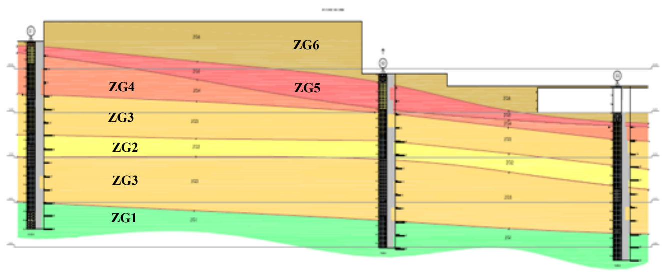

Figure 3 - Soil profile.

Table 1 - Geological and geotechnical parameters

Geotechnical zone	N_{SPT}	Lithology	ϕ' (°)	C' (kPa)	γ' (kN/m3)	E' (MPa)
ZG6	-	Silt-sandy embankments with masonry fragments, all-in-one, concrete, and pavements	5 - 10	-	16 - 17	5 - 10
ZG5	2 - 9	Clayey silts and silty clays	10 - 15	5 - 15	17 - 18	10 - 15
ZG4	16 - 26 (16 - 18)*	Clayey silts and silty clays	15 - 20	15 - 20	18 - 19	20 - 25
ZG3	21 - 56 (21 - 42)*	Clayey silts and silty clays	20 - 25	20 - 30	19 - 20	30 - 40
ZG2	>60	Clayey silts and silty clays	25 - 30	40 - 60	20 - 21	50 - 60
ZG1	>60	Clayey silts, silty clays, calcareous marls/marly limestones	25 - 35	60 - 80	21 - 23	60 - 100

3 ADOPTED SOLUTIONS

The proposed solutions for the execution of facade underpinning and excavation with peripheral earth retaining are interconnected and are as follows:

- Excavation and peripheral earth retaining: to be carried out using the king post wall technology.
- Underpinning and earth retaining of preserved facades: to support the existing facades, it is proposed to install a steel structure for earth retaining and stabilization. The solution of the facade containing structure is beyond the scope of this article.

The proposed solutions considered the necessary coordination between the two types of interventions described.

Concerning the structure of the peripheral earth retaining wall, considering the evaluation of the main constraints, it was proposed that, in general, the excavation of the underground floors be carried out under a peripheral earth retaining using the king post wall technology. The proposed construction technology for the 'king post wall' type earth retaining wall involves the phased execution, from top to bottom, of reinforced concrete panels supported by vertical micro-piles. These micro-piles, materialized by tubular profiles Ø88.9x9.0mm and Ø114.3x9.0mm with external joints, made of high-strength steel (fsyd > 560MPa), will also support the underpinning beams of the facade. These elements should be executed with the minimum possible distance from the facade and walls of neighbouring buildings, solidified through steel brackets.

The total length of the micro-piles will vary based on the elevation geometry, and the minimum of the sealing lengths (calculated by the Bustamante method) to be executed using the IRS system (repetitive and selective injection), employing a double plug and check valves, in competent soils with strength and deformability characteristics compatible with N_{SPT} greater than 35 blows and geologically stable concerning the excavation geometry.

This solution has the advantage of allowing the execution of the permanent wall during excavation. The number of temporary bracings (steel props and anchors), as well as excavation constraints, can be redefined during the construction phase based on the actual characteristics of the excavated soils and the results of the Instrumentation and Observation Plan.

In cases where underpinning the facade is necessary, a beam is required to have the capacity to transmit loads from the facades to the foundations, materialized by the micro-piles.

In the permanent phase, the structure of the slabs and foundation elements will be responsible for the stability of the earth retaining wall, and the temporary props will be removed after the completion of the said structure, and the anchors will be deactivated.

A 3D model is presented in Figure 4, where the existing facade elements to be preserved are represented in light blue, the underpinning beam to be executed in orange, and the earth retaining walls to be constructed for excavation work are shown in grey.

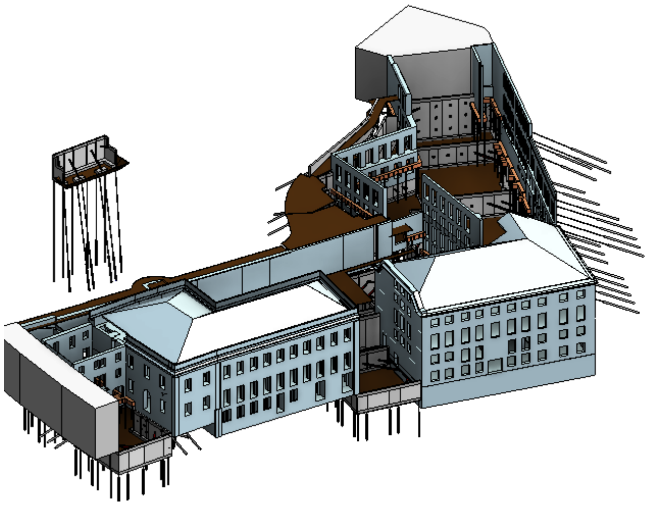

Figure 4 – 3D view of solutions for excavation and peripheral earth retaining.

4 DESIGN

The finite element program Plaxis 2D was used for the design of the peripheral earth retaining walls to be executed using the king post wall technology.

Micro-piles and walls were modelled as "Plate" elements, sealing bulbs were modelled as "Embedded beam row" elements, and pre-stressing tendons were modelled as "Node-to-node-anchor" elements.

The forces are estimated considered the construction process, especially the excavation and construction phases of the peripheral earth retaining wall. Soil elements were modelled considering the previously presented geo mechanical properties and following the "Harding Soil" model.

Figure 5 shows the adopted model. This section corresponds to the maximum excavation height with the highest number of anchors in the whole retaining structure.

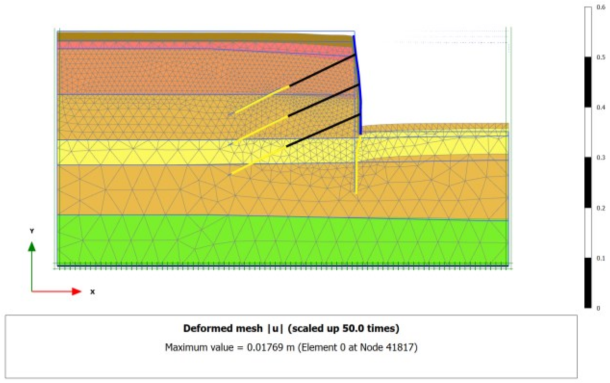

Figure 5 - Total displacements – 1.7cm

After the execution of the interior structure of the building, in the permanent phase, the bracing of the earth retaining is guaranteed by the structure of the underground floors. In the design of the peripheral earth retaining, loads from the building structure were considered. For locations where the micro-pile is located outside the earth retaining wall, it was necessary to verify safety against buckling. In areas where the micro-pile is used simultaneously to support the peripheral earth retaining and for facade underpinning, the additional load from it was considered.

5 MONITORING AND SURVEY PLAN

As the construction is still in the early stages of demolition, the results of the monitoring and survey plan are not yet available. However, the proposed Monitoring and Survey Plan aims to ensure the safe and economical execution of demolition, excavation, and construction of earth retaining structures. It also involves the analysis of the behaviour of preserved facades and neighbouring structures and infrastructure during this construction phase. Thus, the proposed plan should allow the measurement, during excavation and construction of earth retaining structures and the underground floor, of the following parameters:

- Horizontal and vertical displacements of earth retaining structures.
- Horizontal and vertical displacements of neighbouring constructions.
- Measurement of tension/load at ground anchorages.
- Horizontal displacements in neighbouring structures and in the earth retaining.

The parameters described above will be measured using the following:

- Topographic targets for measuring parameters mentioned in a).
- Topographic targets for measuring parameters mentioned in b).
- Load cells in anchorages for measuring the parameter mentioned in c).
- Inclinometers for measuring the parameter mentioned in d).

6 FINAL REMARKS

The purpose of this article was to present the underpinning and peripheral earth retaining solutions adopted in a rehabilitation project to be carried out in the centre of Lisbon, in a densely urbanized area.

One can conclude that each work is unique, and even if the solution is conceptually the same, it should always be adjusted to the existing and exclusive constraints of each work itself. In this case, an attempt was made to define a solution that would better adapt to all mentioned constraints, from both technical and economic issues.

ACKNOWLEDGEMENTS

We thank the Project Owner for the permission to publish this article.

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ECSMGE 24
XVIII EUROPEAN CONFERENCE ON SOIL MECHANICS AND GEOTECHNICAL ENGINEERING

Temporary cofferdam at Caniçada Dam, Portugal

Batardeau temporaire au Barrage de Caniçada, Portugal

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ABSTRACT: The execution of the Caniçada dam new complementary spillway, demanded the construction of a temporary cofferdam to allow the excavation of the discharge tunnel, at the dam south side. The cofferdam with about 20m height and 135m length, intersected weathered to very weathered granite, granite residual soils, fills and sands, with medium average hydraulic conductivity. This scenario led to the execution of a hybrid cofferdam solution: 7m concrete gravity wall over a 30m maximum depth cut-off curtain, using micropiles (tube à manchette) combined with 1m diameter, spaced 0.70m, double row jet grouting columns. The main design and execution criteria are presented, as well as the main QC/QA procedures.

RÉSUMÉ: L'exécution du nouveau déversoir d'inondation complémentaire du barrage de Caniçada, a exigé la construction d'un batardeau temporaire pour permettre l'excavation du tunnel de déchargeur, du côté sud du barrage. Le batardeau, d'une hauteur d'environ 20 m et d'une longueur de 135m, a recoupé des granites altérés à très altérés, des sols résiduels granitiques et des sables, avec une perméabilité moyenne. Ce scénario a conduit à la réalisation d'une solution de batardeau hybride: un mur de gravité, en béton non armée, de 7m de hauteur, sur un écran d' étanchéité de 30 m de profondeur maximale, à l'aide de micropieux (tube à manchette) combinés avec colonnes de jet grouting 1m de diamètre, double rangée et espacés de 0.70m. Les principaux critères de conception et d'exécution sont présentés, ainsi que les principales procédures de CQ/QA.

Keywords: Cofferdam; Jet Grouting; Micropiles; Cut-off

1 INTRODUCTION

The Caniçada dam is a double curvature concrete arch dam with a height of 76m above the foundation level and a crest length of 246m, located at the Cávado river, Peneda Gerês National Park, at Braga District, North of Portugal. The dam was built in 1955 and is resting over the granite bedrock (Figures 1 and 2).

The execution of the Caniçada dam new complementary spillway, demanded the construction of a cofferdam to allow the excavation of the new discharge tunnel portal, at the left abutment of the dam. With this objective, the cofferdam was built with about 20m height and 135m length using a hybrid solution: a concrete gravity wall over a cut-off curtain constituted by jet grouting columns (type 2) and micropiles.

Figure 1. Caniçada dam location.

Figure 2. Construction of Caniçada dam reinforced concrete arch over granite bed rock (courtesy of Rodio).

2 GEOLOGICAL SCENARIO

The geological scenario at the Caniçada dam new complementary spillway is very heterogeneous. After an intensive geological and geotechnical campaign, the following four geotechnical zones were established, from the surface (Figure 3 and 4):

- ZG4 – Fluvial beach sands and heterogeneous fills, with medium to high average permeability.
- ZG3 – Granite residual soil with boulders, with medium permeability.
- ZG2 – Granite rock mass (W3-W4 and F3-F2 to F3-F4) with low average permeability.
- ZG1 – Granite rock mass (W2-W3 and F2-F3).

Figure 3. View of granite boulders at the Caniçada slopes.

3 COFFERDAM SOLUTION

Considering the geological and hydrogeological complexity, as well as both cofferdam geometry and temporary nature, a cut-off curtain solution, combining jet grouting columns (Croce and Modoni, 2007) and injection micropiles (tube à manchette) with 30m maximum depth was built. A concrete gravity wall was built over the cut-off curtain with its crest level 0.5m above the dam maximum water level.

At the cofferdam most representative cross section (ZG3 intersection), the following solutions were adopted, with the double function of cut-off curtain and gravity wall foundation (Figures 5, 6 and 7):

- At the downstream side: two rows of ϕ1000mm jet grouting columns, spaced, respectively, 0.70m and 0.80m in the longitudinal and transversal alignments, with triangular distribution, were built. The micropiles were installed at the jet grouting columns intersections with sleeves at each 1m.
- At the upstream side: one alignment of inclined micropiles (HEB160 steel profiles) and vertical micropiles (HEB140 steel profiles), both spaced 1.6m.

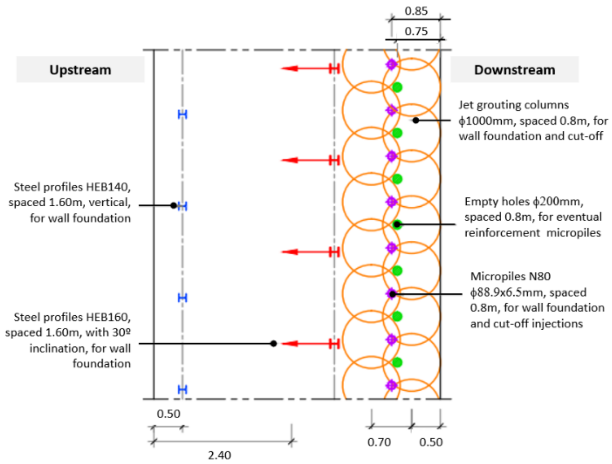

Figure 5. Plan distribution of HEB profiles and cut-off jet grouting and micropiles at the gravity wall foundation.

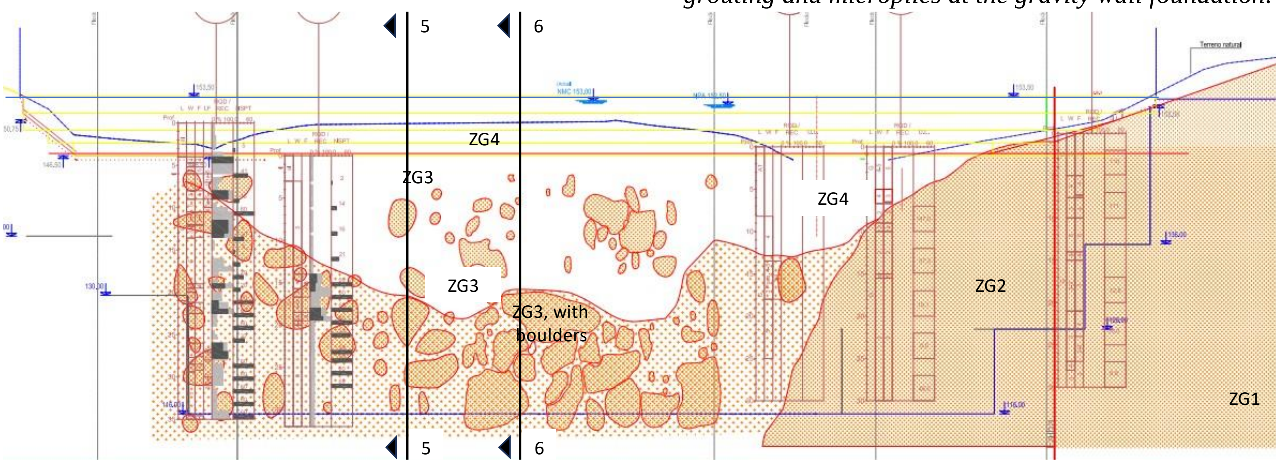

Figure 4. Geological profile (from upstream).

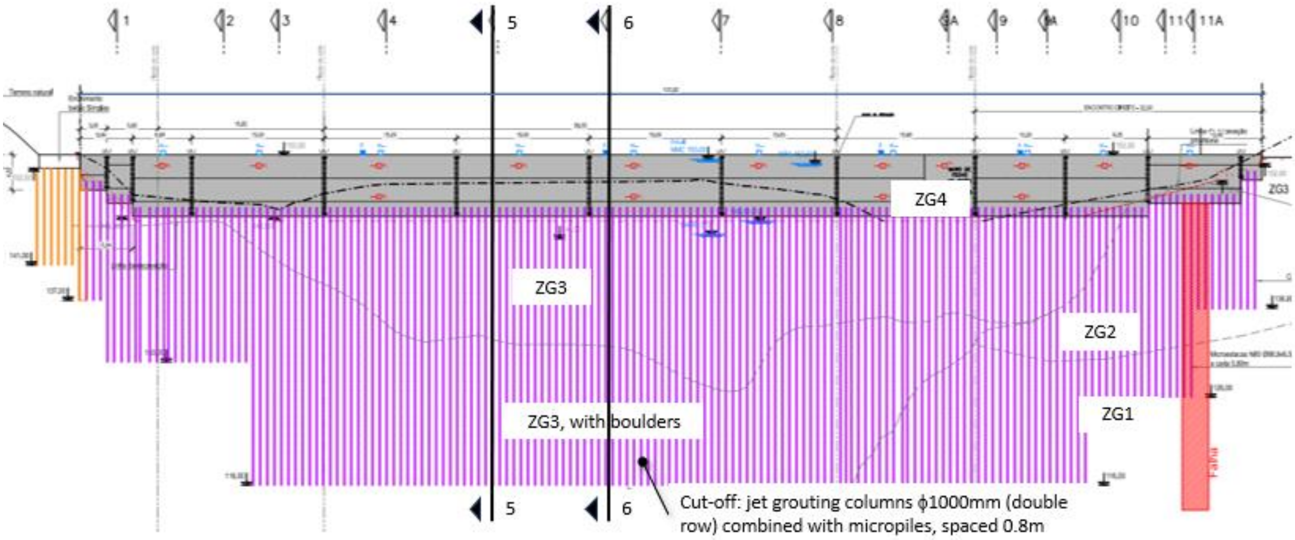

Figure 6. Cofferdam solution elevation (from upstream).

