

# Numerical Analysis of an Unsymmetrical Railcar Unloading Pit and Connection Trench

## ANALYSE NUMERIQUE D'UN PUIT NON SYMETRIQUE POUR L'EXTRACTION ET LA STOCKAGE DES MINERAUX

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**ABSTRACT:** A numerical analysis was performed simulating the deep excavation and dewatering effects on retaining walls of an unsymmetrical railcar unloading pit and trench. The pit has a depth of 22 meters and an internal diameter of 45 meters. The trench is non-collinear with the pit center and it has 12 m width by 122 m length. A 3D Finite Element Model using PLAXIS software was conducted in order to better estimate both general and local effects in the design of the pit's structural elements, mainly due to the singularities introduced by the trench opening on the west side of the pit wall, as well as the trench excavation. Both geotechnical and hydrogeological characteristics of the site were taken into consideration, as well as the main construction stages, covering the excavation sequence and the groundwater inflow analysis.

**RÉSUMÉ:** Dans cet article ont présent une analyse numérique qui a été développe pour simuler une excavation profonde et l'effet du rabattement de la nappe phréatique dans les murs de soutènement d'un puit non symétrique pour l'extraction et le stockage des minéraux avec l'aide des wagons. Avec une profondeur de 22m et un diamètre intérieur de 45m, le puit circulaire a une intersection de 12m avec une zone en couloir rectangulaire d'accès au puit, placée de façon non aligné avec le centre du puit. Un modelé 3D a été développe avec l'aide du software PLAXIS tenant l'objective d'estimer numériquement les effets locaux e globaux dans le dimensionnent des éléments structurels du puit, en particulier les effets de l'excavation dans la zone d'ouverture correspondent à l'intersection avec le couloir d'accès. Les caractéristiques géotechniques e hydrogéologiques ont été considérés dans le modèle numérique, bien aussi comme les phases des excavations plus importants et l'analyse de l'entrée de l'eau à l'intérieur du puit.

**Keywords:** deep excavation; 3D finite element model; soil-structure interaction.

## 1 INTRODUCTION

Deep excavations often comprise a very complex soil-structure interaction. A diversity of aspects must be assessed in order to achieve a safe design, such as geometry, construction sequences,

water flow and stress/deformation states. In common practice, 2D plane strain or axisymmetric finite elements models are widely used to analyse geotechnical structures.

However, usually due to geometric singularities, some structures require a three-dimensional analysis in order to ascertain a more accurate

global and local soil-structure interaction (Hou et al. 2009). In this paper it is presented and discussed the numerical analysis of a circular pit deep excavation connected to a non-collinear trench, using both 2D and 3D finite element models.

## 2 SITE CONDITIONS AND STRUCTURE DESCRIPTION

### 2.1 Geological and hydrogeological conditions

An extensive site investigation campaign was carried out in order to assess the ground stratigraphic profiling and evaluate the geomechanical and hydrogeological materials' properties. The results of the campaign showed the existence of a superficial alluvium unit, of about 7.5m thickness. Underneath that formation, was observed a sequence of silty clayey material with increasing stiffness up to 29 m depth. Between the silty clay materials and the bedrock formation, a sandy layer was discovered with a thickness ranging from 4 to 12 m. Ordovician bedrock was at 40 m depth, mainly composed by very stiff to hard dark grey to black claystone.

The interpretation of the sand layer behavior was fundamental to the structure's design due to its high permeability when compared with the remaining formations. The upper alluvium and lower clayey layers were characterized by permeabilities of  $10^{-6}$  m/s, while the sandy layer permeability was evaluated as  $10^{-5}$  m/s. Being the sandy layer confined between low permeable soil layers, the water in it is subjected to positive pressures which could affect the stability of the base of the excavation by heave effect (Ou 2006).

Water level measurements were assessed continuously indicating a ground water table level ranging between 1 to 6 m depth. This value is related to ocean ties.

### 2.2 Retaining wall structures

The structure is part of an industrial complex for aluminum ore extraction and distribution, formed by a main railcar unloading pit and the conveyor system trench. The pit has a circular shape with a diameter of 44.5 with an approximate maximum depth of 22 m. A 12 m width trench is connected from the bottom of the pit to the surface level, over a length of 122 m. Due to site geological, geotechnical and hydrogeological conditions, particularly the position of the phreatic level, a 1 m thick diaphragm wall was designed allowing the excavation of both pit and trench. An isometric representation of the retaining structure is illustrated in Figure 1.

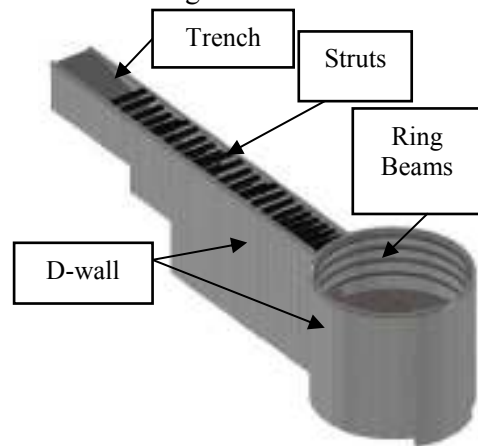


Figure 1. Pit and trench retaining wall isometric view.

An approximately circular pit was shaped with the design of consecutive rectangular D-Wall panels. In order to ensure the D-Wall panels' behave as a continuous structure and also to increase the radial stiffness of the retaining structure, five horizontal levels of reinforced concrete rings were designed (Fig. 2). The fifth ring, placed below the mat foundation level, provided positive support of the D-Wall panels and carry circumferential compression induced by the trench opening. As an additional security measure to prevent eventual lateral water flow through the retaining structure, 800 mm diameter

jet-grouting columns, at the backside of each panel joint, were executed.

In order to avoid lateral instability of the pit's D-Wall panels, on the trench side, four bracing slabs were executed in the backside of those to ensure the transfer of the outward thrust to the trench's D-walls. The horizontal equilibrium of the trench retaining structure was assured by a set of concrete struts (Fig. 3).

In order to avoid hydraulic failure of the low permeable silty clay layers during the excavation works, 7 pressure relief vertical drillings were executed inside the pit and the trench. This drills intersect the sandy layers located beneath the silty clay materials, relieving the hydrostatic pressure of the confined aquifer. To minimize the water flow towards the excavation, the D-Wall panels were design with an average depth of 45 m, assuring a 3 m embedment in the low permeability Ordovician formation.

To ensure the proper foundation of the future structures and to prevent water inflow in the permanent stage, a mat foundation was designed comprising both the pit and the trench. This assures the vertical equilibrium of the long term water pressure that will settle in the bottom of the mat element, with a set of self-drilling micropiles.

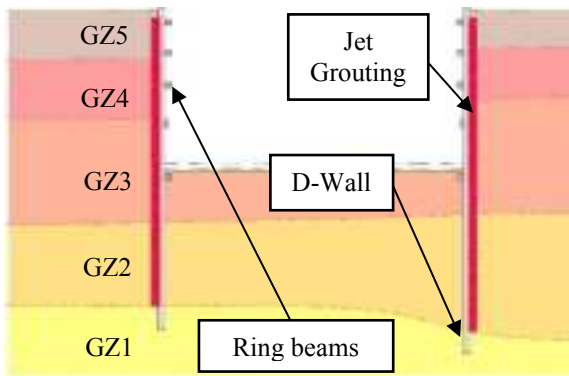


Figure 2. Pit retaining wall cross section.

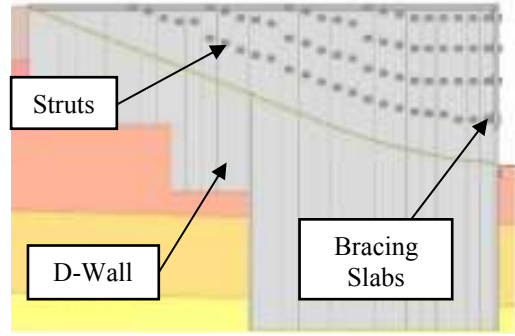


Figure 3. Trench retaining wall longitudinal cross section.

### 3 NUMERICAL MODELS

#### 3.1 General information

A series of 2D and 3D analyses were carried out using the finite element model program PLAXIS. As a first approach 2D models were developed and later used to validate the 3D global model results and to design the sections that were not under the influence of the pit/trench connection. The soil behavior was simulated with the Hardening-Soil model (Brinkgreve et al. 2017), an elastic-plastic model with a multi-surface yield criterion that simulate the increase of the soil's stiffness and it's deformation. The following tables (Tab. 1 and Tab. 2) summarize the main parameters that control the behavior of the soil formations. Due to the type of structure and execution technology, it was considered a soil-structure interaction angle 2/3 of soil friction angle, for all the soil layers.

Table 1 – Soil layers and permeability properties.

	M	ms <sup>-1</sup>
GZ5 - Alluvium	0-7.5	3.36x10 <sup>-6</sup>
GZ4 - Silty clay 1	7.5-15.5	1x10 <sup>-4</sup>
GZ3 - Silty clay 2	15.5-29.0	2.1x10 <sup>-10</sup>
GZ2 - Medium sand	29.0 – 41.0	1.6x10 <sup>-5</sup>
GZ1 – Claystone	>41.0	5.56x10 <sup>-7</sup>

Table 2 – Soil geomechanical parameters.

Soil	$\gamma_{\text{sat}}$	$c'$	$\phi'$	$E_{50}^{\text{ref}}$	$E_{\text{oe}}^{\text{ref}}$	$E_{\text{ur}}^{\text{ref}}$	m
Layers	kN/m <sup>3</sup>	kPa	°	MPa	MPa	MPa	-
GZ5	17	3	21	6	6	18	0.8
GZ4	22	8	34	30	30	90	0.8
GZ3	20	35	25	20	20	60	0.8
GZ2	21	0	38	100	100	300	0.5
GZ1	22	40	38	150	150	450	1.0

Despite the low-permeability layers, in typical excavation works, the critical condition is the long-term behaviour of the soils. Therefore, a drained analysis was performed for both excavation and permanent stages in a conservative design option.

Within each model a hydrostatic analysis was conducted considering the water level one meter below the soil surface. The water level variation and hydrostatic pressures were studied according the staged construction sequence and the relief vertical drilling. Additionally, flow analyses were performed on 2D models in order to determine the water flow at the bottom of the excavation and estimate the surface settlements beyond the wall.