The excavation works inside protected by the cofferdam were performed with 1.25(h):1.00(v) slopes lined with shotcrete, nailed, and drained (Figures 7 and 8).

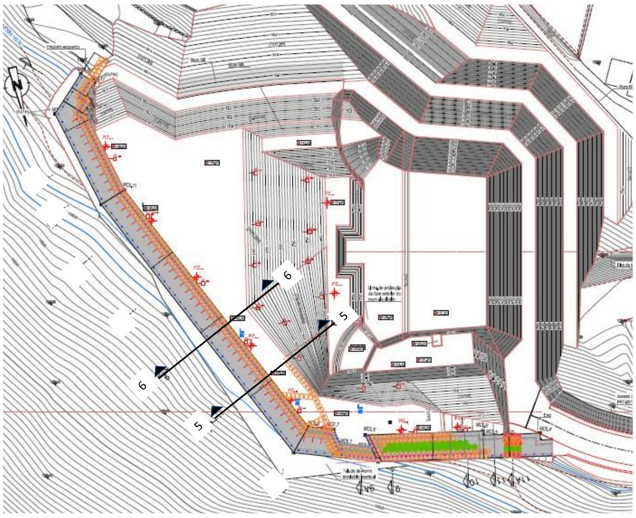

Figure 7. Cofferdam plan with the excavation slopes.

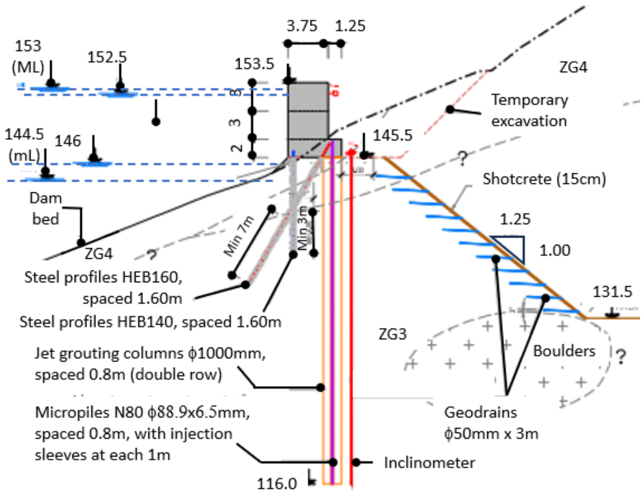

Figure 8. Cofferdam cross section 6-6 (ZG3 intersection).

4 DESIGN

4.1 Cofferdam overall stability

The cofferdam overall stability, including the excavation slope, was checked for several cross sections using SLIDE software, FEM with seepage analysis (Figure 9).

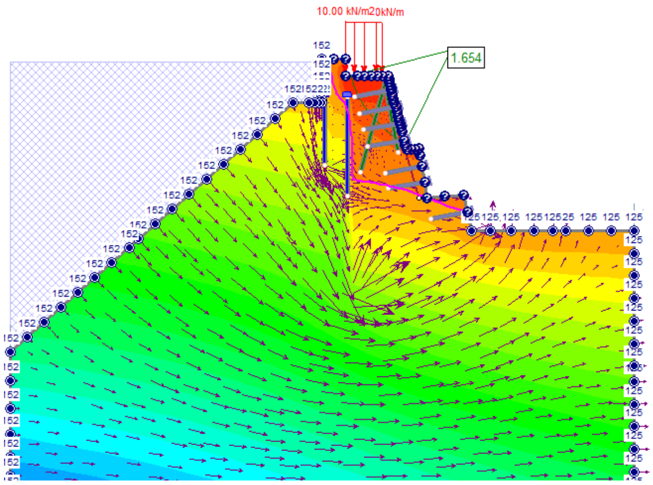

Figure 9. Safety factor for global stabiliy (1.654) at the final excavation level, considering the water inflow, cross-section 5-5.

4.2 Cofferdam hydraulic stability

The cofferdam hydraulic stability, mainly against internal erosion at the excavation base, was also checked for several cross sections using SLIDE software, FEM with seepage analysis. The safety factor for internal erosion was estimated considering the ratio between the critical hydraulic gradient (approximately 1.0) and the estimated hydraulic gradient at the excavation base exit (Figure 10).

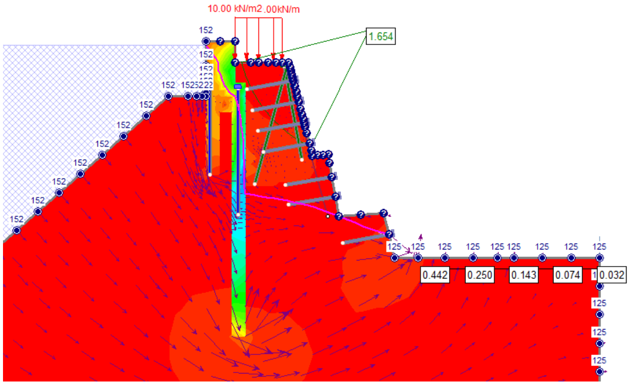

Figure 10. Hydraulic gradients at the excavation base, cross section 5-5.

5 JET GROUTING QUALIY CONTROL AND QUALITY ASSURANCE

5.1 Trial columns

Trial columns were performed to check both the columns geometry and strength at the ZG4 and ZG3. The columns strength and deformability (Young's modulus) were confirmed through UCS tests, performed at 28 days cores and compared with the design parameters: 4MPa and 1GPa, respectively, for 450kg/m^3 of cement consumption (Figure 11).

Figure 11. Trial column's view.

5.2 Execution parameters

All jet grouting execution parameters were recorded using automatic devices and compared with those established based on the trial columns.

5.3 Columns position and verticality

All jet grouting columns position and verticality were confirmed using GPS and digital inclinometer devices.

5.4 Columns integrity and permeability

Jet grouting columns integrity was checked by excavation (Figure 12), coring about 5% of the columns, sounding, cross- hole. Lugeon tests were carried out to control the efficiency of the cofferdam and verify the need of complementary solutions to address watertightness.

Figure 12. View of jet grouting excavated around columns.

6 FINAL REMARKS

The presented case study allowed to confirm the advantages of water cut-off solutions for temporary cofferdams using jet grouting columns combined with micropiles in complex geological and hydrogeological scenarios, even considering the complementary solution of secant bored piles to reduce inflows in the left abutment area of the cofferdam.. As main advantages, compared with more conventional solutions, such as diaphragm walls, can be pointed out the following: minimum ground extraction, behaviour predictability and ease demolition (Figure 13).

Figure 13. View of the excavation final works.

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Foundation solutions near Trancão river, in Lisbon

Solutions de fondations pour les travaux situés près de la rivière Trancão, à Lisbonne

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ABSTRACT: This paper describes the foundations solutions for the main stage for the World Youth Day in Lisbon 2023. The stage is located on an urban solid waste landfill that is sealed through a PEAD geomembrane, which made it impossible to use indirect foundations at an early phase. Underlying the deposits of municipal solid waste, emerges the alluvial formation composed of compressible soils, with weak mechanical characteristics, with variable thickness. In order to reduce the amplitude of differential displacements that could damage the structure, a preload landfill was carried out, in order to minimize the settlements that would occur at the time of the stage installation. During the time of permanence of the preload landfill, several reading campaigns of the instrumentation (topographic marks) were carried out in order to validate the initial assumptions. From the data collected from the instrumentation as well as from the tests carried out on site, the solution of the foundation for the stage was defined. The solution for the cover foundation was defined using ductile iron micropiles driven using the dry method, reinforced with self-drilling rod in the center, properly sealed in the miocene.

RÉSUMÉ: Cet article décrit les solutions de fondation pour la scène principale de la Journée Mondiale de la Jeunesse à Lisbonne en 2023. La scène est située sur une décharge urbaine scellée par une géomembrane PEAD, rendant impossible l'utilisation de fondations indirectes à une phase précoce. Sous les dépôts de déchets solides municipaux, émerge la formation alluviale composée de sols compressibles, aux caractéristiques mécaniques faibles, avec une épaisseur variable. Afin de réduire l'amplitude des déplacements différentiels susceptibles d'endommager la structure, un remblai de précontrainte a été réalisé pour minimiser les tassements qui se produiraient lors de l'installation de la scène. Pendant la période de permanence du remblai de précontrainte, plusieurs campagnes de lecture de l'instrumentation (repères topographiques) ont été menées afin de valider les hypothèses initiales. À partir des données collectées à partir de l'instrumentation ainsi que des tests réalisés sur place, la solution de fondation pour la scène a été définie. La solution pour la fondation de la couverture a été définie en utilisant des micropieux en fonte ductile en utilisant la méthode sèche, renforcés avec une tige autoperceuse au centre, correctement scellée dans le miocène.

Keywords: Micropiles; foundation solutions; preload landfill

1 INTRODUCTION

For the World Youth Day event, a stage had to be set up in Parque Tejo, Lisbon. However, a challenge arose because the selected location was on a landfill of urban solid waste. Between this landfill and the competent Miocene substrate, there was a layer of sludgy materials with weak strength and high deformability characteristics. Consequently, a foundation solution had to be devised to meet the structural requirements of the installation. To address this, it was necessary to install a pre-loading embankment at the site where the stage would be supported by direct foundations. Part of the stage and its covering had to resort to indirect foundations due to an increase in the load.

To validate the solution, tests were conducted on self-drilling micropiles within the embankment.

Figure 1 shows the location of mentioned structure and their surroundings.

Figure 1 - Location of the main stage.

2 MAIN CONSTRAINTS

2.1 Geological and Geotechnical Constraints

The target intervention area is located in a low and flat area on the right bank of the Tejo estuary.

The Main Stage will be founded on the Beirolas landfill, consisting of deposits of contaminated soils and urban waste, located on a heterogeneous landfill layer containing silty-clay materials mixed with sand and rubble of artificial origin. As the site approaches the river, the thickness of the 'artificial' landfill decreases until it disappears in the tidal zone. Beneath the artificial landfill layer, there is an alluvial formation composed of compressible silty mud soils, with highly variable thickness in the Beirolas landfill area. In the Stage area, the depth of silty mud soils increases from 2m to 25m towards the river, remaining nearly constant in the north-south direction. Underlying this layer of clayey mud, there is a Miocene formation consisting of sandstones, calcareous sandstones, and limestones, with variable presence of marly levels.

In Figure 2, we can see a schematic cross section of the main stage.

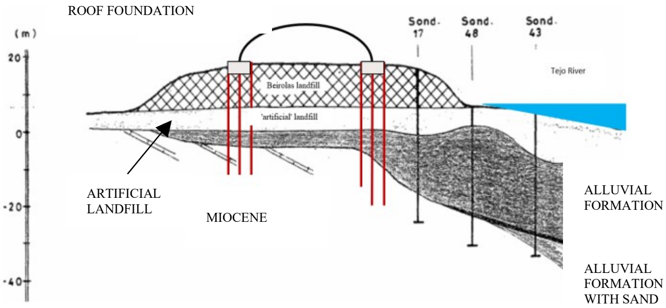

Figure 2 – Cross section of the main stage. Roof foundations.

2.2 Constraints Related to Existing Structures

To establish the foundation solutions for the stage, the initial concept was that indirect foundations were not viable due to the presence of the geomembrane protecting the landfill, making perforation impossible. Moreover, extracting the contaminated soil was not advisable. This premise applied to the entire intervention area, except for the rear zone of the stage – the southern zone – where indirect foundations were contemplated due to the magnitude of the loads to be transmitted to the foundation and the maximum permissible deformations.

2.3 Constraints Related to the Temporary Nature of the Structure

When establishing the foundation solutions for the stage, it was considered that the structure would be temporary and later dismantled. Consequently, a solution was selected for its ease of disassembly, allowing its components to be repurposed in the context of a future project.

3 ADOPTED SOLUTIONS

3.1 Pre-load embankment

The solution consisted of a pre-load embankment with a height of 3.20m in the Stage's implantation zone. The embankment was constructed with material with a bulk density of approximately 20kN/m3. The purpose of this pre-load embankment was to minimize settlements that would occur during the assembly and use of the Stage. The pre-load embankment remained in place for a period of 90 days, experiencing a maximum settlement of 25cm. To verify the design assumptions and anticipate settlements during the assembly of the stage, 16 settlement markers and 2 extensometers were installed during the pre-load phase. The outcomes for the marker A016 are illustrated in Figure 3. In Figure 4 we can see the location of all the settlement marks installed in the area of the pre-load embankment, with the identification of the marker A016.

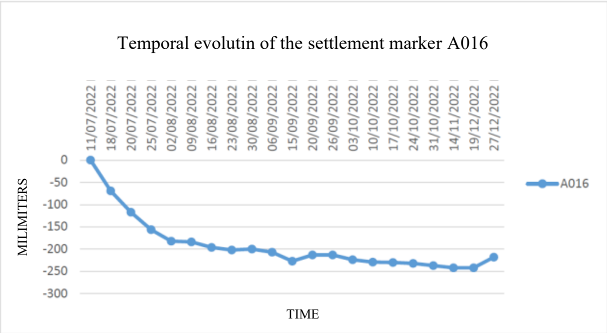

Figure 3 - Time-Strain Graph. Instrumentation of the Pre-load Embankment Zone.

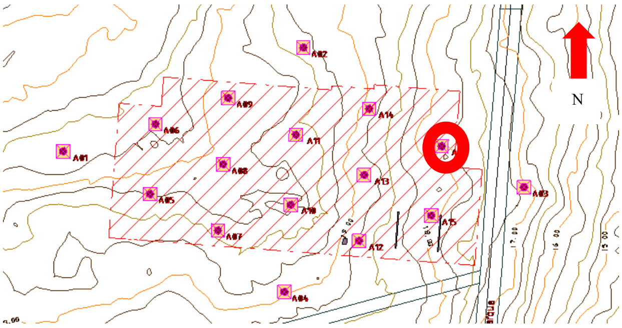

Figure 4 - Location of the settlement marker A016.

3.2 Foundation

To maximize the uniformity of load transmission to the ground, the recommended solution involves the construction of a foundation slab consisting of precast hollow-core slabs with a minimum thickness of 30 cm. Due to the increased magnitude of loads in certain alignments and the irregular geometry of the foundation masses of the stage's roof, it was necessary to recommend an in-situ concrete foundation solution in these areas. Thus, the solution is divided into 4 different types:

- Current area: Solution with hollow-core slabs simply supported on precast peripheral beams with an L-shaped or rectangular section, depending on the existing level difference.
- Periphery of the stage roof foundation: Solution consisting of an in-situ concrete slab with a thickness of 30cm.
- Rear alignments – East and West zones: Solution consisting of an in-situ concrete slab with a thickness of 25cm.
- Rear alignments – central zone:

Foundation beam with a section of 1.65x1.20 m2 supported on ductile iron driven piles with a section of 170x7.5mm filled with C35/45 concrete and 3ø32mm bars inside. The foundation solution in this area was coordinated with the foundation solution of the stage roof, excluded from the scope of this article.

Figure 5 presents the 3D model of the entire foundation solution. Figure 6 and Figure 7 was taken during the construction phase and we can see the assembly of the hollow-core slabs. Figure 8 show the section of the ductile iron driven pile and the asse

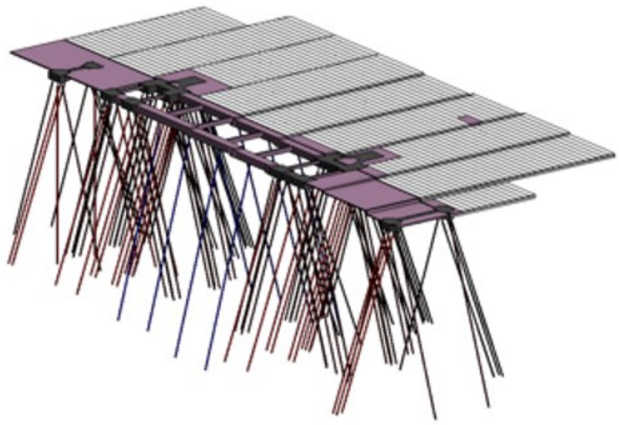

Figure 5 - 3D view of the stage and roof foundations.

Figure 6 - Precast hollow-core slabs.

Figure 7 - Top view of the spread foundation of the stage already assembled.