### 3.2 2D retaining wall models

As a first approach, a 2D axisymmetric analysis of the pit's retaining wall was performed, using plate elements with the D-wall's equivalent flexural stiffness, and fixed-end-anchors with the equivalent flexural stiffness of the ring beams. The 2D axisymmetric model mesh is illustrated in Figure 4.

The opening and the trench effect were not included in this analysis, since the axisymmetric analysis simulates a perfectly circular excavation. Another drawback of this model is the impossibility of evaluate the level of the hoop forces on the wall, which have a main role in circular structures that mobilize the arch effect.

Apart from the mentioned limitations, this model represented an important tool in the validation and calibration of a global 3D model. The results of the 2D model were compared with the 3D

results. In the elements far from the opening, where its influence is less relevant, an identical soil-structure behaviour was observed.

Due to the longitudinal dimension of the structure, a set of 2D plain-strain models were used in the design of the majority of trench's D-wall panels. With the global 3D analysis, it was possible to determine the influence of the opening and, consequently, the range of the 2D analysis. As in the previous case, the trench's 2D plain-strain models were used in the 3D model validation.

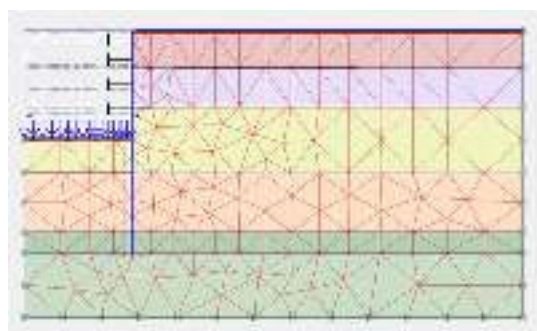


Figure 4 – 2D Axisymmetric pit retaining wall model - mesh.

### 3.3 3D Global retaining wall model

#### 3.3.1 Model definition

Since, both general and local effects in the structural design of the pit and trench were not possible to model with 2D models, it was developed a 3D global finite element model. Figure 5 illustrates the 3D global model mesh.

With a geometry of 200 m x 150 m and 66.5 m depth, the 3D model mesh has 137692 finite elements and 2065380 nodes. The mesh density was optimized using local coarseness features. To reduce the model size and to avoid numerical errors some simplifications were taken into consideration. Therefore, the trench length was reduced and the excavation depth was constant.

The D-Wall panels for both pit and trench retaining walls and bracing slabs were modelled

through plate elements with equivalent flexural stiffness. Considering that the constructive technology of D-walls imposes a non-monolithic connection between successive panels, a hinged connection was modelled between plate elements. The capping beams, the trench concrete struts and the pit ring beams were modelled with beam elements with equivalent stiffness. Tables 3 and 4 summarize the plate and beam element properties.

Table 3 – Plate elements properties.

Structural Element	d m	$\gamma$ kN/m <sup>3</sup>	E MPa	$\nu_{12}$ -	$G_{12}$ MPa
D-Wall	1.0	25	34	0.2	14.17
Bracing Slab	0.85	25	34	0.2	14.17

Table 4 – Beam elements properties.

Structural Element	A m <sup>2</sup>	$\gamma$ kN/m <sup>3</sup>	E MPa	$I_3$ m <sup>4</sup>	$I_2$ m <sup>4</sup>
Struts (200x85mm)	1.7	25	34	0.102	0.567
Ring 4 (100x80mm)	0.8	25	34	0.043	0.067
Rings 1,2,3 and 5 (50x120mm)	0.6	25	34	0.072	0.013
Capping beam (1000x1000mm)	1.0	25	34	0.083	0.083

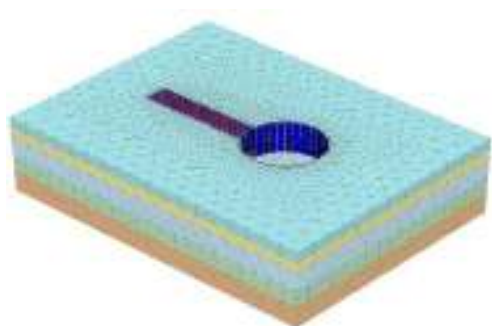


Figure 5 – 3D Global model isometric view – plate elements.

### 3.3.2 Model singularities

The main aspects captured with the 3D model were the local and overall behaviour introduced by the trench excavation on the west side of the pit as well as, in a later stage, the trench/pit opening. The resultant imbalance in the back of the

pit's retaining walls was accounted and balanced with bracing slabs elements. The relative orientation of the trench to the pit was another important issue, since they were not concentric. In the 3D model, the D-wall panels were defined by plate elements with the executed dimensions and were consecutively disposed in order to simulate the real geometrical conditions of the pit retaining wall.

The 3D global model allowed the modelling of the T-panels that connect the trench and pit's structure. For constructive reasons, the web and the flange cage were placed disjointedly so both translational and rotational degrees of freedom were released in those connections. Figure 6 illustrates the model at the zone of the pit/trench

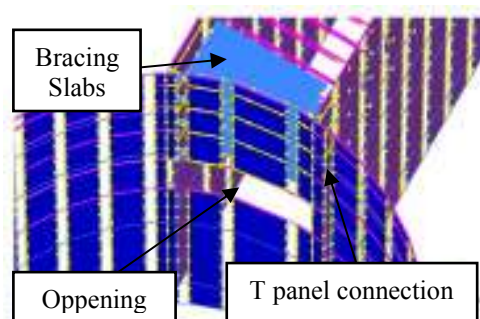


Figure 6 – 3D global model view of the trench/pit connection.

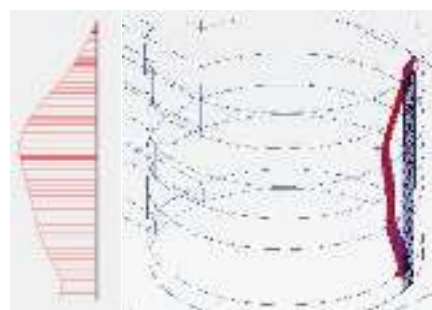


Figure 7 – Pit retaining wall vertical bending moment diagram: 2D axisymmetric model [maximum value: 272 kNm/m] (left) and 3D global model [maximum value: 345 kNm/m] (right).

interception. It can be observed the visible opening, pinned connections between panels and bracing slabs.

## 4 NUMERICAL MODEL RESULTS

### 4.1 2D vs. 3D Pit retaining wall model results.

An analysis comparing the results of the pit's retaining wall stresses and deformations on both the 2D axisymmetric and 3D global models was established, particularly at the opposite side of the pit where its influence was expected to be minor. The main results of the two models, for that section concerning the final excavation stage, are illustrated in the figures 7, 8 and 9.

Regarding the retaining wall bending moments, the results obtained in both models (Fig. 7) show a reasonable approximation concerning diagrams and maximum values reached. Bending moments diagrams are very similar, the peak values for both positive and negative moments are well defined and located at approximately the same depths in both models, showing a value difference of about 20%.

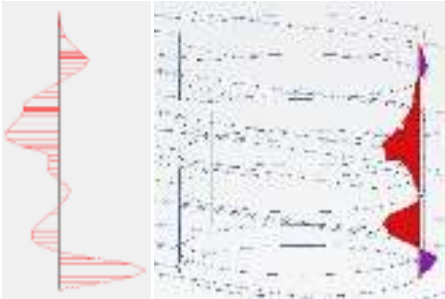


Figure 8 – Pit retaining wall horizontal displacements: 2D axisymmetric model [maximum value: 5 mm] (left) and 3D global model [maximum value: 6 mm] (right).

The horizontal displacement diagram reveals similarities between the two models, and the estimated maximum displacement values are almost identical (Fig. 8).

For a preliminary calculation of the hoop forces, was taken into consideration the effect of earth pressure, ground water pressure and surcharges. In order to simplify the calculation, the stresses

were calculated at the depth where higher hoop forces were expected and for an equivalent soil layer with average geomechanical parameters of all soil layers. With a radius of 22,25 m and an estimated load of 365 kPa, a maximum hoop stress of 8121 kN/m was predicted.

Regarding the normal stress in the tangential direction, referred as hoop forces, although values obtained in 2D model were higher than the 3D model ones, their magnitude is similar and within the order of magnitude predicted by the simplified calculation (Fig. 9). The diagrams agree in both models and, as expected, the hoop forces increase with depth.

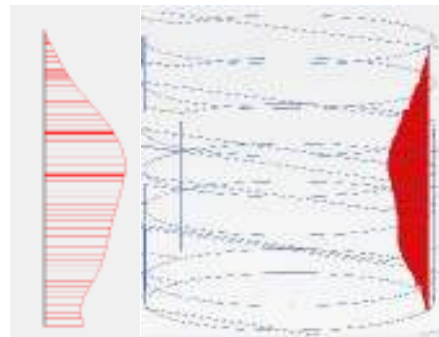


Figure 9 – Pit retaining wall hoop forces diagram: 2D axisymmetric model [maximum value: 7864 kN/m] (left) and 3D global model [maximum value: 6270 kN/m] (right).

### 4.2 2D vs. 3D Trench retaining model results

In order to study the opening effect in the trench wall's behaviour, the 2D model results were compared with the 3D model near the mentioned area. The trench retaining wall stresses and deformations were analysed.

The main results of the two models, for that section and concerning the final excavation stage, are illustrated in the figure 10 to 12. Regarding the 2D model results, it was observed a typical behavior of a retaining wall with multiple passive horizontal supports (Fig. 10).

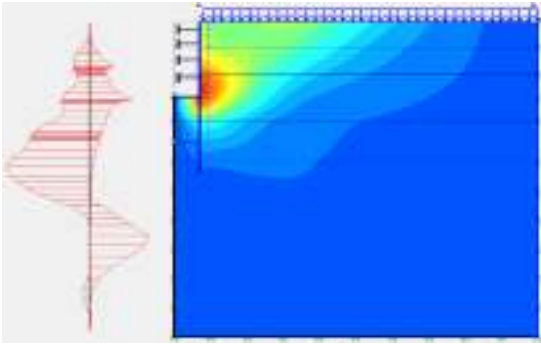


Figure 10 – Trench retaining wall horizontal displacements: 2D plane strain model [maximum value: 41 mm] (left); and bending moment diagram [2381 kNm/m] (right).

Results from the 3D model at a section with the same horizontal supports and excavation depth showed identical deformations and stresses (Figs. 11 and 12), despite the expected difference.

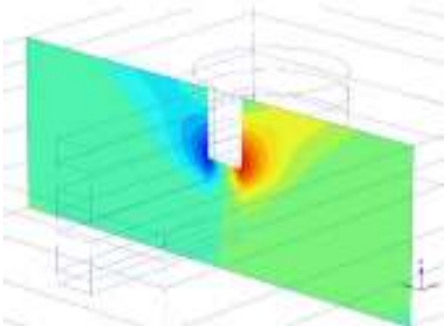


Figure 11 – Trench retaining wall horizontal displacements: 3D global model [maximum value: 35mm].

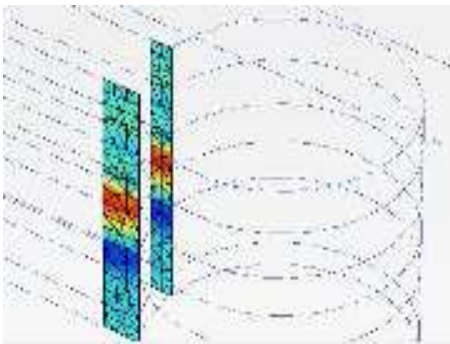


Figure 12 – Trench retaining wall vertical bending moment diagram: 3D global model [maximum value: 2552 kNm/m].

From the previous assessment, it is possible to conclude that the ring beams and bracing slab elements guaranty the overall stability of the pit walls and have a lower impact in trench retaining walls.

### 4.3 3G Global retaining wall model results

Although the 3D global model was developed aiming mainly to validate previous models and simplified calculations, as well as to determine the overall behaviour of the retaining structure, it was also essential in the design of local singularities. The trench/pit connection opening was modelled at the final stage and its effect was observed within the near elements (Fig. 13).

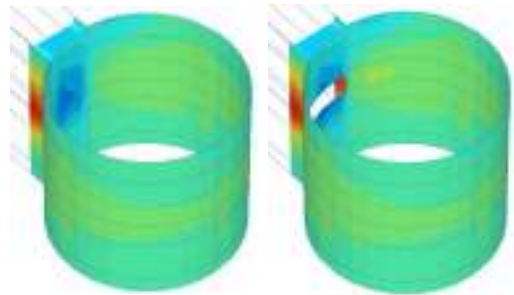


Figure 13 – Retaining wall horizontal displacements: 3D model before (left) and after the opening (right).

The design of the structural elements affected by the opening was validated with simplified calculations, particularly the nearest ring beams, where an increase in its axial forces was expected and had to be assessed (Fig. 14).

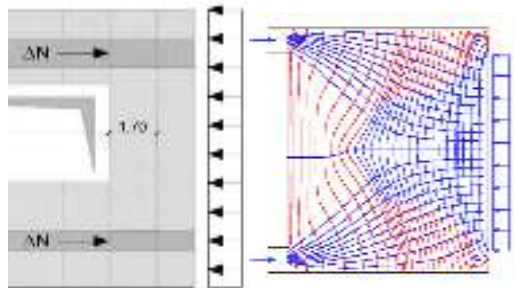


Figure 14 – Simplified model for hoop forces increase due to opening (left). Expected stress path due to opening (right).

The 3D model was also fundamental to obtain the bracing slabs' stresses, since they transfer the outward thrust to the trench retaining walls, resolving the imbalance caused by the trench excavation. The model results confirm the vital role of this elements in the overall stability of the retaining walls. Regarding the walls, due to relevant axial loads in both plan directions, a combined bending axial forces interaction was used at the reinforcement design. Also, the absolute value of torsional bending has been totally added to flexural bending moments on both directions. Absolute values of vertical and horizontal shear has been totally added, considering a total shear acting out of plane. The bracing slab/d-wall interface connection was also design based on the model results. The effect caused by the opening was also observed on the bracing slabs elements, confirming the 3D global model importance since the trench/pit connection opening led to a significant increase of the axial load of the bracing elements, approximately 40% (Fig. 15).

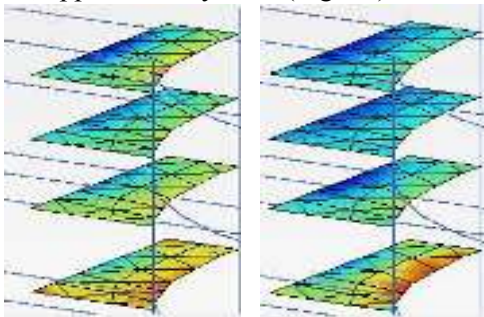


Figure 15 – Bracing slabs axial forces  $N_2$  before [maximum value: 3665 kNm/m] (left) and after the opening [maximum value: 5232 kNm/m] (right).

## 5 FINAL REMARKS

Finite element analyses have been carried with both 2D and 3D models in order to simulate a deep excavation and the necessary dewatering effects for the execution of an unsymmetrical railcar unloading pit and its connection trench. As a first approach, 2D analyses were performed for both pit and trench retaining wall modelling. Those analyses covered separately the pit, exploiting its cylindrical symmetry to establish an

axisymmetric model, and the trench, due to its overall length modelled within a plane strain analysis.

Due to the noncollinearity between the trench alignment and the pit center, the overall stability and behaviour of the structure needed to be assessed. The connection between structures was studied in detail with the aim of understanding their influence on the different elements of the structure. A 3D global model was developed in order to study all the singularities, being validated and calibrated with 2D numerical analyses. The numerical analyses' results, from the 2D and 3D models, showed, in generally, a good agreement, in the areas far from the singularities. Some conclusions on the validity of the 2D models within some parts of the structure were taken and used in the elements design. The 3D model was particularly useful on the analysis of some structural elements, allowing a better understanding of the overall behaviour. Concerning the flow analysis, the 2D models revealed to be adequate.

Since 3D calculation can be time consuming, 2D preliminary analyses can be performed, even when a 3D analyses is geometrically crucial, enabling some initial design calculations and consequentially increasing the level of development of the structure conception for a subsequent implementation in a 3D model. Rather than opposites, 2D and 3D calculations showed to be useful complementary tools in the design of complex tridimensional structures.

The instrumentation readings validate the model results, showing a good agreement between the measurements and simulated values, both in terms of displacements and water inflow.

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# Urgent Stabilization, Reconstruction and Reinforcement Solutions of High Retaining Walls in Lisbon

## Solutions de Stabilisation Impérieuse, Reconstruction et Renforcement de Mur de Soutènement de Grande Hauteur Situé à Lisbonne

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**ABSTRACT:** A partial collapse of a poor reinforced concrete retaining wall, built in 1955 and overcoming a 20m high slope, in February 2017, as well as the subsequent ground sliding, led to severe structural damage of the buildings located at its base. Considering both the precarious structural condition of the buildings, as well as the possible risk of the remaining retaining walls to collapse, there was an urgent need for the execution of a definitive solution that could re-establish the local and global stability of the retaining walls. The implemented solutions aimed to rebuild and to improve the confinement of the retaining walls, while increasing the local and global safety to static, hydrostatic and dynamic loads, as well as the reconstruction of the drainage systems. The works took place between March and August 2017, covering roughly 90m of retaining walls overall extension, simultaneously with the reconstruction of damaged buildings, including their structural reinforcement.

**RÉSUMÉ :** Un collapse partiel d'un mur de soutènement en béton légèrement armé, construit en 1955 et avec une hauteur d'environ 20 m, à Lisbonne, en février 2017, suivant du glissement du sol, a provoqué de graves dommages structurels aux bâtiments situés à sa base. Tenant en compte l'état structurel précaire des bâtiments et le possible risque de collapse de la part du mur de soutènement qui n'y a pas collapse, il a été considéré comme urgent d'exécuter une solution stabilisation, reconstruction et renforcement définitive, avec l'objectif de rétablir la stabilité locale et globale des tous les murs de soutènement. Les solutions mises en œuvre visent à reconstruire et à améliorer le confinement des murs de soutènement, en augmentant la sécurité locale et globale aux charges statiques, hydrostatiques et dynamiques, ainsi que la reconstruction des systèmes de drainage. Les travaux ont eu lieu entre mars et août 2017 et ont couvert environ 90 m de la longueur total des murs de soutènement, parallèlement à la reconstruction des bâtiments adjacents qui ont été affectés, bien aussi comme leur renforcement structurel.

**Keywords:** Retaining wall, Landslide, Collapse, Stabilization, Drainage

## 1 INTRODUCTION

A set of lateral earth retaining walls located in Lisbon, dated from 1955 and with a height of about 20m, partially collapsed on February 27, 2017. An intervention took place comprising both earth retaining walls, covering almost 90m length, and its adjacent buildings, located at their bases, some of them also damaged (see Figure 1).



Figure 1. Aerial photograph of earth retaining walls

The partial collapse occurred in the central zone of this set of cantilever walls, comprising the walls located behind buildings n. ° 108 and n. °106, which experienced severe structural damages result of the walls collapse and consequent landsliding. After the incident the structural stability of the affected buildings was compromised and the overall stability of the remaining earth retaining walls was at risk of imminent collapse, thus the need of intervention was imperious (see Figure 2 and Figure 3).

After the rehousing of buildings residents, a few days after the incident, the intervention works started aiming firstly the restore of the area safety conditions, ensuring the security of operators during the initial works, and thereafter the implementation of reconstruction, reinforcement and stabilization solutions of the earth retaining walls, as well as solutions for the building's reconstruction and structural reinforcement.



Figure 2. View of earth retaining walls after its partial collapse



Figure 3. View from building n.º106 basement after earth retaining walls partial collapse

## 2 EARTH RETAINING WALLS COLLAPSE CAUSE

The information available on the earth retaining walls and the *in-situ* inspection allowed to point out the following possible causes of the incident: a) accumulation of water behind the earth retaining wall and consequent generation of a hydrostatic horizontal pressure, due to extensive irrigation of the garden and eventually water leaks from the pool located above; b) the existence of a clay layer located near the collapse section which may have block the downward percolation of water; and c) ineffectiveness of the drainage system of the walls due to lack of maintenance, simultaneously with the above stated evidences and consequent generation of a hydrostatic horizontal pressure.