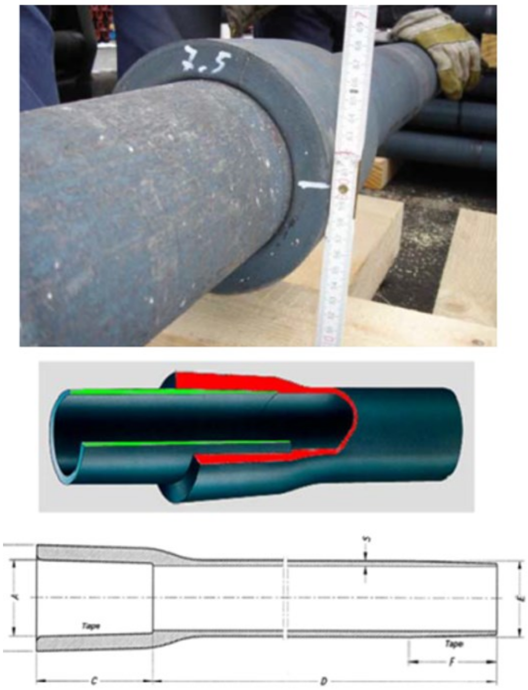

Figure 8 – Ductile iron driven piles with a section of 170x7.5mm.

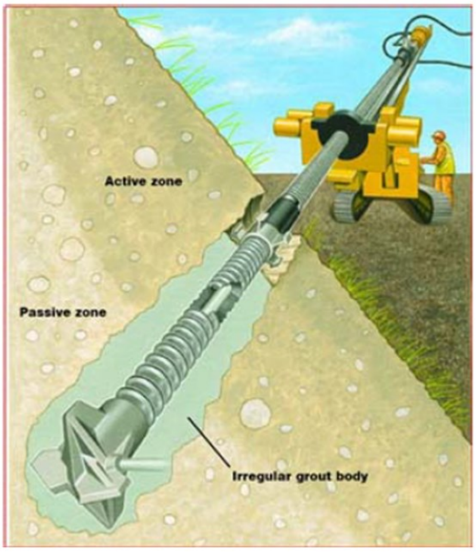

Figure 9 - Self-drilling micropile system along the grout bonding legth. Solution for the stage roof foundation.

4 LOAD TESTS OF DRIVEN MICROPILES

Given the uncertainty associated with the behaviour of the micro piles executed in these materials, it was necessary to carry out load tests to validate the design assumptions. For this purpose, two full-scale load tests were conducted, one in compression and one in tension.

For both test micropiles, interior instrumentation was applied using strain gauges introduced with a ø16mm rod.

4.1 Tension load test

The reaction structure was materialized through a reinforced concrete mass, approximately 2.5m x 2.5m x 1m in size, as shown in Figure 9. The tested micropile had a length of 30.50m, reaching the Miocene substrate, and the self-drilling interior micropile had a length of 41.5m with an approximately 10m grout bonding length in the Miocene substrate (Bustamante and Doix, 1985). The planned test load was 850kN. The load test was conducted with four loading cycles up to the maximum load of 922kN. Additionally, a fifth loading cycle was performed up to 1598kN to test the load-bearing capacity of the sealing in the Miocene stratum. Even after reaching the load of 1598kN, no failure was observed.

Figure 10 - Tension load test.

For the 922kN test load, a strong response was observed in all loading cycles, with a plastic deformation of approximately 13mm and a maximum deformation of 33mm. In the 5th loading cycle, a total deformation of about 59mm was observed, with a residual deformation of 20mm. A set of strain gauges was installed at depths of 2.5m, 10m, 20m, 26m, 30m, and 38.5m, particularly at the transition between the embankment and the alluvial stratum.

The consistency of results in the 4 loading cycles is noticeable, with pressure curves parallel to each other. It is clear that the top of the alluvial stratum is at an elevation of 20m, which is consistent with the results of the boreholes conducted on-site. It is observed that 50% of the load is transmitted to the embankment, and 30% of the grout bonding length was not engaged. However, due to the significant heterogeneity of the embankment material, it was considered that the sealing length should not be less than 10m (Bustamante and Doix, 1985), in order to accommodate the entire tensile load.

The main results of the conducted test are shown in Figure 10 and Figure 11.

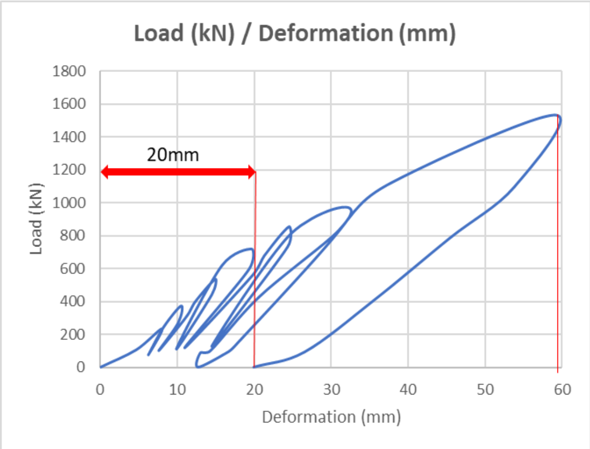

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Figure 11 - Load-displacement graph at the head. Tension load test.

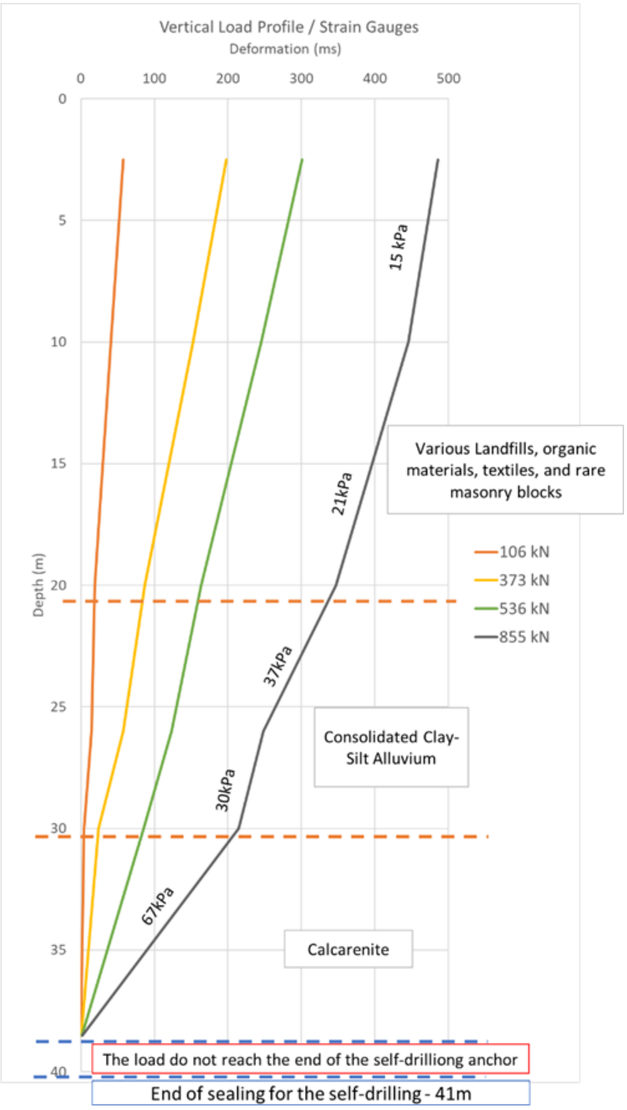

Figure 12 - Strain gauges graph - Tension pre-test.

4.2 Compression load test

The reaction structure was constructed using a portal frame consisting of metal profiles supported on 4 micropiles similar to the micropile tested in tension. The structure is illustrated in Figure 13. The tested micropile had a length of 34.00m, reaching the Miocene substrate. The planned test load was 1820kN. Despite extreme care in aligning the jack with the axis of the micropile, instability of the micropile head mass was observed above a load of 1400kN with horizontal displacement of the mass to the west and north, as shown in Figure 14. Despite the rotation of the mass when the load exceeded approximately 1400kN, a continuous increase in load capacity of approximately 300kN was observed up to 1723kN.

Figure 13 - Reaction structure for the compression load test.

The main results of the conducted test are shown in Figure 14 and Figure 15.

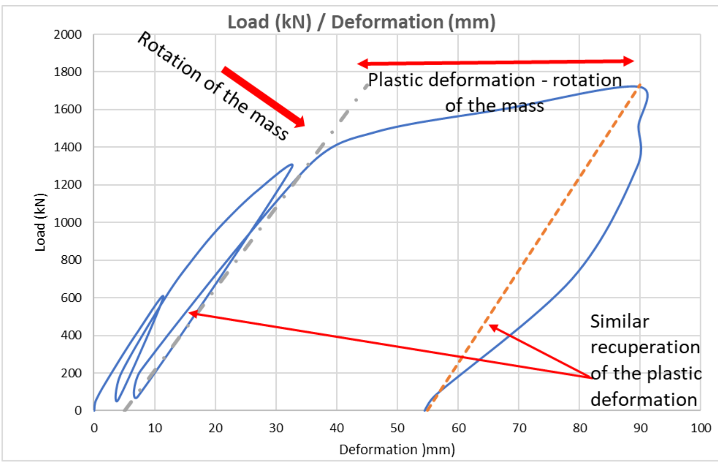

Figure 14 - Load-displacement graph at the head. Compression load test.

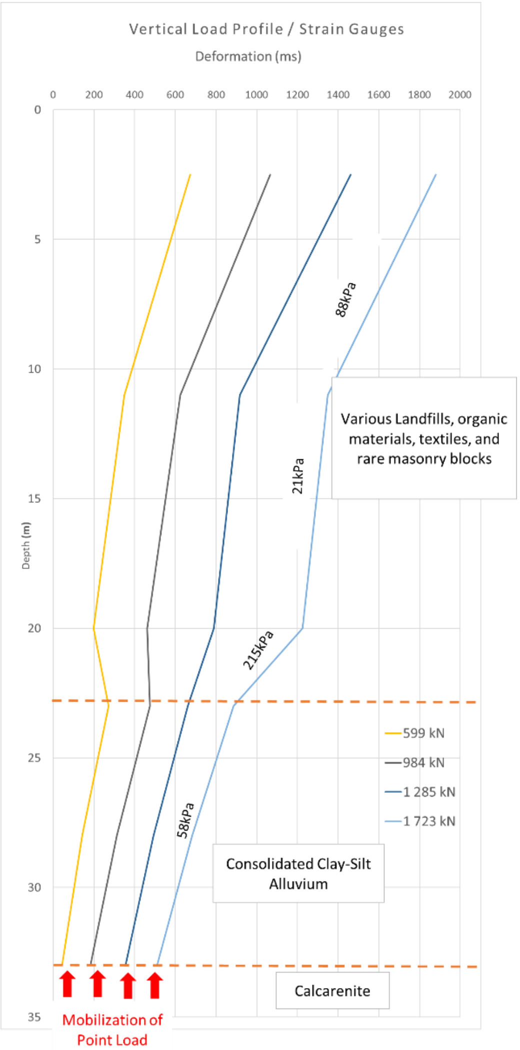

Figure 15 - Strain gauges graph - Compression pre-test.

Figure 15 shows te measuring results of a set of strain gauges was installed at depths of 2.5m, 11m, 20m, 23m, 28m, and 33m, particularly at the transition between the embankment and the alluvial stratum. There is consistency in the results in the 4 loading cycles with pressure curves parallel to each other. It is clearly observable that the top of the alluvial stratum is at an elevation of 23m, which is consistent with information from boreholes conducted on-site. There was a significant load transmission capacity to the embankment with a firm response between 2.5m and 11m and a very good response between 20m and 23m depth. Between the depths of 11m and 20m, the load transmission to the embankment was negligible. The mobilized tip load is estimated to be around 600kN for a load on the order of 1700kN. It is estimated that the compressive capacity of the pile formed by the ductile iron outer micropile and the self-drilling inner bar is above 1800kN, with a plastic displacement of about 20mm and 40mm when subjected to a load of 1800kN.

5 CONCLUSIONS

The purpose of this article was to present the foundation solution for the main stage of the World Youth Day, highlighting the various constraints that influenced the choices and decisions made.

Based on the previously described constraints and on the results of the mentioned load tests, the adopted solution consists of a foundation slab made up of precast hollow-core slabs with a minimum thickness of 30 cm, which can be removed. In the heavily loaded alignments, the solution transitions from direct to indirect foundations using ductile iron driven micropiles, topped with foundation beams measuring 1.65x1.20m, connected by perpendicular beams with a section of 0.40x0.80m.

The possibility of removing the structures was taken into consideration, and whenever possible, solutions were adopted that would simplify demolition/dismantling, allowing the constituent parts of the structures to be potentially reused.

To validate the design assumptions of the foundation micropiles, two load tests were conducted at the stage location, one in compression and one in tension. The obtained results allowed for the early validation of the main design assumptions, ensuring the suitability of the solution to the local geological-geotechnical context.

The suitability of the solution to the implementation site was verified based on the results of the conducted tests and continuous monitoring of the construction work and the micropile driving results.

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We thank the Project Owner for the permission to publish this article.

We also express our gratitude to the team that made the described work possible, particularly to the companies Oliveiras S.A and Geosol S.A, as well as to the company responsible for overseeing the project, Engexpor – Consultores de Engenharia S.A.

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Coastal cliff stabilization and access stair reconstruction at Peneco beach, Algarve, Portugal

Stabilisation de la falaise côtière et reconstruction de l'escalier d'accès à la plage de Peneco, Algarve, Portugal

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ABSTRACT: The coastal cliffs erosion is a natural process that can lead to unstable occurrences and consequently high risk to the beach users. This paper describes the main solutions adopted at Peneco Beach, Algarve, Portugal aimed to mitigate the risk associated with space usage and access to the beach. Simultaneously, severe structural pathologies were also observed at the beach access stairs determining risk to users and therefore stabilization works were needed. The main goal was to develop solutions that could mitigate the users' risk while being conscious about environmental and aesthetics constrains as well as equipment accessibility restrains. The cliff stabilization solution used pigmented reinforced shotcrete combined with steel rockbolts, while the access stair was rebuilt using reinforced concrete and vertical rockbolts as foundation elements.

RÉSUMÉ : Les falaises côtières se détériorent en raison de l'érosion, qui est un processus naturel, mais qui peut conduire à des occurrences instables et, par conséquent, à un risque élevé pour les utilisateurs de la plage. Cet article décrit les principales solutions adoptées à Peneco plage, en Algarve, Portugal, visant à atténuer les risques associés à l'utilisation de l'espace et à l'accès à la plage. Dans le même temps, ont également été observées de graves pathologies structurelles au niveau des escaliers d'accès à la plage, déterminant le risque pour les usagers et donc des travaux de stabilisation ont été nécessaires. L'objectif principal était de développer des solutions capables d'atténuer les risques pour l'utilisateur tout en tenant compte des contraintes environnementales et esthétiques ainsi que des contraintes d'accessibilité des équipements. La solution de stabilisation de la falaise a utilisé du béton projeté renforcé pigmenté combiné à des clous de sol en acier, tandis que l'escalier d'accès a été reconstruit en utilisant du béton armé et des clous de sol verticaux comme de même de fondation.

Keywords: Coastal cliffs, risk mitigation, rockbolts, shotcrete

1 INTRODUCTION

Coastal cliffs are areas of natural geological risk of instability due to their sedimentary genesis and exposure to atmospheric and marine agents with consequences for people and property. The risk associated with the instability of cliffs in beaches has been mitigated by several interventions along Portugal's coastline. At Peneco beach, Algarve, the reconstruction of the beach access staircase was implemented, restoring the safety conditions, and mitigating the erosion phenomena of the adjacent cliffs that determine an increased risk for people.

2 REFERENCE SCENARIO

At Praia do Peneco the morphology of the cliffs is a clear expression of the lithologies, as well as the processes and mechanisms of evolution present, mainly due to differential marine erosion. At the cliff face, erosion is accelerated by the intense vegetation that occurs in the place, since the growth of roots helps the opening of discontinuities and the consequent detachment of blocks (see Figure 1).

Figure 1. Cliff before intervention.

The top of the cliff is occupied by hotel and restaurant buildings and viewpoints, where small settlements, displacements and fractures were visible. The scarce presence of drainage and waterproofing devices in the area means that on days of high rainfall, water infiltrates into the existing cracks, dragging finer soil particles, promoting internal erosion phenomena.

The anomalies observed were representative of movements that have been progressively occurring over the years. The area of greatest concern corresponded to the top of the cliff, specifically to the viewpoint where tension cracks were observed in orientation parallel to the slope clearly showing signs of the occurrence of movements in this place.

The beach access staircase presented several structural pathologies caused by differential settlements (see Figure 2). The origin of such pathologies can be related to the lack of drainage devices, and a possible deficient foundation condition. Rainwater made its way to the foot of the cliff, infiltrating under the staircase by dragging particles, causing the observable differential settlements.

Figure 2. Staircase structural pathologies.

In this context, an intervention on cliffs stabilization and staircase reconstruction was considered critical for safety reasons.