The nature of cantilever wall may have eased the incident since they are more than 60 years old and are primarily composed by a low resistance

concrete, slightly reinforced, and showing severe corrosion pathologies.

### 3 URGENT REINFORCED MEASURES

Aiming the safety restore of the area, ensuring the security of operators during the initial works the following emergency interventions were carried out: a) slope re-profiling, essentially on the top consisting of landfill materials; b) removal of potentially unstable blocks located on the surface of the slope; c) slope protection with sprayed concrete reinforced and drained. Those works were highly conditioned and they were mostly carried out with lifting equipment; and d) implementation of temporary stabilization measures on buildings such as shoring before the removal of the landslide materials and debris (see Figure 4).



Figure 4. View of building n.º108 basement shoring

### 4 GEOLOGICAL AND GEOTECHNICAL SCENARIO

At the incident area the geological map of Lisbon at a scale of 1:50,000 shows the surface presence of marine sedimentary facies identified as MQB and dating from Miocene period. These materials are mainly composed by sands with some punctual intercalations of clay layers. The MQB layer overlap a limestone unit, identified as MCV.

Structurally the geologic units show a monocline with NNE-SSW direction and slope to E-SE, being this structure propitious to the formation of steep slopes.

The study of stabilization, reconstruction and reinforcement solutions was made through finite element models of the cantilever walls that were carried out considering a geological and geotechnical zoning. That zoning was established taking into account also the information from some boreholes, executed within the scope of the above building's projects and accounted for the presence of the following materials in depth: a) landfills of variable thickness, consisting of very heterogeneous materials, both sandy and silty-clayey characterized by SPT test values between 4 and 8 blows; b) sand layers from MQT facies mainly composed of fine to medium sand often with limestone fragments. This layer presents superficially a lower density showing in its first 7m depth SPT values between 26 and 38 blows and below that, a rapid increase in its resistance with SPT values greater than 60 blows from the 9m depth.

Regarding the hydrogeological conditions and with the information gathered from studies on near areas, one could conclude that the soils were not very likely to present ground water flows. However, given the permeability levels of surface layers it was essentially to consider the possibility of water infiltration of rain and other sources of water.

Using a finite element model, the geomechanics parameters were calibrated from a back analysis considering a limit stability scenario.

### 5 RETAINING WALL INVESTIGATION

The cantilever retaining walls dated from 1955 had been constructed against an excavation slope overcoming an average height of 20m and a length of about 90m, presenting vertical joints. The walls have a thickness ranging from 20cm at the top to 2m at the bottom and their foundation is a single strip footing of about 6m wide that is

coincident with the building's basement floor. According to the original project the cantilever walls were design for the support of a sandy material with a density of  $16\text{kN/m}^3$ , an internal friction angle of  $40^\circ$  and at the base a bearing capacity of  $400\text{kPa}$ . The project also indicates the existence of only one layer of reinforcement. The design did not consider hydrostatic pressures prescribing the execution of weep holes at each  $2.5\text{m}$ .

A preliminary campaign of in-situ investigation and laboratory testing was performed aiming to check the geometry of the walls fundamentally its thickness at various depths and its resistance. Therefore, several samples were extracted from the wall and taken to laboratory in order to perform UCS tests with measure of Young's modulus (see Figure 5).



Figure 5. Concrete cantilever wall samples

The extraction of samples allowed the confirmation of the geometry indicated in project drawings and later their testing determined an average failure compression stress of  $11.25\text{MPa}$  with a standard deviation of  $3.41\text{MPa}$  and an elastic modulus of  $17.2\text{GPa}$  on average, proving the low resistance of the wall.

## 6 STABILIZATION, RECONSTRUCTION AND REINFORCEMENT SOLUTIONS

The solutions of stabilization, reconstruction and reinforcement were design aiming to restore quickly the global stability of the cantilever walls and to reconstruct the collapsed walls. The urgency of the intervention as well as the site operational restrictions governed the possible solutions. The design was based on the flowing assumptions: a) the need to rebuild the collapsed walls, increasing their global and local stability, for static, hydrostatic and dynamic loads; b) the need to re-confine against the slope the remaining walls, increasing their global and local security for static, hydrostatic and dynamic actions; and c) the need to replace the original drainage elements and assure an adequate drainage system.

Thus, two different designed and later implemented solutions were established. The solution identified as 'Solution A' was executed in the extension of the collapsed walls, that is behind the buildings n. °s 106 and 108, and a solutions identified as 'Solution B' was executed on the remaining walls behind the buildings n.°s 102, 104, 110 and 112.

### 6.1 Solution A

The solution for the partially collapse walls comprised the re-confinement of the remaining walls through the execution of a  $50\text{cm}$  thickness reinforced concrete wall connected to permanent ground anchors with low prestressed loads. Also, the walls strip foundation was reinforced using micropiles both vertical and inclined.

Above the sectioned wall level, a new reinforced concrete wall of  $35\text{cm}$  thickness was executed and it was connected to the existing slope thorough concrete slabs and soil nails (see

Figure 6). Behind the new wall a backfill of expanded clay lightweight aggregates (LWA) was placed properly wrapped in a geotextile of filtration and separation. This solution allowed the design of the new wall for lower lateral pressures as

well as a proper drainage system since the fill material is highly permeable and the affluent water could easily be expelled through the wall drainage ducts. The drainage system was reinforced also with the installation of sub horizontal geodrain pipes.

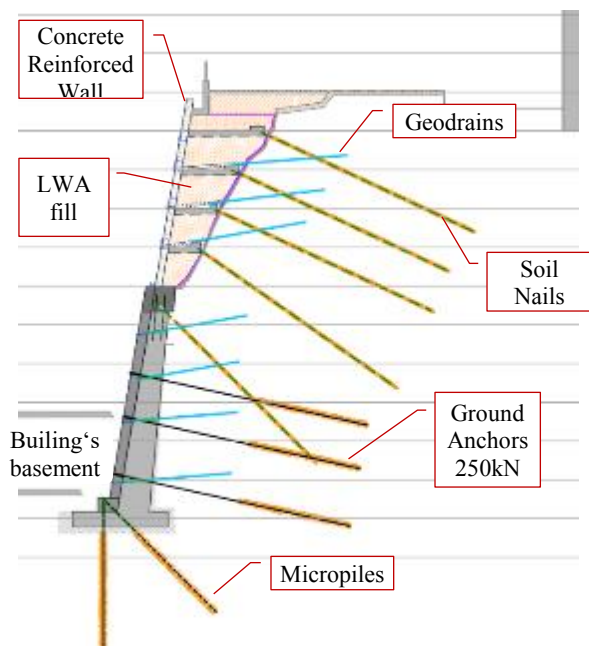


Figure 6. 'Solution A' draft - Cross section

## 6.2 Solution B

On the non-collapsed cantilever walls permanent ground anchors with low prestressed loads were installed in order to re-confine them against the soil slope. For that purpose, a steel grid composed of steel profiles previously coated and painted against corrosion was placed on the wall face vertically supported on a 50cm reinforced concrete wall. At the level of the building's basement, the strip foundation was also reinforced through the installation of micropiles both vertical and inclined (see Figure 7).

Before the implementation of the re-confinement solutions, the existent plaster was removed from the walls face and lined with a high resistance mortar, reinforced with a carbon fibre mesh. Regarding the drainage system, multiple

sub horizontal geodrain pipes were installed in order to prevent the accumulation of water behind the wall.

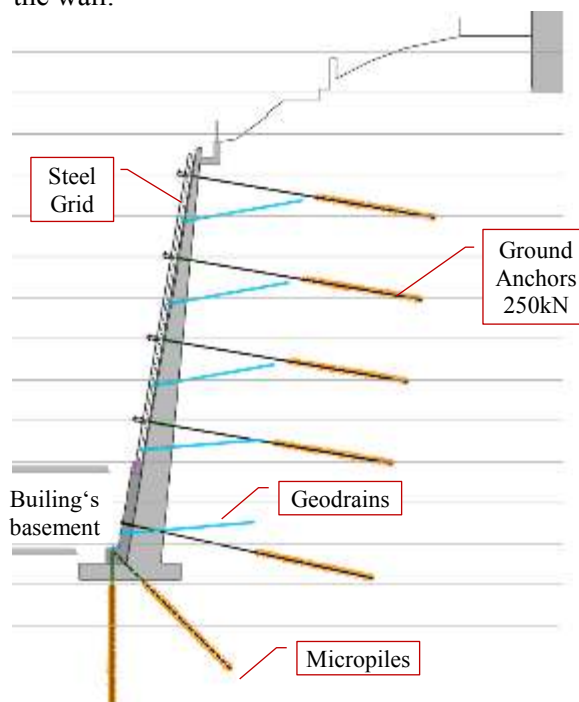


Figure 7. 'Solution B' draft - Cross section

## 7 SAFETY FACTOR INCREASE DUE TO INTERVENTION

The study of the designed solutions was carried out essentially through finite element analyses. Since the problem is a fundamentally two-dimensional, plane strain analyses were performed using 2D PLAXIS software. Firstly, the geomechanics parameters were calibrated from a back analysis considering a limit stability scenario where the failure surface was the closest as possible with the observed *in-situ* (see Figure 8).

Subsequently, the parameters obtained from back analysis were used on the design of the reinforcement and reconstructed elements also through finite element analyses, which allowed also the assessment of the global safety factors increase.

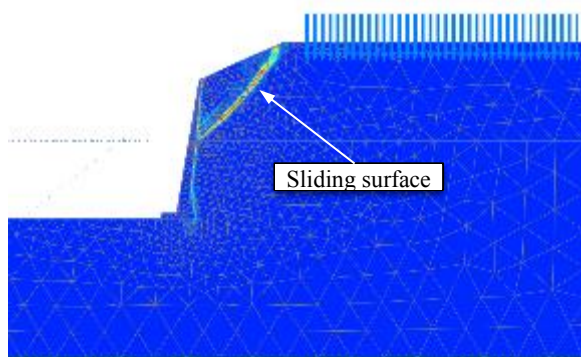


Figure 8. Back analysis - Plaxis 2D output

The design through finite element analyses took into account the following loads: a) static horizontal pressure due to the own weight of the soils; b) static horizontal pressure associated with a load of 200 kPa representative of the adjacent building at the slope crest; c) hydrostatic horizontal pressure in an accidental scenario, corresponding to a situation of ineffectiveness of the drainage systems and consequent installation of a water level beyond the wall up to about 2m depth; and d) pseudo-static pressure, relative to the seismic loads, quantified according to EC8. The following table (Table 1) summarizes the assessed global safety factors accomplished with the solutions implemented.