3 MAIN CONSTRAINS

3.1 Geological and geotechnical constrains

Based on the Explanatory Report and the Geological Map of Portugal at a scale of 1:50000, sheet 52-B, it is possible to confirm the presence a Miocene unit (M^1) presented as carbonate rocks materializing the cliff (see Figure 3).

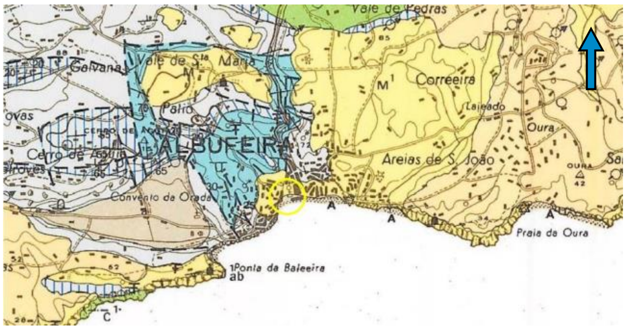

Figure 3. Portugal's Geological Map [sheet 52-B].

Field observation detected the presence of fine sandstones and siltstones, hard and compact, with yellowish tones, materializing metric benches, sometimes combined with centimetric horizons of silts and clays of grey tones (see Figure 4).

Figure 4. Cliff's surface before intervention.

The rock mass along the cliff is stiff and presents fractures that are moderately distant (F3-4). The transversal profiles of cliff can be described as slightly convex, steep slope and with sub-vertical sections, in the most competent rock formations. The cliff profiles are very rough, due to intense fracturing and high density secondary cementation of discontinuities (Marques, 1997).

The geological unit has, in general, a fissure-type permeability, where the circulation of water is essentially through the network of fractures and fissures. Thus, the circulation of groundwater is dependent on the degree of fracturing and weathering of the rock mass.

3.2 Other constrains

Given the location of the cliff, there were no significant constraints for access to the base of the cliff. However, given its height of about 20 m, the proposed solutions determined the need to use light weight equipment that could be operated hanging from a mobile crane.

The stabilisation solutions to be implemented had to use adequate materials considering the particularly aggressive environment due to the proximity to the sea. The interventions were also conditioned by the need to include the execution of a new rainwater drainage system, under the structure of the staircase.

Finally, it is also important to highlight the need to limit the impact of the intervention, especially in terms of the landscape and architectural framework of the proposed solutions always ensuring the indispensable safety conditions and their technical and economic suitability.

4 STABILIZATION SOLUTIONS

To mitigate the risk an intervention on cliffs stabilization and staircase reconstruction was implemented. For the cliffs surface a stabilization solution using rockbolts, geodrains and reinforced shotcrete was considered aiming the mitigation of the phenomena of superficial instability and erosion. Regarding the staircase pathologies a full reconstruction was proposed.

4.1 Cliff intervention

The intervention consisted of application of shotcrete, partially reinforced with metal fibres, and applied in layers with a total thickness of 12 cm. The application of shotcrete with C30/37 class resistance, took place by wet application and with the application of metal fibres, with a minimum dosage of 30 kg/m3, only in the first two layers. In the case of cavity areas, they were pre-filled, also with the application of layers of shotcrete. In the last layer of sprayed concrete, 4cm thick, applied without metallic fibres, a yellow pigment was incorporated for a better landscape integration of the solution (see Figure 5).

Rockbolts were materialized by 36 mm diameter bars, in A500/A550 continuous thread steel, in hot-dip galvanized rod with 85 µm minimum thickness by ISO 1461, placed in holes with a minimum diameter of 116 mm and sealed with cement grout of suitable characteristics. With this process, it was intended to disturb to a minimum the mechanical characteristics of the natural rock during the execution of the works. The diameter of the nails was defined considering a proper sacrificial thickness despite the galvanization.

With a length of 6 m and an inclination of 20º, in addition to fixing the layer of shotcrete, the nails crossed some existing sub-horizontal discontinuity planes near the cliff face. The nails were arranged in an approximate mesh of 3.0m (V) x 3.0m (H), and their position was adjusted during the works according to local needs.

To ensure the internal drainage, preventing the possible generation of hydrostatic pressure in the shotcrete coat, caused by water infiltration, sub-horizontal 4.0 m long geodrains were installed with 50 mm diameter perforated piped and wrapped in TecGeo 300 type geotextile. The gravitational flow of the captured waters is guaranteed because these elements were installed with an upward inclination of 5º with the horizontal.

Figure 5. Cliff's surface after intervention.

4.2 Staircase intervention

Given the severe structural pathologies of the staircase the intervention included its reconstruction executed on top of the existing one. Expanded clay aggregates were used for lightweight filling bellow the staircase concrete structure and rockbolts were used as foundation (see Figure 6).

For a conjoint behaviour of the entire structure, reinforced concrete cross beams were built to ensure a better connection between the various walls, and at the end of these a deep foundation was ensured, materialized by vertical rockbolts sealed in the rock mass.

Given the height of some retaining walls that materialize the staircase, geodrains were installed preventing the build-up of hydrostatic horizontal pressures behind those walls and using the drainage system, constructed bellow the new staircase, to drive those natural waters to the cliff base.

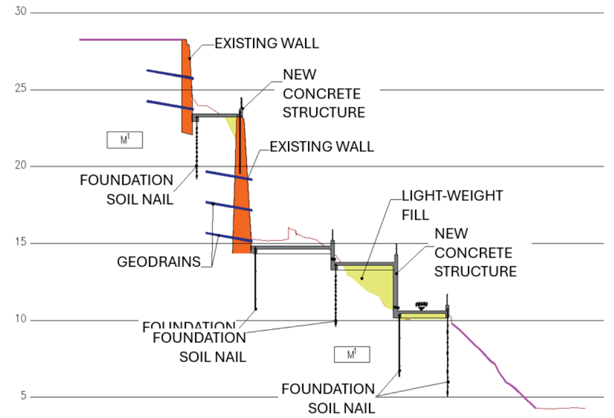

Figure 6. Staircase reconstruction – Cross section (not to scale).

5 MONITORING AND SURVEY PLAN

To monitor any relatively slow sliding movements, inclinometers were installed at the top of the cliff to measure the horizontal deformation of the rock mass. The intervention also provided for an observation plan, which should focus on the conservation and maintenance of the installed systems, to confirm the proper functioning of these structures. To this end, the observation of the intervened cliff will be made through the implementation of the following measures:

- Photographic coverage of the cliff after the intervention has been carried out, with the aim of defining an initial reference situation;
- Periodic photographic coverage in a manner similar to that recorded for the initial reference situation, in order to facilitate its comparison and the detection of any pathologies;
- Periodic reading of the inclinometers installed at the top of the cliff;
- Periodic inspection of the installed drainage systems and verification of their condition.

The information collected within the scope of the measures above described should be evaluated by a technical specialist to continuously validate the suitability of the solutions implemented and the need to carry out maintenance and conservation interventions or confirm the need for future stabilization interventions.

6 MAIN CONCLUSIONS

This article described the main solutions implemented and completed in the summer of 2022, with aim to mitigate the risk associated with the use of space and access to the bathing area at Praia do Peneco, in Algarve, Portugal (see Figure 7). The maintenance of the reinforcement elements and drainage elements installed, namely through the application of the defined monitoring plan, is fundamental to confirm the suitable behaviour of this type of structures and solutions during their lifetime. Special awareness should be considered within the monitoring plan implementation before and after periods of greater rainfall and greater sea agitation.

Coastal cliff erosion phenomenon can cause significant issues for public safety and serious damage to infrastructures. Thus, coastal cliffs stabilization solutions are crucial preventing the evolutive erosion that can lead to a natural disaster.

Figure 7. View after intervention.

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The authors would like to thank the Albufeira City Council, the Owner, for the authorization for the writing and publication of this article. They also consider important to underline that the solutions implemented are the result of teamwork, in which the important role of the companies should be highlighted: GeoAlgar, the entity responsible for the preparation of the Geological and Geotechnical Study and Teixeira Duarte, Engenharia e Construções S.A., the contractor.

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GEOTECHNICAL ENGINEERING

Ground improvement and special foundations at North Lisbon Logistic Platform, plot 1, Portugal

Solutions d'amélioration du sol et fondations spéciales sur la Plateforme Logistique au Nord de Lisbonne, parcelle 1, Portugal

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ABSTRACT: The North Lisbon Logistic Platform is located over soft soils at Tagus river right bank. At the Plot 1 an industrial warehouse with about 100.000 m^2 was built. For the warehouse foundations the following solutions were adopted: i) structural elements: reinforced concrete driven piles; ii) indoor pavements: load transfer platform, granular fill reinforced with geosynthetics, over rigid inclusions, geoconcrete columns (GCC) with under reamed base and capped by stone columns; iii) outdoor pavements: load transfer platform, granular fill reinforced with geosynthetics, over stone columns. The main design and execution criteria are presented, as well as the main results of both site monitoring and full-scale load tests.

RÉSUMÉ: La plateforme logistique du nord de Lisbonne est située sur des sols meubles sur la rive droite du Tage. Sur la parcelle 1, un entrepôt industriel d'environ 100.000 m^2 a été construit. Pour les fondations de l'entrepôt, les solutions suivantes ont été adoptées: i) éléments structurels: pieux battus en béton préfabriqué; ii) revêtements intérieurs: plateforme de transfert de charge, remblai granulaire renforcé de géosynthétiques, sur inclusions rigides, colonnes en géobéton (GCC) avec base sous alésage et coiffées par des colonnes ballastées; iii) revêtements extérieurs: plateforme de transfert de charge, remblai granulaire renforcé de géosynthétiques, sur colonnes ballastées. Les principaux critères de conception et d'exécution sont présentés, ainsi que les principaux résultats du suivi du chantier et des essais de charge en vraie grandeur.

Keywords: Driven piles, rigid inclusions, stone columns; load transfer platform; soft soils

1 INTRODUCTION

The North Lisbon Logistic Platform (NLLP) is located in Castanheira do Ribatejo, Portugal. This platform was built with the intend of creating an interconnection point for international, national and regional logistical flows within the Lisbon and Tagus Valley region. An example of recent construction in NLLP is presented in Lopes et al (2023) .

On plot 1 of the NLLP a new industrial building with an area of approximately 100.000 m^2 was built. In the aerial view in Figure 1, it is possible to identify its location.

The target area of intervention is located on the right bank of the Tagus River, that, from a geological-geotechnical point of view, is dominated by the alluvial terrain of the lower Tagus, characterized by the presence of soft soils with reduced bearing capacity. In the particular area where the new structure is located, the alluvium reaches considerable depths, of the order of 12.0 to 18.0 m, posing serious challenges to the construction of any structure in that location.

Figure 1. Aerial view of the construction site (image taken from Google Earth).

2 GEOLOGICAL AND GEOTECHNICAL CONDITIONS

The site under study fits, from a geomorphological point of view, in the middle of the alluvial plain of the Tagus river, close to the right bank of this main watercourse.

Local geological conditions therefore indicate alluvial formations, of Holocene age, deposited over a sedimentary substrate, called terrace deposits, attributed to the Plio-Pleistocene.

According to the information obtained during the geotechnical investigations, it appears that the local geological-geotechnical scenario is characterized by a layer of surface deposits, developing to a depth variable between 2.0 m and 3.0 m. According to the available sampling, it is a heterogeneous layer made up of a mixture of very soft to medium consistency clays, silts and sands ($0<N_{SPT}<8$). It is considered likely that these materials present an overconsolidated state, as a result of fluctuations in the water table.

Immediately below the layer of surface deposits, to a maximum depth of 12.0 to 18.0 m, there are alluvial soils, essentially made up of (i) very loose to loose sandy-silty horizons detected with thicknesses varying between 1.5 and 6.0 m ($0<N_{SPT}<8$); and (ii) very soft to soft silty clays with thicknesses varying between 6,0 and 11.0 m ($0<N_{SPT}<4$).

Underlying the alluvial layer, there are deposits of Plio-Pleistocene terraces, characterized by the interspersed occurrence of clay, sand and of gravel horizons, all with significant lateral continuity.

As depth advances, an increase in the resistant behaviour exhibited by this formation is observed, as shown by the generally increase of N_{SPT} values in depth, with the lowest values, generally varying between 14 and 30 blows, coming from the top of the formation, showing the occurrence of a more weathered surface zone, with behaviour of moderately dense to dense materials and generally hard to very hard consistency. An exception is made for some specific levels where the SPT tests indicate soft to medium consistency soils among the most compact materials.

The position of the water table is dependent on the water level in the Tagus river, being influenced not only by seasonal variations, but also by the daily tidal cycles felt in the estuary. During the prospecting geotechnical works, the presence of the water table was detected at a depth varying between 1.68 m and 2.12 m.

3 SOLUTIONS

Given the existence of highly compressible cohesive soils, the presence of geological and geotechnical conditions that are globally unfavourable to the accommodation of high and/or persistent loads is evident, with the risk of accentuated and evolving deformable behaviour over time. Consequently, there is a concern about mitigating immediate and deferred deformations, in accordance with the structure serviceability requirements. In this context, the following solutions were adopted:

a) Pile foundations for the warehouse resistant structure;
b) Ground improvement solutions of the foundation soil of interior and exterior pavements using (i) rigid inclusions, using the GEOPIER® GCC system, with a bi-module gravel and concrete cap and (ii) stone columns, using the GRAVA IMPACT® system.

3.1 Pile foundations

The proposed deep foundation solution consists of prefabricated reinforced concrete driven piles, of the TERRA® type, capable of accommodating compression, tension, flexural-compression, flexural-tension and shear loads.

T-350 piles (350x350mm) with a minimum length of 25.0 m were considered, with a seismically reinforced section in the first 12.0 m to be considered from the base of the pile cap. In Figure 2 a photograph of the prefabricated reinforced concrete driven piles on the construction site is presented.

Figure 2. Driven piles.

The stated piles are capped by reinforced concrete pile caps of 1, 2, 3, 4, 5, 6 and 7 piles that, whenever submitted to bending moments that are not possible to accommodate by the binary of forces achieved by axial stresses developed on the vertical piles, are interconnected by a grid of foundation beams, also in reinforced concrete.

Along the peripheral alignment, a solution using sub-verticals piles (10º from the vertical) was also defined. These will have the function of accommodating the vertical and horizontal forces coming from the earth retaining walls on the periphery of the building.

In addition to what was mentioned above, the piles must also be responsible for dissipating the shear coming from the pillars and walls, as well as ensuring the accommodation of the bending stresses developed in depth due to the action of the shear forces.

In Figure 3 a view of the 3D geometric model of the structure's foundation solution is presented.

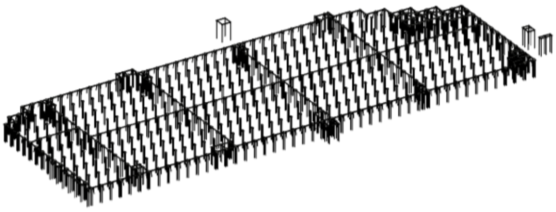


Figure 3. 3D geometric model of the structure's foundation solution (not to scale).

3.2 Ground improvement

Considering the geotechnical conditions described, aiming to reduce total and differential settlements, it was decided to implement, for the indoor pavement as well as the exterior loading/unloading docks pavement area where heavy vehicles will be parked, a ground improvement solution comprising the execution of rigid inclusions, using the Geo-Concrete Columns (GCC®) of the GEOPIER system, with under reamed base, under a load transfer platform, consisting of layers of granular fill reinforced with geosynthetics.

The aforementioned rigid inclusions are arranged in different square arrays depending on the magnitude of the expected service live load. In Figure 4 and Table 1, a schematic plan is presented, with the location, spacing and magnitude of the different service live loads considered.

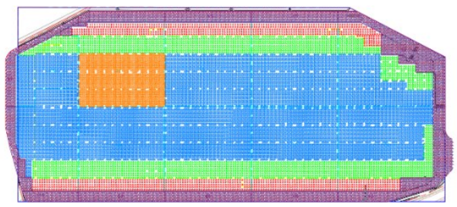


Figure 4. Schematic plan with the ground improvement solutions (not to scale).

Table 1. Live loads and ground improvement techniques considered for the pavement's foundation soil.

Zoning	Live load	System	Spacing
1	75 kN/m^2	GCC Rigid inclusions	2,30 m
2	50 kN/m^2		2,60 m
3	32 kN/m^2		3,10 m
4	20 kN/m^2		3,25 m
5	20 kN/m^2	IMPACT Stone columns	2,50 m

Prior to the installation of the rigid inclusions, in order to reach the levels foreseen for the pavements, an embankment with an average thickness of approximately 2.0 m was built. In this context, to minimize the excavation works, it was decided to execute pre-drilling works and install a stone column cap on top of the concrete columns.

For the exterior pavements, in the area intended for car traffic, likewise aiming to minimize total and differential settlements, a ground improvement solution using stone columns, using the GRAVA IMPACT® system was adopted. The stone columns are arranged in a triangular mesh spaced 1.2 m, under a load transfer platform, consisting of layers of granular fill reinforced with geosynthetics.