Table 1. Global safety factors

Solution	Static	Accidental	Pseudo-Static
A	1.7	1.7	1.3
B	2.1	1.9	1.6

Finite element analyses were also used to verify the safety of structural elements in which the national and international guidelines were adopted. In the case of the sealing bulbs of ground anchors, micropiles and soil nails their load capacity was assessed using Bustamante method (Bustamante et al., 1985), and later on site the assumptions previously taken were validated through in situ tests (EN1537: 2013, EN14490: 2010 and EN14199: 2015).

## 8 CONSTRUCTIVE CONDITIONS

The urgency of the intervention as well as the site operational restrictions governed the implemented solutions which were designed in order to ensure its feasibility in a safe and economic way, considering also the circumstances related to accessibility, operational space, constructions procedures and work planning.

Due to lack of space and poor accessibility the tower crane was installed at the closest street, so all equipment and materials needed to be transported to the site through lifting above the buildings. These circumstances lead also to the use of small machinery equipment inside building's basements to remove of soil and debris and, also, the executions of ground anchors and micropiles (see Figure 9).



Figure 9. Micropile equipment inside building basement

The use of climbing scaffolding on the central cantilever walls was crucial allowing two simultaneous work fronts: a) the upward reconstruction of the wall, the execution of soil nails and the filling with LWA; and b) the removal of the soil mass and debris resulting from the collapse and landslide and subsequent reinforcement of the remaining wall (see Figure 10).



Figure 10. Climbing scaffolding and works bellow

## 9 MONITORING PLAN

The use of a monitoring plan during the intervention allowed a continuous analysis of the behaviour of the walls and surrounding structures, assuring mainly the safety of workers.

The monitoring was carried out using topographic targets, installed in the retaining walls and neighbouring buildings, aiming to measure eventual movements during the intervention works (see Figure 11).

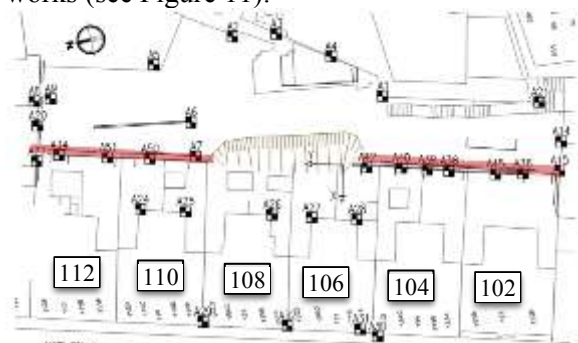


Figure 11. Initial topographic target location on plan

During the reconstruction works the topographic targets were monitored once a week and allowed the site risk management. Since the targets first

installed on the walls needed to be replaced during local intervention the replacement targets needed to consider the previous movement history.

Regarding the post-construction instrumentation and monitoring plan its goal is the measure of the efficiency of the reinforcement and drainage solutions implemented during the service life of the retaining structure. The following devices were installed: topographic targets; ground anchor load cells; inclinometers and piezometers (see Figure 12).

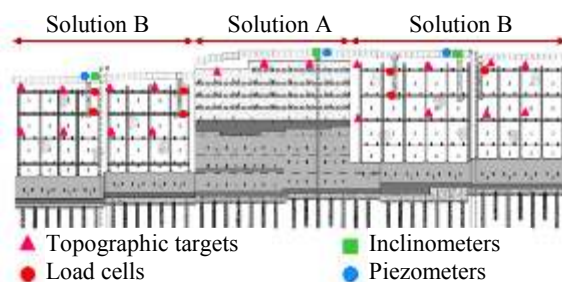


Figure 12. Service life instrumentation and monitoring elevation view

## 10 FINAL REMARKS

This paper aimed to present the solutions implemented as reconstruction, reinforcement and stabilization measures for urgent and definitive restore of the retaining structures safety conditions. Its design was naturally governed by the constructions conditions as well as by the imperative need of intervention, keeping in mind the aesthetics of the solution (see Figure 13).

The work was completed within a period of about 6 months, including also the reconstructions and reinforcement of the damaged buildings. During that time the monitoring plan revealed to be extremely important since it allowed a continuous analysis of the behaviour of the walls and surrounding structures, assuring the safety of the workers, as well as the site risk management.

After intervention completion and during the retaining structure service life the monitoring

plan is also vital since it will allow the confirmation of the expected behaviour of the retaining

structures and reinforcement elements, as well as to assess the need for future interventions.



Figure 13. View after final works

Although there is no evidence on presence of groundwater flows, the inflow of rainwater and water from other sources must be considered. Thus, the inspection and maintenance of the drainage systems must be assured in order to drain the eventual water which can eventually be accumulated behind the cantilever wall.

Finally, the occurred incident proved the importance on the set up of a Lisbon city risk map allowing the risk management of old retaining structures with low or inexistent drainage and structural maintenance.

## 11 ACKNOWLEDGEMENTS

The authors are grateful to the project owner, Lisbon Municipality, for their authorization on the writing and publication of this paper.

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# Retaining Wall Solutions for Underground Extension of Hospital da Luz in Lisbon - Portugal

## Solutions de mur de soutènement pour l'extension souterraine de l'hôpital da Luz à Lisbonne – Portugal

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**ABSTRACT:** The paper addresses the solutions executed for the excavation and retaining walls of the four basements, included on the enlargement of Hospital da Luz, in Lisbon. The main existing conditions will be presented, namely the geological-geotechnical, the urban envelope and the presence of the Lisbon Metro tunnels, very close to the south area of the excavation plot. The working site, located between the existent hospital, Av. Condes de Carnide, Rua Aurélio Quintanilha and Av. Lusíada, comprises an area of approximately 9.800m<sup>2</sup> and a maximum excavation depth of 16m. In order to maximize the economy-safety binomial, without forgetting the issues of functionality and simplicity of construction, it was necessary to develop different solutions, namely anchored bored pile curtains, bored pile curtains supported by reinforced concrete slab strips (area adjacent to the Metro tunnel) and temporary retaining walls "Berlin king post wall". The main criteria for the design of the implemented solutions, as well as the main results of the monitoring system will be presented and compared with the project predictions.

**RÉSUMÉ:** Le document traite des solutions mises en œuvre pour l'excavation et pour les murs de soutènement des quatre sous-sols, compris dans l'agrandissement de Hospital da Luz, à Lisbonne. Les principales conditions existantes seront présentées, à savoir la géologie et la géotechnique, l'enveloppe urbaine et la présence des tunnels du métro de Lisbonne, très proches de la zone sud de la parcelle d'excavation. Le chantier de travail, situé entre l'hôpital existant, la Av. Condes de Carnide, la Rua Aurélio Quintanilha et la Av. Lusíada, couvre une superficie de 9.800 m<sup>2</sup> et une profondeur d'excavation maximale de 16 m. Afin de maximiser le binôme économie-sécurité, sans oublier les problèmes de fonctionnalité et de simplicité de construction, il était nécessaire de développer différentes solutions, à savoir des rideaux à pieu forés soutenus par ancrages ou par les dalles de béton armé (zone adjacente au tunnel du métro) et des murs de soutènement temporaires avec des profilés HEB e barres de bois. Les principaux critères de conception des solutions mises en œuvre, ainsi que les principaux résultats du système de levé topographique, seront présentés et comparés aux prévisions du projet.

**Keywords:** Retaining Walls; Bored Piles; Ground Anchors

## 1 INTRODUCTION

The enlargement of the “Hospital da Luz de Lisboa”, which will be described throughout the present article, is located in a plot, delimited by the building of the current hospital and by several streets and avenues, this plot was originally occupied by a Museum of Lisbon Firemen’s, as shown in Figure 1.

In order to ensure the consistency of the information, analyses and considerations presented throughout this article, recognizing that there are significantly different solutions corresponding to different areas of the work, determined by different constraints, it was decided to divide this article into three parts which correspond to three significantly different retaining solutions. Before the description of the three solutions and their design methods, the global existent conditions were ex-

plained. Aspects such as the geological-geotechnical and hydrogeological scenario, the architectural and functional needs of the building, the existence of important structures and infrastructures located in the vicinity were also detailed.

In order to materialize the excavation of the four underground floors, in a safe condition, in particular for the adjacent structures and infrastructures, three different peripheral retaining wall solutions were executed, and can be summarized as follows: curtain of bored piles, temporarily anchored, along the elevation adjacent to Av. Condes de Carnide; curtain of bored piles, horizontally supported with slab strips made according to the top-down methodology (Pinto et al., 2017 and 2015), along the elevation adjacent to the Av. Lusíada; and "Berlin king post wall" temporarily anchored, along the elevation adjacent to Rua Aurélio Quintanilha (Figure 1).



Figure 1. View of urban envelope and pre-existent structures

## 2 MAIN EXISTING CONDITIONS

### 2.1 Geology and hydrogeology

The geological-geotechnical and hydrogeological characterization of the formations interested in the work described was carried out through the execution of field tests and laboratory tests, namely: eleven boreholes with sampling and SPT

test, installation of three piezometers, granulometric analysis, determination of the water content, determination of the limits of consistency, determination of chemical aggressiveness and determination of resistant parameters by direct shear tests.

Based on the information collected with the tests described above, the following formations were identified, in agreement with the information available in the Lisbon Geological Chart:

Landfills – soils of a diverse nature, but with predominance of silt-clayey soils and sand-silty soils;

Prazeres clays and limestones – soils dating from the Miocene, constituted by the monotonic alternation of sedimentary layers of fine granulometry soils, constituted by more or less silty clays

Formation of Benfica – soils of Oligocene, characterized by purplish brown sediments, composed of sands with extensive granulometry and occasional fine pebbles (occurring at about 30m depth, ie not intersected by the present excavation).

The water table was observed at depths varying between 3m and 13m depth, seeming to follow the old topography of the site, with flow from

NE to SW. However, the hydrogeological productivity of this site was very low considering the small permeability of the clay soils.