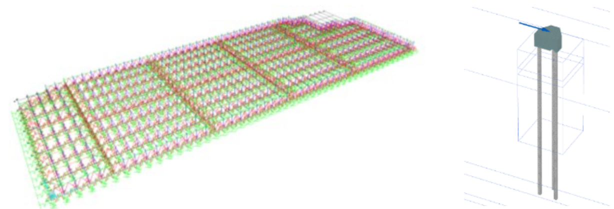
In Figure 5 a photograph of the load transfer platform is presented.

Figure 5. Load transfer platform.

4 DESIGN

4.1 Pile foundations

The foundations as well as the building's structure were modelled using the SAP2000 finite element software, in which the various structural elements were represented with their true dimensions and stiffness. In Figure 6 the 3D foundation model is presented.



a) b)

Figure 6. a) Foundation's structure 3D SAP2000 finite element model. b) Pile Plaxis 3D finite element model.

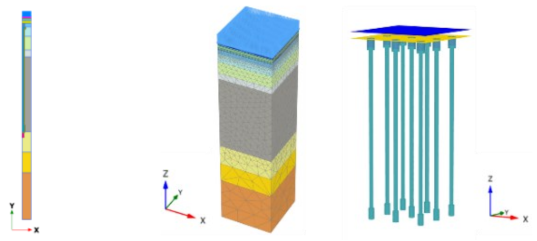
For a more realistic analysis of the forces and displacements to which the structure and its foundations will be subjected, spring elements with vertical, horizontal and flexural stiffness of the various types of pile cap and prefabricated piles in each vertical structural element (columns and façades) were considered. The vertical and flexural spring stiffness were estimated based on the vertical static load tests carried out. The Plaxis 3D finite element software was used to determine the horizontal stiffness of the piles, as well as to quantify the axial stresses due to the effect of negative skin friction and bending moment distribution along the piles.

4.2 Ground improvement

The study of the ground improvement solutions was based on an initial calibration phase through stress-strain analyses, namely, two-dimensional finite element analysis, in axisymmetry condition, using the Plaxis 2D software, simulating two of the static vertical load tests carried out on site.

In a second phase, the project ground improvement solutions were modelled, also through two- and three-dimensional finite element analysis using the Plaxis 2D and Plaxis 3D software respectively.

With this analysis the stresses, strains and displacements in the rigid inclusions, stone columns and geosynthetics were estimated. The settlements on the pavements were also calculated and compared with the floor's serviceability requirements. In Figure 7 some of the 2D and 3D calculation models are presented.



a) b)

Figure 7. Ground improvement finite element calculation models a) Plaxis 2D, axisymmetry conditions. b) Plaxis 3D.

5 MONITORING AND FULL-SCALE LOAD TESTS

Prior to the execution of the work, a set of static and dynamic load tests on vertical prefabricated driven piles were carried out. The aforementioned tests allowed the analysis of the behaviour of the piles with regard to force-settlement curves, being therefore fundamental elements in the design stage.

As part of the quality and execution control of the stone columns, as well as the GCC rigid inclusions, a set of static vertical load tests were also carried out, considering loads higher than the expected service load.

Additionally, in order to evaluate the performance of the ground improvement solution in the medium and long term, a full-scale test was carried out, through the construction of an experimental landfill over four GCC rigid inclusions lasting approximately 5 months, up to 200% of the service load. The results, presented in Figure 8, confirmed that the expected settlement was inferior to the considered admissible value. In Figure 9, it is possible to view the full scale and static vertical load tests.

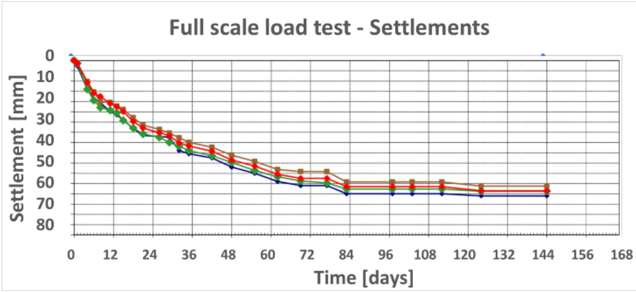


Figure 8. Full scale load test – Settlement evolution over time.

a) b)

Figure 9. a) Full scale load test. b) Static vertical load test.

6 FINAL REMARKS

The framework of the work described determined the need to develop economic foundation solutions compatible with the structure serviceability.

In this context, it should be noted that the adopted ground improvement solutions allow for the reduction, in the foundation structural elements, of the design seismic horizontal forces originating from the expected high magnitude pavements live loads.

It also made it possible to reduce shear stresses and bending moments at the floor slab, thus allowing for considerable savings in the reinforced concrete pavement structure.

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Special foundations at the cycle-pedestrian bridge over the Trancão river, Portugal

Fondations spéciales du pont cyclable-piéton sur la rivière Trancão, Portugal

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ABSTRACT: This paper describes the special foundations solutions adopted at the cycle-pedestrian bridge built over the Trancão River, connecting Lisbon to Loures. The bridge structure is a laminated wood deck, supported by steel cables, with a 65m central main span. Considering the very low resistance and high deformability of the superficial alluvial layers, it was necessary to design a deep foundation solution, resting at the Miocene bedrock. Reflecting the local geological scenario, as well as the restraints related with the use of heavy equipment at the riverbanks, the foundation solution consists of micropiles, vertical and inclined, executed using the self-drilling methodology, associated with the execution of 500mm diameter mini jet grouting columns, ranging from about 30 to 20m. The main quality control / quality assurance (QC/QA) results, including two full scale tension load tests, are presented.

RÉSUMÉ: Cet article décrit les solutions de fondations spéciales adoptées pour le pont cyclable-piéton construit sur la rivière Trancão, reliant Lisbonne à Loures. La structure du pont est un tablier en bois lamellé collé, soutenu par des câbles en acier, avec une travée centrale de 65 m. Compte tenu de la très faible résistance et de la déformabilité élevée des couches alluviales superficielles, il était nécessaire de concevoir une solution de fondation profonde, reposant sur le substratum rocheux du Miocène. Compte tenu du scénario géologique local, en plus des contraintes liées à l'utilisation d'équipements lourds sur les berges, la solution de fondation consiste en des micropieux, verticaux et inclinés, exécutés selon la méthodologie de auto forage, associés à l'exécution de mini colonnes de jet grouting de 500 mm de diamètre, allant de 30 à 20 m. Les principaux résultats du contrôle de la qualité e du contrôle de l'exécution (CQ/QA), bien aussi que deux tests de charge de traction à grande échelle, sont présentés.

Keywords: Jet Grouting, Micropiles

1 INTRODUCTION

The new cycle and pedestrian bridge over the Trancão river was built before the World Youth Day 2023 (WYD), with the main objective to connect the paths already existent at both banks of the Trancão river (Loures at North and Lisbon at South).

With this purpose, a three span (28m+65m+28m, 121m overall length) cable stayed bridge, with laminated wood deck, steel towers and cables, resting over reinforced concrete columns and abutments, was built. At Loures and Lisbon sides, respectively, access ramps with 82m and 70m were built, the last one parallel to the Trancão river (Figures 1 and 2).

Figure 1. Bridge location (Google maps, without a scale).

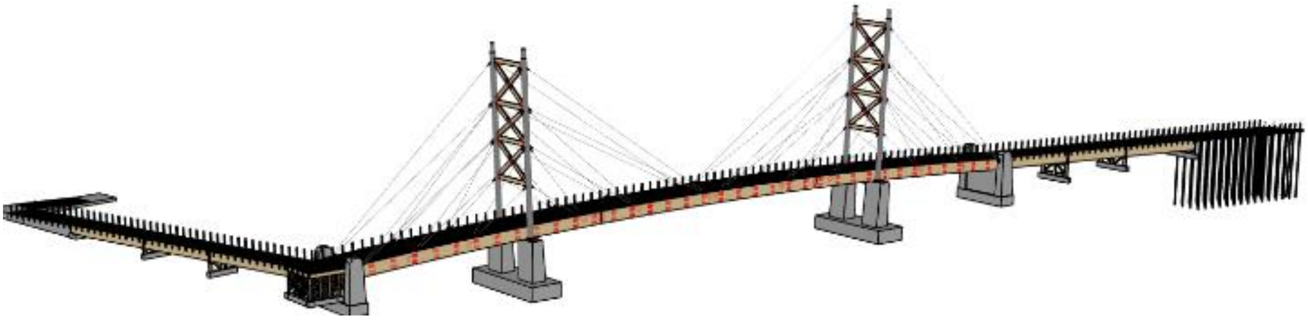

Figure 2. Bridge 3D view (JETsj project).

2 GEOLOGICAL CONSTRAINS

The geological investigation campaign included the execution of four boreholes and allowed the confirmation of the ground geotechnical zones (ZG). It was possible to confirm, from the surface, the existence of 2.5m heterogeneous landfills (ZG1), resting over very soft alluvial (Tagus River basin) materials with overall thickness ranging from 14 to 22m, with N_{SPT} blows not bigger than 2 and undrained shear strength (Su) not bigger than 20kPa (ZG2).

Under that layer, the Miocene loose sandy soils (ZG3) over the sandstone bedrock (ZG4) were found, the last one with very good resistance and deformation characteristics (Figure 3).

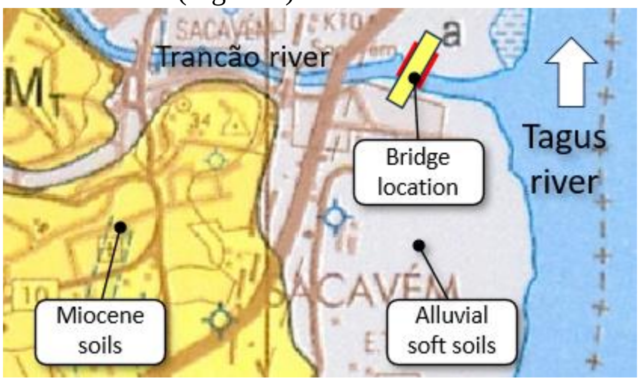

Figure 3. Geological scenario plan with bridge location (Portugal geological map, without a scale).

3 FOUNDATIONS SOLUTION

Considering the geological and geotechnical restraints, the compression and tension loads to be transmitted by the bridge to the ground, as well the restraints related with the use of heavy equipment at the riverbanks soft soils, the foundation solution consisted on micropiles ANP ϕH1600-76mm, ϕH0950-51mm and ϕH0550-38mm hollow steel bars with sacrificial grouting bits, vertical and inclined, executed using the upper – bottom self-drilling methodology, associated with the execution of 500mm diameter mini jet grouting columns at the ZG1, ZG2 and ZG3 geotechnical zones, ranging from about 30 to 20m (Figure 4).

Maximum jet grouting pressure was 200bar, compatible with the micropiles coupler's resistance. Due to the Atlantic Ocean proximity, leading to water chlorides, for durability reasons, pozzolanic cement with a consumption ratio of 160kg/m^3 was used.

The jet grouting columns had three main functions: i) increase the micropiles stiffness mainly against buckling; ii) increase the micropiles protection against corrosion; iii) reduce the micropiles overall length at the Miocene bedrock, particularly when the columns intersected the Miocene loose sands, at the South riverbank.

Figure 4. View of the micropiles hollow bars and bits.

The micropiles were capped by reinforced concrete caps, supporting the bridge columns and abutments. The loads were transmitted to the Miocene layer mainly by shaft resistance: i) at the South riverbank, starting at the loose sandy materials taking advantage of the 500mm jet grouting columns shaft resistance (qs); ii) at the North riverbank, at the Miocene bedrock, where the maximum diameter was only 250mm (Figure 5).

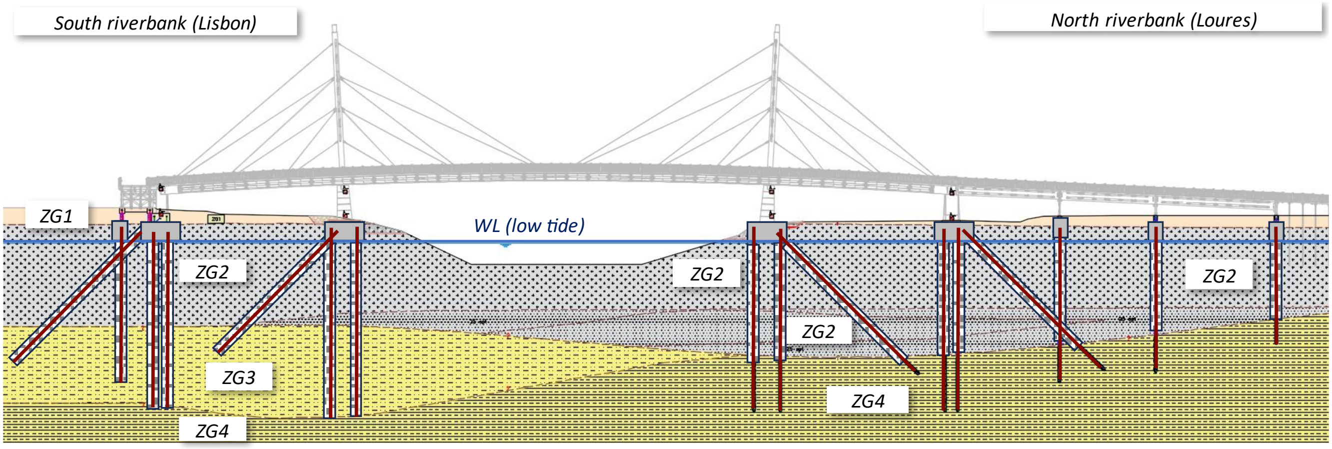

Figure 5. Bridge elevation with geotechnical zones (JETsj project, without a scale).

4 QUALITY CONTROL AND QUALITY ASSURANCE

4.1 Trial columns

Before the execution of the full-scale load tests, several jet grouting trial columns, with different cement consumption, were performed to check both the columns geometry and resistance at the very soft resistance alluvium layer (ZG2).

The columns resistance was evaluated trough unconfined compression strength (UCS) tests, performed trough cores collected from the columns at 28 days. At UCS tests both the columns unconfined compression resistance and Young's modulus were checked and compared with the design requests at 28 days: 3MPa and 1GPa, respectively (Figure 6).

Figure 6. Trial column view.

4.2 Tension Full Scale Load Tests

To confirm the design main assumptions, mainly the micropiles and jet grouting shaft resistance (qs), two suitability tension full scale load tests were performed, one in each riverbank, close from the bridge central columns.

The reaction structure was a reinforced concrete cap ($2x2x1m^3$) supported by four inclined self-drilling micropiles. The central tested micropile was reinforced with one ϕ35mm Gewi Plus S670/800 bar, protected by a smooth sheath at the landfill (ZG1) and alluvium (ZG2) layers, ensuring the unbonding length. A hollow plunged jack was used to apply a maximum load of 500kN to the micropile through the ϕ35mm bar (Figure 7). The maximum applied load was 1.67 times bigger than the maximum tension service load, approximately 350kN (Figures 8 and 9).

Figure 7. Full scale tension test: reaction structure view.

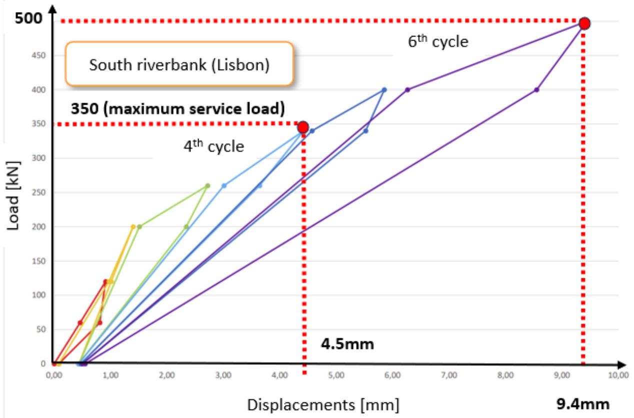

Figure 8. Load vs head displacements (South riverbank).

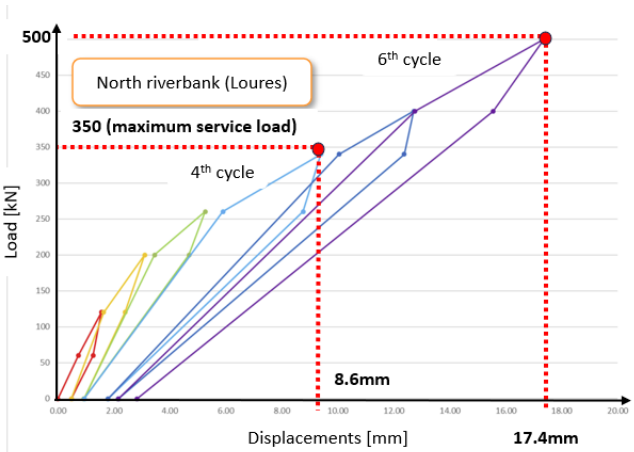

Figure 9. Load vs head displacements (North riverbank).

After 6 load and unload cycles the maximum displacements were 9.4mm and 17.4mm at, respectively, the South and North riverbank load tests. The plastic displacements were, respectively, 0.5mm and 2.5mm. The explanation for those differences could be related with the micropiles stiffness, as on the South riverbank the loaded micropile full length was inside a jet grouting column at the Miocene loose sandy soils (ZG3, with qs not lesser than 25kPa) and the alluvium (ZG2) thickness is lower (Figure 5).

Despite the tension loads at the micropiles are mainly due to wind and seismic loads, on both load tests the creep coefficient, for a minimum permanent load time of 30 minutes, was lesser than 1mm. The obtained results allow to confirm the suitability of the design and execution parameters.

4.3 Execution parameters recorder

All the jet grouting execution parameters were automaticallty recorded using a computer device and compared with the ones established based on the trial columns and suitability full scale load tests results.

5 DESIGN

The bridge foundations were designed using PLAXIS 2D, FEM stress-strain software. The shaft resistance values were initially estimated using reliable proposals (Bustamante, 2002) for jet grouting. The model was adjusted by back analysis after the full-scale load tests. According to the bridge monitoring, using prisms and topographic readings, since its opening to cycling and pedestrian traffic (Figure 10), the deck displacements have been in accordance with the estimated ones at the design stage.

Figure 10. Bridge view just before the YWD, August 2023 with the deck being reinforced for the pope's car crossing.

6 FINAL REMARKS

The presented case study allowed to confirm the advantages of foundation solutions using mini jet grouting columns combined with self-drilling micropiles in complex geological and geotechnical scenarios. As main advantages, compared with conventional solutions, can be pointed out: light, small and versatile equipment, minimum ground extraction, schedule prediction, and overall costs.

ACKNOWLEDGEMENTS

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Deep and complex excavation in an urban environment in Miraflores, Oeiras

Excavation profondes et complexes en milieu urbain à Miraflores, Oeiras

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ABSTRACT: This paper addresses the design and building solutions for the peripheral earth retaining structures, required for the construction of a development, which is a building complex of two building each 14 storeys high. The excavation footprint has approximately 6000 m2 with the deepest excavation depth of 9 m. The majority of the excavation intersected on heterogenous landfills, resting on top of Lisbon's Vulcanic Complex. The site is located in Miraflores, near Monsanto Forest Park. Surrounding the site there are public roads and other building constructions taking place. To minimize settlements around the excavation a secant pile wall was built, and braced either by a concrete strip slab or temporary ground anchors, the north elevation has an open filed, which was used to create a slope, to access to the final excavation level allowing an easy way to remove the soil.

RÉSUMÉ: Cet article traite des solutions de conception et de construction pour les structures périphériques de soutènement en terre, nécessaires à la construction d'un lotissement, qui est un complexe immobilier de deux bâtiments de 14 étages chacun. L'empreinte d'excavation est d'environ 6000 m2 avec une profondeur d'excavation maximale de 9 m. La majorité des fouilles ont été recoupées sur des décharges hétérogènes, reposant sur le complexe vulcanique de Lisbonne. Le site est situé à Miraflores, près du parc forestier de Monsanto, autour du site, il y a des routes publiques et d'autres constructions en cours. Pour minimiser les tassements autour de l'excavation, un mur de pieux sécants a été construit et contreventé soit par une dalle de béton ou des ancrages au sol temporaires, l'élévation nord a un dépôt ouvert, qui a été utilisé pour créer une pente, pour accéder au niveau d'excavation final permettant un moyen facile d'enlever le sol.

Keywords: Munich Walls, Berlin Walls, Mixed Walls, Deep Excavations

1 INTRODUCTION

This paper addresses the peripheral earth retaining walls solution for the construction of a private condominium with two buildings of 14 storeys high on a plot with over 6000m^2, and three levels below ground, with a maximum excavation over 9 m.

The main constraints of earthwork were the tight deadline to execute the excavation work, the geological and geotechnical environment, and the proximity to nearby constructions. Figure 1 presents the locations of the constructions site.

Figure 1 - Site Plan plus ML path.

This article aims to address in more detail the solutions of peripheric retaining walls taking account than in west side is not possible to execute provisory anchors and that 60 000 m3 needed to be excavated in less than 6 months.

2 MAIN CONSTRAINTS

From the site geological and geotechnical investigation, it was possible to confirm that the site was located on the Lisbon's Vulcanic Complex, covered by a heterogeneous landfill deposit layer. In Figure 2 is presented the geological chart of Portugal, 34-D Lisbon.

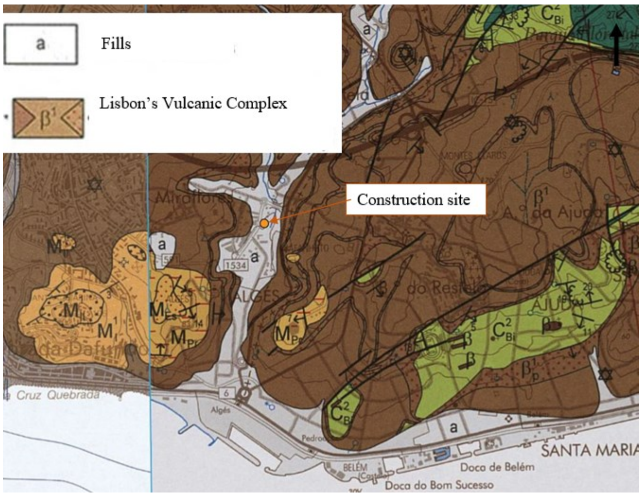

Figure 2 - Geological Chart of Portugal.

The laboratory testing on the basalt rock mass shows a high modulus of elasticity and compressive resistance, in line with historical results in excavations in the area.

The existence of two PT (Transformation Post) constraints the solution, as electric cables with median tension were active in the area with no possible deactivation.

At last, the main constraint was the time schedule for the execution of the peripheric retaining walls and the excavation, in less than 6 months.

3 CONCEPTUAL SOLUTIONS

The conceptual solution for the excavation was developed considering time restrictions, space limitation and the type of geotechnical field. The design solution was an overlapped pile wall consisting on 500mm diameter piles spaced 400mm. The primary piles consist of unreinforced piles, which reach the basalt rock mass. As for the secondary piles, which are reinforced, they have an embedment length of 2,5 m on the rock mass underneath the maximum excavation depth, ranging from 7 to 12 m. Figure 3 and 4 show pile wall section.

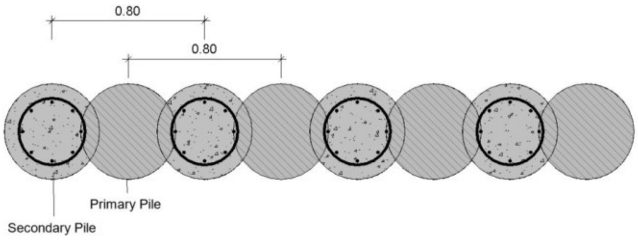

Figure 3 - Overlap pile wall.


Figure 4 - Overlap pile wall preparation in situ with marks.

The overlapped pile wall was horizontally supported by two types of solution. In South, North and East side, provisory anchor levels were considered with pretension. Figure 5 shows the section solution.

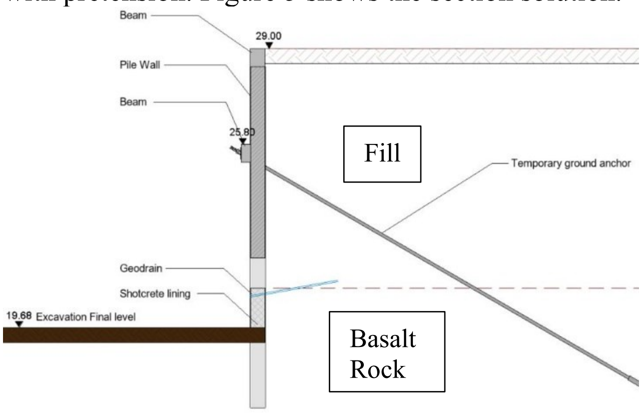

Figure 5 - Section solution for North, South and East side.

On the West side, provisory anchors were not possible due to the proximity to adjacent lot excavation.

Due to time restrictions, the solution for horizontal support was slab bed solution. In this solution, the slab is partially executed before the full excavation. The pile wall has enough stiffness to allow 3m excavation with no support, followed by the execution of the slab bed that is fixed in the extremities by the pile wall showed in Figure 5 and or by sub vertical micropile elements, as Figure 6.

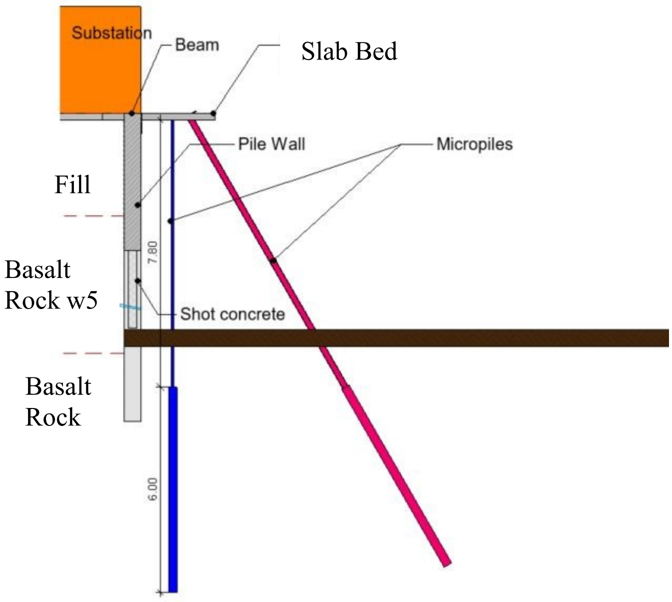

Figure 6. Section solution for West side.

Due to length of the slab bed, vertical support was needed to control the deformation of the slab bed in excavation phase.

Taking in consideration the overall force in the slab bed in excavation phase, additional reinforcement bars were considered, in addition to the reinforcement bars predicted in the stability project. Figure 7 show BIM model for the excavation.

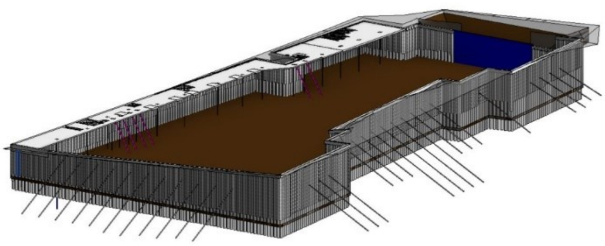

Figure 7 - BIM model.

The design of the peripheric retaining wall was developed with Plaxis 2D models and Autodesk Robot models for the strip slab (figure 8 and 9).

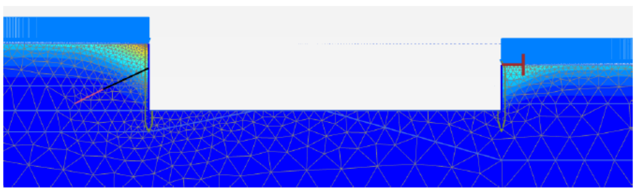

Figure 8 - Plaxis 2D model.

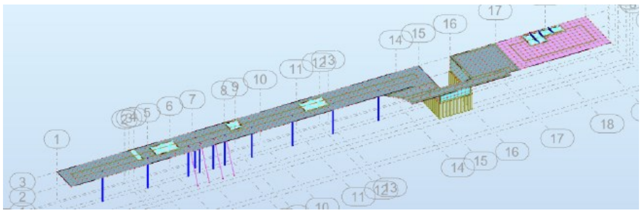

Figure 9. Autodesk Robot Model.

Due to the complexity of the geotechnical field, it was executed a survey and observation plan in situ, to confirm the design criteria and evaluate the behaviour of peripherical retaining wall structure.

4 EXECUTION

The solution of pile wall allows the execution of the peripheric retaining wall with multiple site fronts, with 5 pile machines at the same time.

Intended to support the high intensity work of 5 pile machines, 2 auto concrete mixers were in site to support the piles machines.

Figure 10 - East Side with 2 pile machines.

Figures 11, 12, 13 and 14 show the retaining pile wall solution at bottom excavation or during excavation.

Figure 11 - West Side solution with slab bed.

Figure 12 - East Side solution with provisory anchors.

Figure 13 - South Side solution with provisory anchors. (definitive structure already in place).

Figure 14 - West Side (Left) and East Side (Right) solution at bottom excavation. (definitive structure already in place).

5 MONITORING AND SURVEY PLAN

The instrumentation adopted in situ consisted of a wide range of devices, of which the following stand out:

- Topographic targets in the peripheral retaining structures, placed during their execution.
- Topographic targets in the surrounding tower cranes, installed before the start of excavation work.
- Piezometers, installed to control the position of the water table.
- Inclinometers, installed to control the evolution of horizontal displacements of the terrain at depth;
- Load cells, used to evaluate loads on temporary anchors;

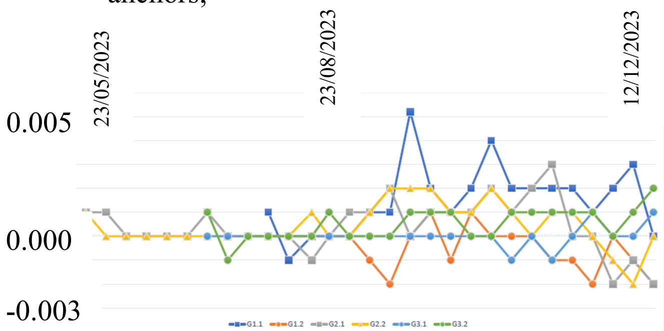

Figure 15 - Towers Cranes vertical deformation (m).

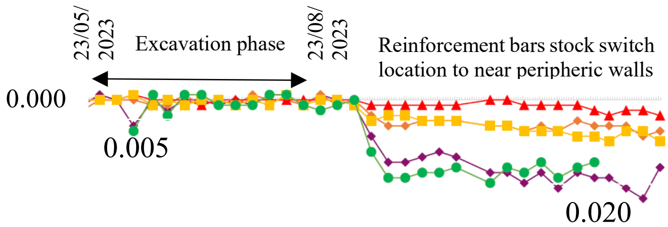

Figure 16 - Horizontal deformation pile wall (m).

The survey and observation plan confirmed the design criteria of the project. Nevertheless, during the construction of the structure from the bottom of the excavation, the reinforcement bars stock switch to a very close area near the peripheric retaining wall, which results in a significant horizontal deformation, yet, under accepted values.

6 CONCLUSIONS

This article highlights the mains solutions of peripheric earth retaining walls with time constrains and nearby excavations constrains. Risk management, conceptual confirmation, assessment of good behaviour of the structures were only possible with the execution of a Monitoring and Survey Plan.

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Excavation, earth retaining solutions and facades underpinning of a historic building in Estoril, Portugal

Excavation, solutions de soutènement des terres et reprise en sous-œuvre des façades d´un bâtiment historique à Estoril, Portugal

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ABSTRACT: This paper aims to present the solutions for excavation, peripheral earth retaining walls and centenary facades underpinning of a future residential building, located in Estoril, Cascais, Portugal. Firstly, to preserve the building´s facades, a temporary structural system was installed, allowing the facades bracing, ensured by the execution of horizontal steel distribution beams and vertical steel frames, founded on vertical micropiles. Secondly, the facades were underpinned through the execution of two twin underpinning beams, connected by pre-stressed bars, founded also on vertical micropiles. For the execution of the underground floors, it was necessary to excavate a maximum of 9 m under the original building´s foundations, using the king post walls earth retaining technique braced horizontally by steel props, ensuring the compatibility of the underpinning and the retaining facades solutions. Lastly, the monitoring and observation of the peripheral retaining structure and facades is also pointed out, encompassing detailed analyses and comparisons with the initially estimated values from the design phase.

RÉSUMÉ: Cet article vise à présenter les solutions pour l'excavation, les murs de soutènement périphériques et la reprise en sous-œuvre des façades centenaires d'un futur immeuble résidentiel situé à Estoril, Cascais, Portugal. Tout d'abord, pour préserver les façades du bâtiment, un système structurel temporaire a été installé, permettant le contreventement des façades assuré par la réalisation de poutres de distribution horizontales en acier et de cadres verticaux en acier, fondés sur des micropieux verticaux. Deuxièmement, les façades ont été soutenues par l'exécution de deux poutres de soutènement jumelles, reliées par des barres précontraintes, également fondées sur des micropieux verticaux. Pour l'exécution des étages souterrains, il a été nécessaire d'excaver jusqu'à une profondeur maximale de 9 m sous les fondations originales du bâtiment, en utilisant la technique de soutènement des murs à paroi berlinoise, contreventée horizontalement par des étais en acier, assurant la compatibilité entre le soutènement et les solutions de soutènement des façades. Enfin, la surveillance et l'observation de la structure de soutènement périphérique et des façades sont également soulignées, englobant des analyses détaillées et des comparaisons avec les valeurs initialement estimées lors de la phase de conception.

Keywords: Earth retaining structures; king post walls; underpinning; facades; micropiles.