## 2.2 Adjacent Urban Infrastructures

As mentioned previously, the intervention area is located in a densely urbanized area, surrounded by important communication routes of the city of Lisbon (Figure 1). Such as streets and tunnels of the Lisbon Metro. Figure 2 shows in detail the different confrontations of the excavation site, where it is evident the Av. Condes de Carnide, (North), the Rua Aurélio Quintanilha (East), the Av. Lusíada and the tunnels of the ML (South) and the structure of the current Hospital.(West).

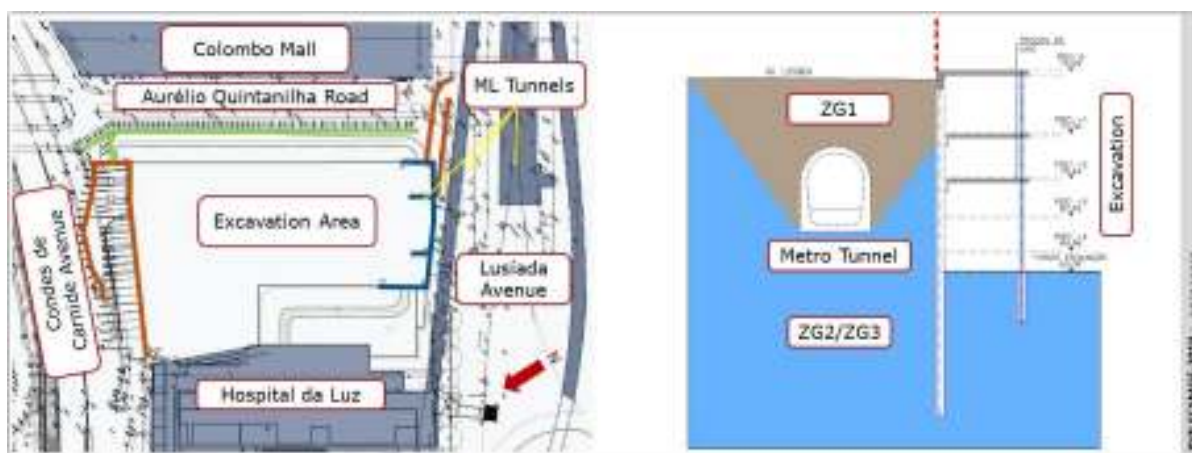


Figure 2. Aerial view of urban envelope

## 3 ADOPTED SOLUTIONS

### 3.1 North elevation – Adjacent to Av Condes de Carnide

Based on the existing conditions, in particular topographic, geological-geotechnical and occupation of the neighbourhood, a curtain of bored piles, reinforced concrete, Ø600mm apart of each 1.20m, with maximum total length of about 22m, in order to ensure an embedded

length, below the final excavation level, of 5m. Considering the geological conditions of the site and the length of the piles, they were executed with Kelly telescopic bar and only with temporary casing at the first meters of the drilling shaft, (landfill materials).

The exposed ground between piles was protected with a 10 cm thick sprayed concrete layer, applied in two phases and reinforced with metallic fibres and drained with sub-horizontal geodrains. To ensure the horizontal support during the final stage, whenever the underground slabs

did not connect to the bored piles curtain, an additional reinforced concrete wall was connected to the piles.

The implemented solution allowed an excavation depth of about 15m, necessary for the execution of the 4 underground floors. Throughout the present elevation, since there were no relevant restrictions and in order to guarantee a high excavation rate, the retaining wall was horizontally supported by means of several levels of temporary anchors. The supported earth pressures were determined by the terrain geotechnical parameters and the loads applied on the surface. In order to ensure a better distribution of the forces in the

curtain and to avoid an excessive concentration of loads, around the ground anchors, several distribution beams were executed in front of the bored piles.

The ground anchors had 5 x 0.6 "strands to accommodate a maximum tension load of 600kN and were spaced of 3.6m. In order to avoid the possibility of intersections of these elements with existing installations and structures, as well as to allow the execution of the grouted length (minimum length of 7m) in terrain competent and geologically stable in relation to the excavation geometry, the anchors were drilled with variable inclinations and variable lengths.



Figure 3. Section Cut and Construction Works

### 3.2 East elevation – Adjacent to Rua Aurélio Quintanilha

Considering the existing and previously described constraints, a temporary Berlin king post wall with a height varying between 5m and 10m, and temporarily anchored at 2, 3 or 4 levels, was executed along the edge adjacent to Rua Aurélio Quintanilha (Figure 4), which allowed the complete demolition of the existing building and the maintenance of traffic at Rua Aurélio Quintanilha.

The temporary Berlin king post wall technology consists of the pre-installation of vertical

steel profiles, grouted in the ground, followed by the placement of wooden bars between them during the excavation and supported by ground anchors distributed along horizontal steel beams.

Considering the geotechnical characteristics of the existent soils, namely their cohesive nature, the vertical HEB 160 profiles, which reached maximum lengths of 22m and were planned to be installed using a small drilling diameter (250mm), were installed using CFA technology assembled on a bored pile rig, which allowed higher construction rates.

In the present case, HEB profiles were installed with distances of 1.2m and 1.5m, and the wooden bars placed between them had a thick-

ness of 10cm. The temporary anchors, with variable spacings between 2.4m and 3.6m, were applied on distribution beams made of metallic laminated steel profiles, materialized by 2 x UPN 300 profiles, in order to accommodate the pressures determined by the terrain and the loads.

From bottom of the retaining wall, to allow the complete demolition of the existing building, was carried out an additional 12m excavation. The excavation slopes were an inclination of 2:3 (h:v), which allowed to reach the base of the excavation for the implantation of the foundations of the new building (Figure 4).

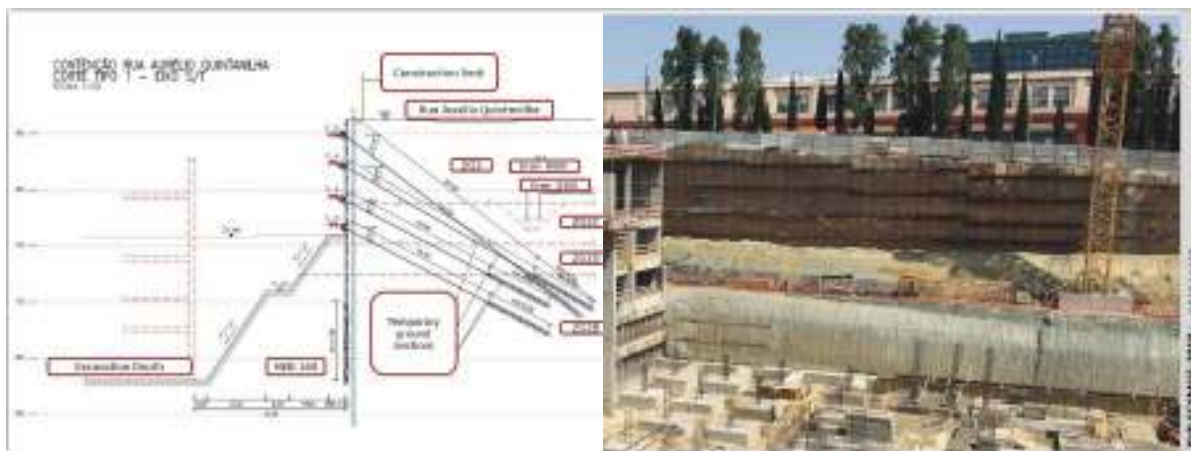


Figure 4. Section Cut and Construction Works

### 3.3 South elevation – Adjacent to Av. Lusíada and Metropolitan Tunnels

In the southern edge of the excavation area, a retaining wall solution similar to the one described for the North edge was implemented. However, considering the neighbourhood conditions present in this zone, particularly the ML Tunnels, the use of ground anchors to horizontal support the wall, was not geometrical viable. Thus, in this elevation, a retaining system consisting of partial slabs executed by top-down method was chosen, which consists of a set of horizontal beams (sections of slab of future underground floors). Considering high length of this elevation, two intermediate buttresses were executed to reduce the span of the slabs. With the exception of the two central buttresses and the steel profiles for temporary support of the slabs, all other structural elements were integrated into

the final structure of the underground floors. This solution was inspired by previous works, where it proved to be very appropriate (Pinto et al., 2017 and 2015).

Due to the existing conditions, namely the subway tunnel, the slabs were executed at floors 0, -1 and -2, and the excavation between floor 3 and the foundations' depth was carried out only after the complete construction of the above underground floors, in order to better control the maximum displacements of ML tunnels (Figure 5).

The partial slabs had a variable width between 6.0m and 9.0m, a current thickness of 30cm, 2 lateral spans of 15.0m and a central span of 25.0m. The partial slabs were supported during the excavation phase in the bored piles curtain and in HEB240 profiles, spaced of 7.50m. These profiles had 2.0m of embedded length inside piles of Ø600, with a total length of 4m below the bottom of the excavation.



Figure 5. Section Cut and Construction Works

## 4 DESIGN METHODS

### 4.1 North elevation – Adjacent to Av Condes de Carnide

The design of the bored piles curtain was carried out using a nonlinear finite element model, in a flat state of deformation, using the software Plaxis 2D. The structural elements were modelled with linear elastic elements of the frame type, the ground anchors and their grout bodies with anchor type elements and geogrid type elements, respectively. The soil layers were modelled with non-linear finite elements of 15 nodes, using a constitutive law of Hardening Soil type and obeying the Mohr Coulomb rupture criteria.

### 4.2 East elevation – Adjacent to Rua Aurélio Quintanilha

For the design of the temporary Berlin-type king post wall executed along Rua Aurélio

Quintanilha, the same software was used, as well as, the same type of simplified models. However, for this solution it was necessary to ensure the analyses of the safety against overall stability of the slopes between the base of the temporary retaining wall and the bottom of the excavation.

### 4.3 South elevation – Adjacent to Av. Lusíada and Metropolitan tunnels

Considering the complexity of the solution of the present elevation, namely the interference with ML tunnels and the difficulty of bi-dimensionally modelling a predominantly three-dimensional effect, a Plaxis 3D model was performed (Figure 6). Despite the 3D model did not ensure adequate results for the design of all the structural elements, due to high computational requirements (difficult to be compatible with the project development period), it allowed to obtain important information for the calibration of the 2D models and, in this way, to guarantee good estimations of stresses and realistic deformations, which were verified through an adequate system of monitoring.