1 INTRODUCTION

The present article outlines the solutions adopted for the retention and underpinning of an existing old facade, as well as for the excavation and peripheral earth retaining structure, enabling the construction of an underground parking lot for a future residential building.

Within the intervened plot, there was a vacant building whose main facades were, for the most part, preserved. The exterior walls were composed by ordinary stone masonry, with a thickness of 0.80 m, decreasing in height. Generally, the facades exhibited a considerable state of degradation, particularly on the southern facade, with the possible risk of partially collapsing. Figure 1 presents an aerial perspective of the intervention site before the construction work´s.

Figure 1 – Aerial view of the intervention site, in its current state.

The new building will be situated on Avenue Senhora Monte da Saúde, covering an approximate ground area of 670 m^2. It will comprise a parking lot, a ground floor, three elevated floors and a slopping roof. The intervention area, within which the building is situated, encompasses around 1446 m^2. Figure 2 depicts a virtual view of the future residential building.

Figure 2 – Virtual view of the future residential building.

2 MAIN CONSTRAINTS

2.1 Geological and geotechnical conditions

To characterize the geotechnical behaviour, associated with the soils present at the site, a prospecting campaign was conducted involving the execution of two mechanical boreholes accompanied by Standard Penetration Tests (SPT) and the collection of samples for macroscopic classification. Additionally, one hydraulic piezometer was installed in one of the boreholes.

According to the results obtained, the geological structure of the site is characterized, superficially, by the presence of Landfill Deposits (At) mainly composed of rocky blocks and clayey sands with an average thickness of 1.0 m. Notably, no standard penetration test was conducted on these modern anthropogenic materials. Subsequently, underlying these fill deposits, Cretaceous formations (C^1_{Ba} e C^1_A) are manifested at this location by three geotechnical horizons. The conducted borehole surveys unveiled the presence of clays and sandy clays (ZG_{2A} - C^1_{Ba}), with variable thickness ranging between 7,50 m and 10,50 m and N_{SPT} values ranging from 15 to 41 blows. Addtionally, fine clayey sands to sandy clays (ZG_{2B} - C^1_{Ba}) were encountered at depths varying from 7,5 m and 12,0 m, with N_{SPT} values spanning from 23 to 46 blows. Lastly, a lower stratum was characterized by moderately to highly altered and fractured limestones (ZG_{2C}-C^1_A), interspersed with fine-grained sandstones, in which Rock Quality Designation (RQD) values ranged from 32% to 49%. This unit was identified at the dephs of 10.0 m and 11.8 m.

In terms of hydrogeology, there is no water level at the excavation site according to the conducted survey, which should not interfere with the local hydrogeological regime.

2.2 Neighbourhood conditions

The excavation site is situated in an urbanized area, bounded by Avenue Senhora Monte da Saúde to the West, and several other neighbouring constructions. To the North, the plot is bordered by two residential buildings with two and three elevated floors, respectively, and both without underground basements. To the South, it is delineated by the backyard and annex of a neighbouring plot. To the East, it is surrounded by a one-story church, also without underground basements.

Additionally, it is worth noting the presence of party walls along the lot boundaries and the proximity of an existing wall, located inside the lot, to the proposed king post wall solution at the rear of the façade to be preserved. In Figure 3 it is possible to observe the main boundaries of the excavation site.

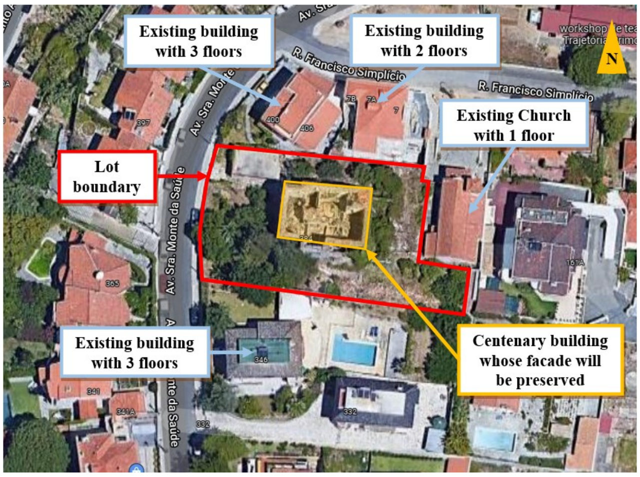

Figure 3 – Main boundaries of the excavation site.

3 ADOPTED SOLUTIONS

3.1 Facades retention and underpinning

Given the facades high deterioration level, particularly on the southern side, and the need to demolish/dismantle a portion of it, it was decided that, prior to the facades retention work, an extensive treatment to the existing cracks should be undertaken through the injection of grouts based on hydraulic lime. Subsequently, steel angle profiles were installed at the corners of the facades to be preserved, and three levels of tensioned 26 mm galvanized tie rods were installed to ensure the maximization of safety during the facades retention works (Figure 4).

Figure 4 – Prior facade retention works, involving the installation of steel angles and steel tie rods per floor.

The facade retention structure was composed by steel distribution beams, which had the primary objective of equalizing horizontal loads on the wall and directing it towards 8 bracing frames, particularly loads induced by wind action. These distribution beams were constituted by HEB 180 and UPN 260 steel profiles, while the bracing frames were constructed using steel trusses comprised by HEB 240 vertical columns, HEB 200 horizontal beams and LNP 150x150x10 diagonal steel angles.

In order to provide an appropriate foundation with adequate stiffness and strength for the bracing frames of the retention structure, considering tensile and shear forces to which the structure might be subjected, reinforced concrete massif´s were executed, indirectly founded on micropiles. The micropiles were executed using N80 Ø139.7x10mm profiles with external connections between sections and sealed through the Repeated and Selective Injection system (IRS), in competent and geologically stable ground, with N_{SPT} > 60 blows.

To underpin the facades, a system composed by two rows of micropiles was designed, connected by two underpinning beams (one on the exterior and another on the interior of the existing walls), joined by prestressed Gewi type bars. In Figure 5 , the defined facades retention and underpinning solution can be observed.

Figure 5 - Facades retention and underpinning solution.

Additionally, given the high deterioration level of the facades, there was the need to reinforce the stone masonry walls by executing a reinforcement wall with a thickness of 8 cm. This wall consisted of a reinforcement mesh and shotcrete, executed from the interior and securely anchored to the existing facades (Figure 6). Upon completion of the facades retention and underpinning works, excavation and peripheral containment activities were initiated.

Figure 6 – Strengthening of stone masonry walls with a reinforced concrete layer.

3.2 Excavation and peripheral earth retaining structures

The excavation and peripheral earth retaining structures were executed following the King Post Wall technology, with careful consideration given to compatibility with the facades retention and underpinning works. This solution involved a phased construction, from top to bottom, of reinforced concrete panels. The process commenced with the execution of the primary panels, followed by the secondary, and ultimately, the tertiary panels.

The panels were horizontally braced by provisional elements such as steel props, tie rods and temporary pre-stressed ground anchors, and vertically supported by micropiles. The bracing elements were composed by HEB 180 and HEB 220 steel profiles, braced by HEB 120 profiles with S275 JR steel grade. In convex corners, resistance to impulses was ensured by pre-stressed Gewi tie rods.

The executed ground anchors comprised 4 strands, each with a diameter of 1.5 cm, and with a tensile load of 450 kN. The bonded lengths were accomplished using the IRS system, incorporating double obturators and non-return valves, with a minimum hole diameter of 200 mm and minimum length of 4.0 m. To prevent potential intersections with existing infrastructures and structures, as well as to allow proper sealing in competent and geologically stable soil, these anchors were inclined at 30.0º and 35.0º horizontally.

Vertical micropiles were designed to support the vertical loads imposed on the peripheral earth retaining structure, particularly by the self-weight and by the vertical component of the ground anchors. These elements were executed using N80 Ø139.7x10.0 mm tubular profiles with external joints, made of high-strength steel (f_{syd} > 560MPa), placed inside a 200 mm hole diameter and bonded for a length of 6.0 m, using the IRS system. The micropiles were connected at the top by capping beams and were generally embedded in the king post walls, which had a thickness of 0.30 m.

In Figure 7, the excavation and peripheral earth retaining solution are depicted.

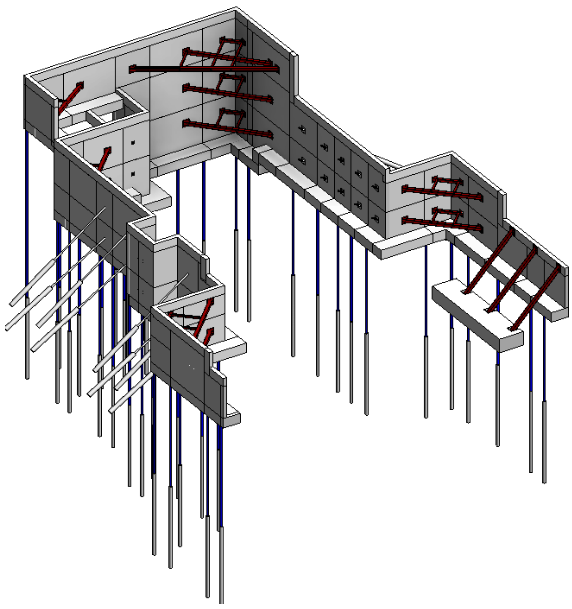

Figure 7 – Excavation and peripheral earth retaining solution (BIM model in software Revit).

4 GEOTECHNICAL AND STRUCTURAL DESIGN

4.1 Facades retention and underpinning

The behaviour of the facades retention structure was analysed in terms of forces, reactions and deformations using finite element models created in the Autodesk Robot Structural Analysis Professional 2022 software (Figure 8). This allowed the simulation of the wind effects, the most critical factor to this case. The facades bracing system was modelled in the program using "steel members" type elements with their respective mechanical properties. The micropiles were depicted as fixed supports.

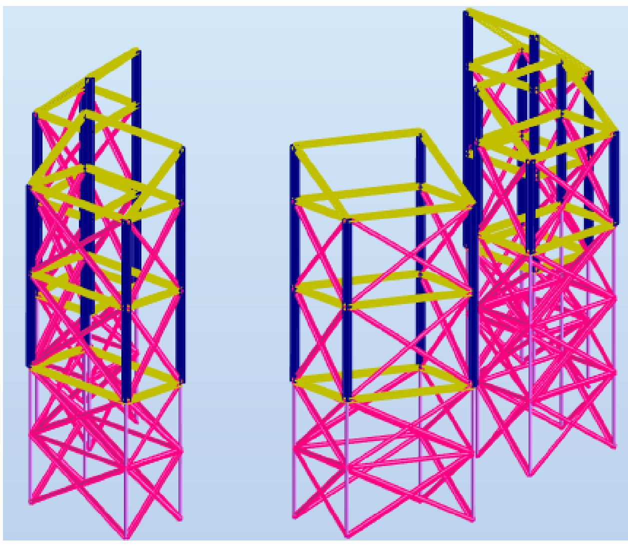

Figure 8 – Facades retention structural model, developed in Robot software.

4.2 Excavation and peripheral earth retaining structures

The assessment of the presented peripheral earth retaining solution in terms of forces and deformations was conducted for all major construction phases using a finite element program, PLAXIS 2D. A constitutive model was developed to simulate the soil behaviour characteristics of a "Hardening Soil", considering a nonlinear constitutive relationship and variations in the soil's stiffness under applied stress states. In the context of soil geomechanical parameterization, the reference values obtained from the geological-geotechnical study were incorporated in PLAXIS software, to simulate the soils behaviour, as detailed in Table 1.

Table 1 - Geotechnical soil parameters.

Soil Type	ZG_1	ZG_{2A}	ZG_{2B}	ZG_{2C}
N_{SPT}	-	15-41	23-46	> 60
Υ [kN/m^3]	19	20	20	21
Φ [º]	25	-	35	35
c_u/c' [KPa]	-	90	1	5
E_{50}^{ref} [MPa]	7,5	30	50	180
E_{ur}^{ref} [MPa]	22,5	90	150	540
m [-]	0,5	0,7	0,9	1

<u>Soil Type</u>: ZG_1 – *Loosely compact sandy-clay fill*; ZG_{2A}– *clay and sandy clay soil;* ZG_{2B} – *fine clayey sand to sandy clay soil;* ZG_{2C} – *fractured limestones interspersed with fine-grained sandstones.*

<u>Soil Parameters</u>: *Υ – Volumetric weight; Φ' – effective friction angle; E_{50}^{ref} – elastic modulus corresponding to 50% of the ultimate tensile stress defined for a reference pressure (p_{ref} = 100kPa); E_{ur}^{ref} - elastic modulus for unloading/reloading at a stress level equal to the reference pressure (p_{ref} = 100kPa); m – power that controls the stress dependency of the elastic modulus.*

In the developed models, the king post walls and micropiles were modelled using "plate" elements with elastic behaviour. However, the zones corresponding to the free length and the bond lengths of the micropiles were modelled using "embedded beam row" elements. The temporary ground anchors were modelled using "node-to-node anchor" elements and their respective bond lengths were modelled using "embedded beam row" elements. The steel props were modelled using "anchors", with the equivalent stiffness to the defined bracing system.

The behaviour of the peripheral retainment structure was analysed for the main excavation phases, assessing crucial design parameters such as forces of the containment structures, deformations, stress states and stability of the soils to be contained. The bonded lengths of the micropiles and temporary ground anchors were determined according to the methodology proposed by Bustamante and Doix (1985). Figure 9 and Figure 10 present the deformed mesh of the developed PLAXIS models.

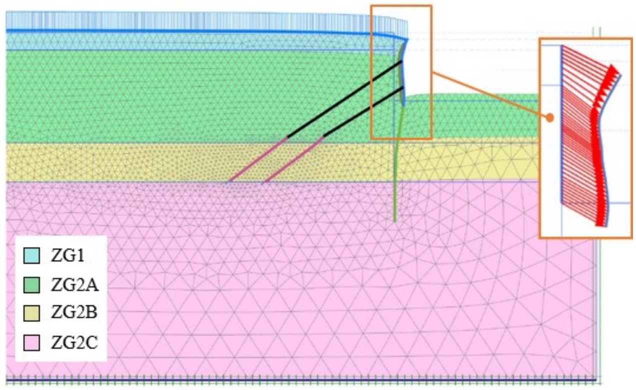

Figure 9 – Deformed mesh of the excavation´s base level phase - King post walls braced by pre-stressed ground anchors.

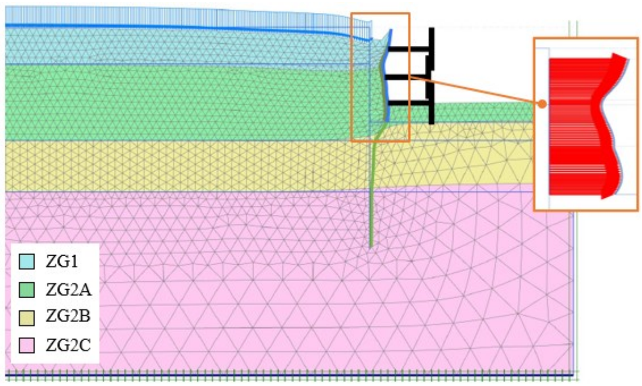

Figure 10 – Deformed mesh of the excavation´s base level - King post walls braced by steel props.

The maximum horizontal displacement estimated for the king post wall supported by two levels of pre-stressed ground anchors was 10,90 mm, for a total excavation height of 5,90 m (L/540), meeting the ELS requirements.

For the scenario in which the earth retaining structure is braced by three levels of steel props, for a total excavation height of 8,20 m, the maximum horizontal displacement estimated was 9,40 mm (L/870), meeting the ELS requirements.

5 MONITORING AND SURVEY PLAN

The established Monitoring and Survey Plan (MSP) on site had as its main objective to ensure the safe execution of the demolition, retention and underpinning works of the facades, as well as the peripheral earth retaining structure of the underground floors. Additionally, it aimed to analyse the behavior of neighboring structures and infrastructures during the construction works.

In the initial phase, for the control of displacements, rotations and measurement of masonry wall crack openings during the demolition and reinforcement of existing structures, an automated monitoring system was implemented for a 2-month construction period.

The entire field data reading and acquisition system were fully automated, with readings taken every 1 hour. Sensors were installed for data acquisition and transmission to a wireless web platform. Figure 11 illustrates the application of the web monitoring system for this specific construction project.

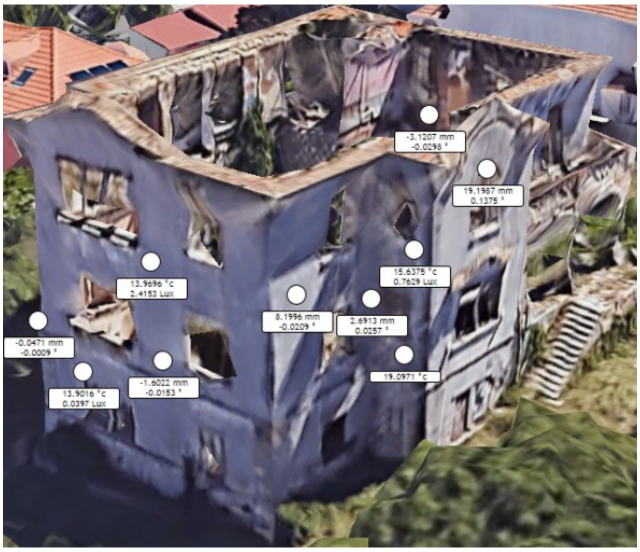

Figure 11 -Aerial view of the intervention site (Google Earths image) – Web Monitoring System

The implementation of this automated monitoring system not only enabled a more accurate and systematic control of the behaviour of all critical structures, but also streamlined the adjustment of construction methodologies during the execution of the new structures, potentially resulting in enhanced productivity.

The implemented MSP on the existing facades included the following fully automated sensors/instruments:

- 12 triaxial tiltmeters for measuring rotations and displacements of the preserved facades and neighboring structures and infrastructures;
- 4 vibrating wire crackmeters (strain gauges) for measuring crack openings;
- 40 manually read crackmeters for measuring crack openings;
- 2 Gateways, which correspond to the data collection and network data transmission system via 3G;
- 2 solar panels for energy supply to the Gateway.

In the second phase, for excavation and peripheral earth retaining construction works, the following instruments were installed, for a 6-month construction period:

- 14 triaxial tiltmeters placed on the king post walls, to measure rotations and displacements;
- 4 electric load cells, to measure the pre-stressed loads applied to the temporary ground anchors;
- 1 inclinometer positioned behind the peripheral earth retaining walls, to measure horizontal and vertical displacements of the retaining walls;
- 1 piezometer to measure underground water pressure.

Based on the conducted modelling, alert and alarm criteria was established for all instruments and monitored structures.

6 CONCLUSIONS

In the scope of this article, it is expected to demonstrate the technical efficiency of the facades retention and underpinning solution as well as the king post walls solution, braced by various structural elements, such as struts and temporary ground anchors. Furthermore, it is expected that the peripheral earth retaining solution present´s actual displacements lower than those estimated in design and display a highly stable behaviour during the excavation works currently underway.

ACKNOWLEDGEMENTS

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Industrial warehouse retaining walls and pavements strengthening at Quinta do Adarse, Alverca, Portugal

Renforcement des murs de soutènement et des chaussées d'un entrepôt industriel à Quinta do Adarse, Alverca, Portugal

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ABSTRACT: The aim of this article is to present the solutions for the main retaining walls and pavement strengthening, adopted in an industrial warehouse at Quinta do Adarse, in the proximities of the Tagus River right bank, at Alverca, Portugal. The existent cantilever reinforced concrete wall with 300m of length and 6m of maximum height, was strengthened with a cap distribution beam due to excessive deformation. The cap distribution beam was supported by inclined steel struts and founded on micropiles. At the warehouse pavements, some differential settlements were detected around the stiffer structural foundations, mainly due to the water inflow at the pavement earth fill foundation, which required a strengthening solution. Therefore, injections were used to avoid the replacement of the pavement and minimize the impacts on the warehouse's normal operation.

RÉSUMÉ: Le but du présent article est de présenter les solutions pour les principaux murs de soutènement et le renforcement de la chaussée, adoptées dans un entrepôt industriel à Quinta do Adarse, à proximité de la rive droite du Tage, à Alverca, Portugal. Le béton armé en porte-à-faux existant, qui est un mur de 300 m de longueur et 6 m de hauteur maximale, a été renforcé avec une poutre de répartition de couverture en raison d'une déformation excessive. La poutre de distribution du chapeau était soutenue par des entretoises en acier inclinées et fondée sur des micropieux. Au niveau des revêtements de l'entrepôt, certains tassements différentiels ont été mobilisés autour des fondations structurelles plus rigides, principalement en raison de l'afflux d'eau au niveau des fondations en terre de la chaussée, ce qui a nécessité une solution de renforcement. Par conséquent, des injections ont été utilisées pour éviter le remplacement du revêtement et minimiser les impacts sur le fonctionnement normal de l'entrepôt.

Keywords: strengthening; wall; micropiles; pavement; injection

1 INTRODUCTION

The aim of the present work is to present the adapted solutions for strengthening the retaining walls and pavements of an operating industrial warehouse, by using micropiles, cap distribution beams, and ground consolidation with polyurethane resin injection.

The reinforcement and strengthening of retaining walls and pavements at operating industrial warehouses require solutions with minimum impact on the normal operation.

Strengthening solutions with micropiles have advantages due to their diversity and execution possibilities for small spaces with difficult access, or sensitive to noise and vibrations, as well as soils with the presence of boulders, rock, concrete, and voids. The micropiles are also distinguished by the fact that they can be carried out on slopes (Barbosa, 2019).

To strengthen the pavements, a non-invasive technique may be required, with the physical-mechanical characteristics of the injected material remaining unchanged over time, so the solution of injecting expansive resin to treat the pavement foundation ground allows for a safe and practical intervention (Favaretti et al., 2014).

2 THE INDUSTRIAL WAREHOUSE

The site is a warehouse with an area of 20,250 m^2 (270 m x 75 m), built in the 1990’s at Quinta do Adarse, Alverca, Portugal (Figure 1). The retaining wall is a reinforced concrete cantilever structure with 6 m of height extending over 320 m of length. The wall was designed to support the difference of approximately 4 m, existing between the lot where the warehouse is located and the adjacent areas.

The pavements with 5.0 m x 5.0 m panels were built with 0.10 m concrete industrial ground floor reinforced with steel mesh AR 30. The pavements were founded on a 0.20 m thick layer of crushed stone or aggregates fill, separated from the underlying ground, most likely pre-existing landfills by a geotextile blanket. As it was an operating industrial warehouse under operation (Figure 2a), the stresses on the pavement are related to the weight of the materials stored on shelves (Figure 2b) as well as to the traffic of loading and transport equipment (forklifts).

Figure 1. Aerial view of the site (Google Earth).

Figure 2. a) External view of the warehouse and b) Internal view of the materials stored on shelves.

3 GEOLOGICAL AND GEOTHECNICAL PROFILE

The site under study is located, from a geological point of view, over an alluvium (a) and river terrace deposits (Qf), from the Quaternary, associated with the Tagus River basin, according to Sheet 34 B - Loures of the Geological Chart of Portugal at scale 1/50.000. It is constituted of clays, silty clays, sands, and silty sands with pebbles and sometimes conglomerates with limestone elements. These formations are based on Miocene layers of sands, sandstones, mudstones, and limestones from the Tortonian (MT).

The solutions proposed are based on the results of the tests carried out in the geological-geotechnical study, and in the values presented in the reference bibliography for alluvial soft silty-clay in Portugal (Esteves, 2014).

4 PATHOLOGIES

4.1 Retaining wall pathologies

There are occasional and localized phenomena of instability in the walls leading to cracks and excessive deformation (Figure 3).

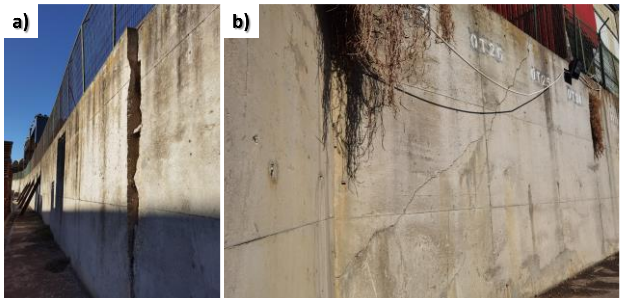

Figure 3. Pathologies at the retaining wall: a) excessive deformations b) cracks.

It should be noted the presence of four silos on the upper level, whose foundation solution was unknown. These structures may have increased ground pressure on the retaining walls. It can also be seen that, in some cases, the walls have already been reinforced in the past, with steel bracing solutions (Figure 4a) and nails or using tie rods (Figure 4b), which failed to stabilize the wall.

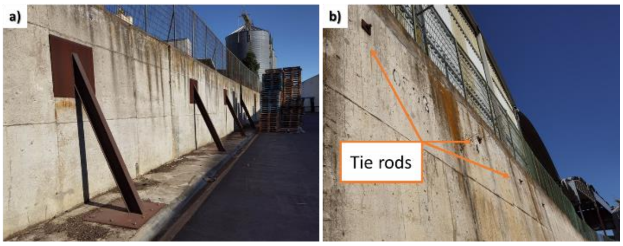

Figure 4. a) Steel bracing b) nailing or tie rods on the retaining wall.

4.2 Pavements pathologies

The loads to which the warehouse pavements are submitted are due to the weight of the materials stored on the shelves as well as the loads associated with the passage of loading and transport equipment (forklifts). It was seen that the pathologies worsened in the southern part of the warehouse indicating an increase in the pavements loads (Figure 5).

The pathologies, which required the adapted strengthening solutions, were identified in the pavements, facades, unloading dock, and stairs.

Close to the facades and columns, the presence of cracks in the concrete pavement was observed, as well as evidence of differential settlements between the pavement and the remaining structural elements (Figure 5a-b). On the walls of the east and west facades, which were built of prefabricated concrete slabs, horizontal cracks were also observed (Figure 5c).

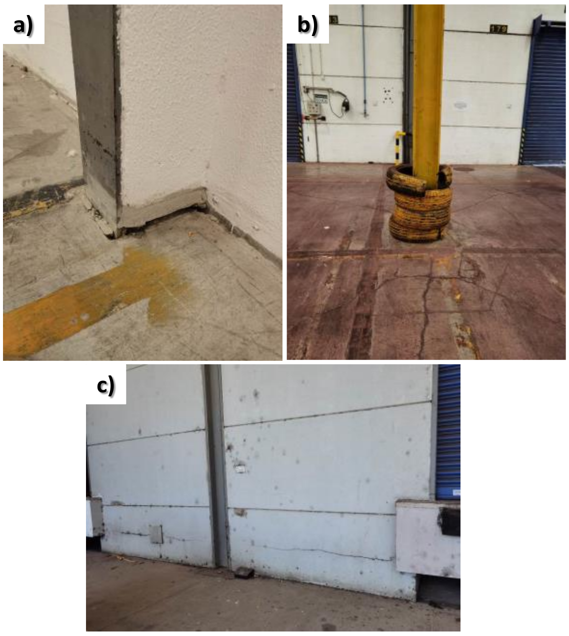

Figure 5. Main pathologies. a) differential settlements between the pavement and the facade, b) degradation of the pavement around a pile cap, c) cracking in the east side of the warehouse.

5 COMPRESSION STRENGTH TESTS ON CONCRETE

To find out the compressive strength of the existing wall concrete, six drillholes were made along the length of the wall, located at a height of 2.0 m perpendicular to the wall face. The collected specimens had a diameter of 82 mm and a length between 37.5 cm and 41.5 cm.

In general, the specimens were homogeneous, with a good distribution of limestone aggregate, but with some voids. These voids may indicate a lack of vibration during pouring.

Figure 6. Uniaxial compression-strength testing – UCS.

Uniaxial compression strength (UCS) tests were carried out on five of the collected specimens (Figure 6) to measure the mechanical resistance of the existing concrete wall. The uniaxial compression strength of the samples ranged between 16.56 MPa and 28.30 MPa.

6 STREGTHENING SOLUTIONS

6.1 Retaining wall strengthening

The proposed solution includes the construction of a bracing element with the aim of increasing the wall resistance to horizontal pressure. This element consists of a 0.45 x 0.70 m distribution beam at the wall's mid-height, which is designed to transfer the horizontal forces to inclined micropiles, with tubular section of Ø88.9 x 12.0 mm and Ø88.9 x 9.5 mm, spaced 4.50 m, in the direction of the wall's development. The micropiles had a variable length of between 14 m and 15 m, thus guaranteeing minimum sealing lengths of 8 m and 9 m in the ground layer (natural ground with a NSPT of more than 20 blows).

To accommodate the increased vertical load caused by the axial force on the inclined micropiles, capping and distribution beams, were also built. Vertical micropiles were installed, also spaced 4.50 m. The vertical micropiles had an estimated variable length of between 11 m and 13 m, thus guaranteeing minimum sealing lengths of 6 m and 8 m in the stiffer ground layer.

The boreholes for placing the micropiles were at least Ø220 mm, and the tubes were subjected to filling and sealing injection. The injection was carried out using an appropriate system – I.R.S. (Repetitive and Selective Injection), using non-return valves and a double plug, for the length corresponding to the sealing bulb.

As the beam is supported directly on the micropiles, part of the steel tube was exposed permanently. Therefore, the micropiles were properly protected against corrosion, with a sacrificial thickness, compatible with the conditions of the environment and the life of the structure. Figure 7 shows the typical cross-section of the proposed solution.

Sub-horizontal geodrains with a diameter of Ø50 mm, creped and wrapped in 300 gr/m^2 geotextile, with a length of 3 m, were also installed, to relieve hydrostatic pressures. In addition, cracks in the existing wall were filled with non-shrinking mortar. To prevent the reoccurrence of pathologies, AISI316 stainless steel staples were also installed, spaced 0.40 m, whenever the cracks were larger than 10 mm.

Compression tests were performed in the concrete to evaluate the state of degradation of the wall. The structure was initially modelled using the GEO5 software to represent the current situation of the existing reinforced concrete wall, which only resists the static conditions, with a unitary safety factor. Subsequently, a second model was developed also considering the actions correspondent to earth pressures and surcharger to estimate the respective efforts and, therefore, to calculate the reinforcement structure in accordance with current regulations.

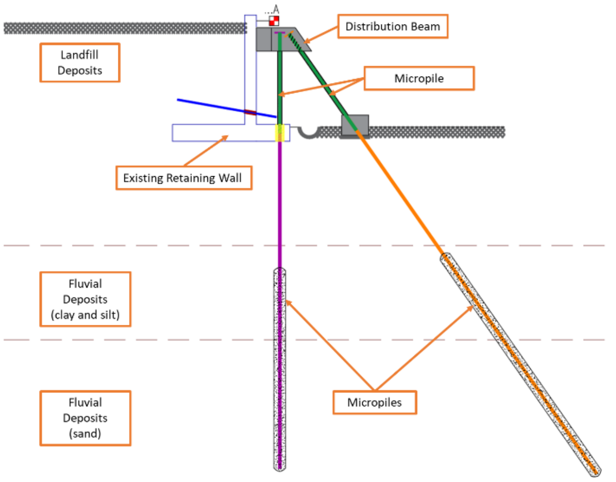

Figure 7. Cross section of the wall reinforcement solution

6.2 Pavement strengthening

The locations where internal erosion of the soil underlying the pavement was suspected a ground consolidation solution using the Uretek - Floor Lift type resin, or equivalent, was chosen. The resin has a density of 45 kg/m^3 (minimum) and expansion capacity up to 30 times its initial volume in free expansion.

This technique makes it possible to increase the compactness of the ground immediately below the pavement, without excavations, vibrations, or mechanical forces, thus minimizing the risk associated with possible damage to existing structures. The injections were made through small diameter holes, thus avoiding the need to build a new pavement structure (Uretek Floor Lift, 2023). In addition, this technology allowed to restore the pavement's planimetry and re-establish its contact with the ground (Figure 8).

In this context, the adapted solution consisted of drilling small diameter holes, arranged in a square grid of 1.20 x 1.20 m. Injection tubes were installed in these holes, approximately 6 and 12 mm in diameter. Through these tubes, a polyurethane resin was injected between the base of the industrial floor and up to approximately 1.0 m depth. In the first injection phase, the polyurethane resin, which is in liquid form, penetrates the voids in the soil and subsequently expands, sealing the surrounding soil, forming a rigid foam that increases the ground compactness. The injection process is monitored by reading equipment based on laser technology, indicating the increase in the soil's load-bearing capacity and the recovery of the differential settlements.

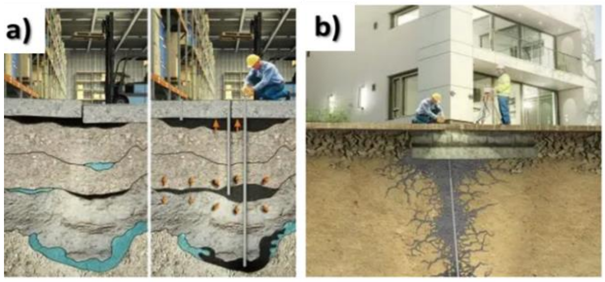

Figure 8. Injection system - Uretek Floor Lift (https://www.uretek.pt).

7 FINAL REMARKS

The aim of the present work was to present the designed solutions for strengthening main retaining walls and warehouse pavements. The strengthening solutions were defined considering the constraints, the severity of the detected pathologies and economic viability, with the aim of improving and guaranteeing the continued safe use of the warehouse area.

In the case of the retaining walls, the solution was associated to the construction of a bracing element for the existing support structure, which is a 0.45x0.70m distribution beam at the wall's mid-height, designed to transfer the horizontal forces to inclined micropiles. For the pavement, a ground consolidation solution of the Uretek - Floor Lift type was used, followed by drilling of holes in a square grid where polyurethane resin was injected.

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