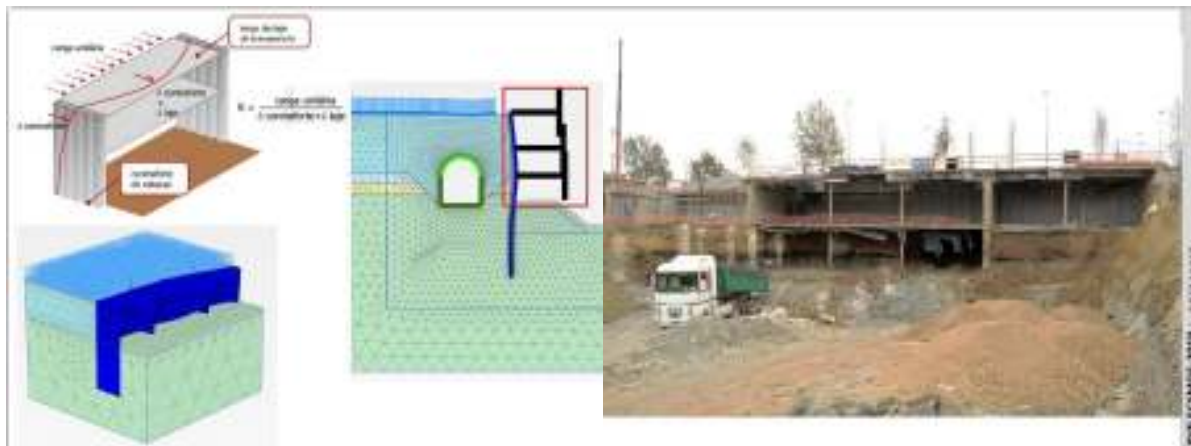


Figure 6. Models outputs and Construction Works

## 5 MONITORING

Regarding the size and complexity of the work described, it was necessary to implement a demanding monitoring system for the retaining walls and ML tunnels, which included the installation of topographic targets, inclinometers, piezometers, load cells and levelling marks (Figure 7). Based on the weekly readings of all the devices listed above, including the targets and

marks installed on the rails and ML tunnels, it was possible to validate the design assumptions and, when and where necessary, to adapt the constructive phases to minimize excessive deformations.

To understand the significant load increase in the anchors shown in Figure 7, it should be noted that the same was already foreseen at the design stage and results from the increase in the level of the terrain located at the top of the piles, several meter above the capping beam.



Figure 7. Monitoring results and Construction Works

## 6 FINAL REMARKS

Throughout the present article it was tried to demonstrate the importance of adopting different and adjusted solutions to the constraints of each zone of a complex worksite, in order to meet the client's expectations. The choice of proposed solutions was based on the main constraints in the worksite described, in particular the geological and geotechnical conditions, as well as the neighbourhood conditions. The main geotechnical solutions used consist of bored piles curtains, supported by ground anchors and partial slab and temporary Berlin king post wall, in order to maximize the suitability of the solutions to the described constraints.

It should be noted that, at the present moment, the entire excavation has already been carried out and all the underground floors have already been executed, with real measured deformations generally lower than those estimated at the design stage. The exception to this situation occurred in only on four monitoring sections in one of the three ML tunnels, in which the alert criteria were slightly exceeded and, consequently, some construction procedures and stages were adjusted in a timely manner. Namely, the excavation of the last berm was delayed to the end of the works, after all the underground slabs were executed.

## 7 ACKNOWLEDGEMENTS

The authors acknowledge to the Owner of the Work presented, the authorization for the writing and presentation of this article, as well as the excellent collaboration with all the actors of the work, inspection, other designers and contractors.

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# Infinity Tower, high rise building in Lisbon: innovative solutions for a deep and complex excavation

## Infinity Tower, bâtiment de grand hauteur à Lisbonne: solutions innovateurs pour une excavation profonde et complexe

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**ABSTRACT:** The Infinity Tower is a high-rise building, with 26 upper stories, being built in Lisbon. With 4 basement floors with about 4.600m<sup>2</sup> plan area, its construction comprises an approximately 18m maximum depth excavation, intersecting mainly heterogeneous fills and the Lisbon Volcanic Complex. The site is surrounded by vital infrastructures such as a 70 years old wastewater tunnel, with 8m width and a plain concrete structure (Alcântara wastewater tunnel), a viaduct abutment and a railway line. To reach the foundation level safely, minimizing the surrounding ground disturbance, a retaining wall solution with reinforced concrete bored piles wall was designed. Due to the close presence of the described infrastructures, the basement slabs were used as strips to brace the wall, cast against the ground during the excavation works, and bridging spans of about 50m. This paper describes the main design topics of this project, including the BIM approach.

**RÉSUMÉ:** L'Infinity Tower c'est un bâtiment à grand hauteur, avec 26 étages, qui est en train de commencer à Lisbonne. Il aura 4 sous-sols avec une area de 4.600m<sup>2</sup>. Cette construction demande une excavation d'environ 18m of maximum profondeur, que va intersecter de remblais hétérogènes et le complexe volcanique de Lisbonne. L'excavation sera entourné para des importantes infrastructures, comme le tunnel de drainage des eaux résiduelles avec une âge de 70 ans (Alcântara tunnel), un pont automobile et une ligne de chemin de fer. Pour prendre le niveau final d'excavation, minimisant la perturbation du sol, une paroi à pieux forés a été projeté. Tenant en compte la proximité des infrastructures invoques, des tronçons des dalles des sous-sols ont été utilisés pour contenir la paroi à pieux, bétonnes contre le sol et avec une portée d'environ 50m. L'article présent les principaux critères de dimensionnement, inclus les avantages de l'approche avec la technologie BIM.

**Keywords:** deep excavation; bored piles; BIM.

## 1 INTRODUCTION

The Infinity Tower will be an iconic building in the Lisbon skyline with an impressive modern architecture, being the tallest residential building in Lisbon with 26 upper stories and 4 basements, resting over an area of 4600m<sup>2</sup> and demanding a maximum excavation depth of 17.60m (Figure 1).

The main issues are related to the demanding conditions, mainly: the geological and geotechnical scenario, the topography as well as some vital neighbor infrastructures, such as the 70 years old wastewater sewage tunnel, with 8m width and plain concrete structure, (Alcântara WW tunnel), a roadway viaduct and a railway line (Figure 3).

This case study also approaches the use of the Building Information Modeling (BIM) for a complex geotechnical project. BIM helps to promote an early stage design collaboration with different project parties due to its capability to work as an information repository where the data is centralized in the BIM model (Azhar, 2011). This collaboration is important to tackle the uncertainty and reduce the reduplication of work, leading to an improved efficiency in the project delivery (Hardin, 2015). The centralized parametric elements in which the 3D/BIM models are based make possible that design changes are automatically assimilated in all drawings and views. The time spent in documentation can be shifted to the development and improvement of the design solutions.

In this project, the architecture model was used as a base for the geotechnical design and 3D modeling of the conceived solutions, allowing to achieve an accurate geometric coordination and efficiency gains in the structural design and project documentation.



Figure 1. Infinity Tower virtual image

## 2 MAIN RESTRAINTS

### 2.1 Geological and geotechnical conditions

The characterization of the underground conditions was made through 9 boreholes with SPT tests and sampling collected for visual classification and laboratory tests. The excavation area is

located over the Lisbon Volcanic Complex materials, covered by a very heterogeneous landfill deposit layer, with a maximum thickness of about 12m. This area was divided into 4 geotechnical zones: ZG1, regarding the heterogeneous urban landfill layer; ZG2 for pyroclastic tufts and low-quality basalts; ZG3 and ZG4 for medium to high-quality basalts (Table 1 and Figure 2).

Table 1. Geotechnical zones and parameters

Geotechnical Zone	Description	$\gamma$ (kN/m <sup>3</sup> )	$\phi'$ (°)	$c$ (kPa)	$E_s$ (MPa)
ZG1	Landfill (SPT<20)	18	30	0	15
ZG2	Pyroclastic tufts and low-quality basaltic W4 to W3-4; F5 to F4-5 with recovery ranging from 60% a 100% a $RQD \geq 0%$	22	33	50	85
ZG3	Basalts W3 to W4-2; F4 to F4-5, with 90% recovery ranging and $RQD \geq 20\%$ interbedded with basaltic W3-2, F4-5 with 100% recovery and $47\% \leq RQD \leq 74\%$	23	37	80	120
ZG4	Basaltic W3-2 to F4-3, with 100% recovery and $55\% \leq RQD \leq 79\%$	23	45	100	150

It should be pointed out that the groundwater table was not observed above the final excavation level.

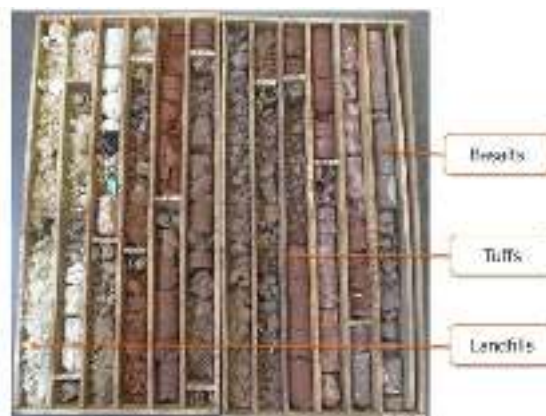


Figure 2. Boreholes cores samples

### 2.2 Topographic and urban restraints

The site is located over a 10m high hill, facing the railway line, consequently leading to an excava-

tion depth ranging from 17.60m to 6.25m in opposed alignments: West (railway line) and East (WW tunnel and roads), as shown in Figures 1 and 3.



Figure 3. Topographic and urban restraints

### 2.3 Neighboring conditions

As already stated, the site is located on an urban area. In the West, it is limited by the Lisbon sub-urban railway line, at South side by a roadway viaduct, and at the North-East and East sides are limited by road traffic and pedestrian streets, over the old wastewater tunnel (Figure 4).



Figure 4. Site location and neighbor conditions

### 3 ADOPTED SOLUTIONS

The adopted earth retaining solutions were designed considering the existing restraints with the purpose to control the ground deformations and execute the excavation with the minimum interference with the surrounding infrastructures and services, taking in account the safety, constructability, schedule and economic factors.

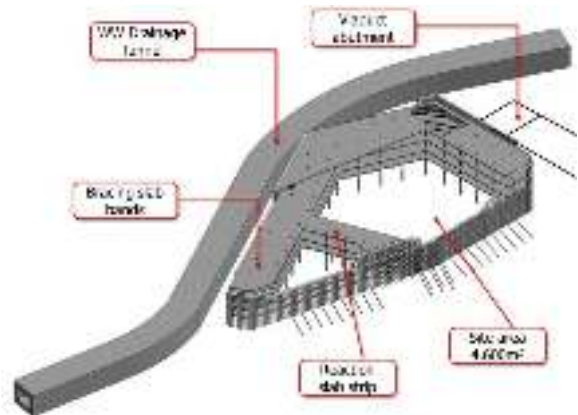


Figure 5. Virtual 3D perspective of the solution

The designed solution was a bored pile wall (BPW) with 600mm diameter piles spaced at centers from 0.80m to 1.20m, according to the geological and geotechnical conditions. The total pile's length ranges from about 22m to 10m, all with a minimum embedment length of 4m at the Lisbon Volcanic Complex materials.

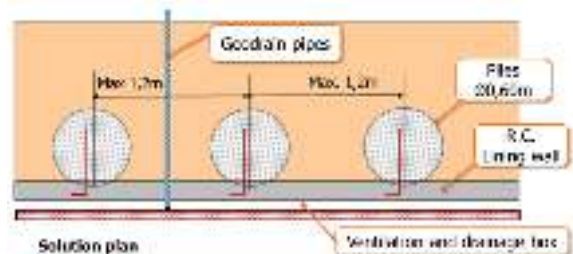


Figure 6. Plan of the adopted solution

The BPW was braced, at each floor level, by reinforced concrete slab bands and temporary

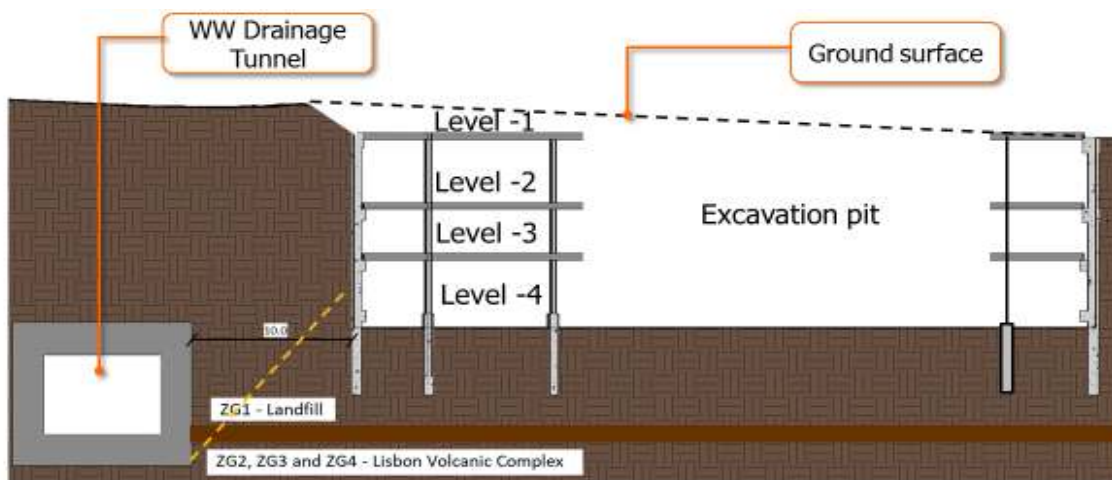


Figure 7. Cross section of the adopted solution at the North side

ground anchors (West side). The soil located between the pile faces will be lined by a shotcrete layer of 80mm minimum thickness. At each excavation phase, corresponding to about 3m depth, geodrains, with a minimum of 50mm diameter, will be installed with at least 3.60m plan distance to ensure the wall drainage (Figures 5, 6 and 7).

In the West front, the wall will be braced by one level of temporary ground anchors. at level -2, spaced in plan 3.60m. As mentioned, the remaining excavation sides will be braced by slab bands, with 12m width and 35cm minimum thickness, compatible with both the architectural and the structural solutions. The slab bands will ensure a stiff bracing to the solution and will be incorporated on the basements final structure (Pinto, 2017).

These slab bands will temporarily be supported by vertical steel profiles HEB260 embedded inside 600mm piles, with an embedment of about 4m below the excavation final level. The slab bands above level -2 will be braced by reaction slab strips, which will react against the BPW at the West side, where the volcanic complex is closest to the ground surface (Figure 8).

It should be pointed up that the very heterogeneous geological and geotechnical scenario, ranging from heterogeneous fills to basalts

(W3 and F3), associated to the demanding neighbor conditions, mainly the wastewater tunnel and the roadway viaduct, lead to the adoption of a BPW solution, bracing at both the tunnel and the viaduct sides by slab bands, as main consequence eliminating the need for the use of temporary ground anchors.



Figure 8. Plan view of the level -2.

## 4 DESIGN

The architecture project was available in a 3D BIM model and it was used as a base for the modeling and design of the retaining solution, allowing to explore some of the advantages of the BIM

methodology (Figure 11). The existing topography was modeled in the BIM software Revit 2018 and then, the architecture model was linked to the file, allowing a geographic and geometric coordination (Figure 9).



Figure 9. Architecture and geotechnical coordinated 3D models

The designed solution was adjusted with the support of a finite element software (PLAXIS2D). The solution was analyzed in terms of stresses and displacements considering the geotechnical zones and its corresponding parameters (Table 1).



Figure 10. Perspective of the 3D BIM model

The geometry from the BIM model was exported to the structural analysis software (SAP2000) using the standard IFC file type and the stresses obtained from the PLAXIS2D were introduced in the model. The retaining solution was then iteratively optimized according to the stresses and displacements obtained from the analyses (Figures 12 and 13).

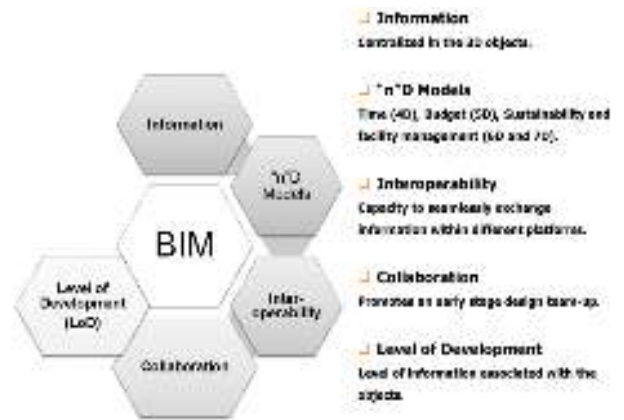


Figure 11. BIM concept

When the design and corresponding 3D model were completed, the earthwork quantities were readily available, the same for the structural elements. The length of each steel profile and volumes of concrete used for the bored piles, slab strips and beams could be accurately estimated. This is because the BIM methodology is based on parametric modeling which means that each design element can host relevant data that can be used through all the project lifecycle.



Figure 12. SAP2000 3D model for the slab bands

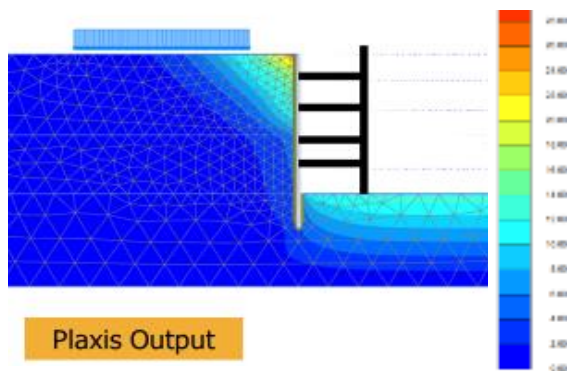


Figure 13. Plaxis 2D output: horizontal displacements for the South side cross section ( $\times 10^{-3}$  m)

## 5 MONITORING AND SURVEY PLAN

As usual in complex geotechnical works, the adopted solution should be confirmed, and the design geotechnical parameters will be calibrated considering the observation of the solution behavior on site. For this purpose, during the excavation works, a complete monitoring and survey plan will be implanted on both the BPW and neighbor structures and infrastructures, using mainly the following devices: inclinometers, topographic prisms and load cells.

Alert and alarm criteria will be accessed considering the predicted displacements at the design stage. If the criteria will be overpassed, reinforcement measures will be adopted.

## 6 FINAL REMARKS

In spite of the excavation works are predicted to start by the summer of 2019, the authors believe that the adopted solutions will have a good performance, from both technical and economic perspectives, and mainly the deformations control through the use of the bracing slabs bands concept, which is being used in Lisbon from about 15 years ago.

The presented case confirmed also that the use of the BIM methodology (Fig. 14) allowed an accurate coordination with the architecture project

and promoted efficiency in terms of project documentation, especially when changes in the design were needed. The interoperability among software allowed that the geometry from the 3D BIM model could be exported, avoiding the re-modeling and possible geometry inaccuracies. The 3D visualization of the project and the restrains helped to find and optimize the overall engineering solutions.

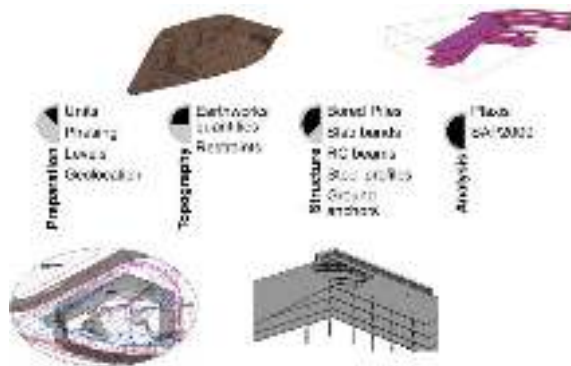


Figure 14. BIM methodology

## 7 ACKNOWLEDGEMENTS

The authors are grateful to the site owner, for his permission to the presentation of this paper.

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# Landslide risk mitigation of “São Pedro de Alcântara Viewpoint Slope“ in Lisbon Historical Center

## Mitigation du risk de glissement de la coline du mirador de São Pedro de Alcântara, dans le centre historique de Lisbonne

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**ABSTRACT:** São Pedro de Alcântara is a viewpoint at the crest of one Lisbon’s main hills. The correspondent slope has been under observation and monitoring aiming to follow up its behavior and to foresee possible instability phenomena affecting the slope behavior under both static and dynamic loads. Considering the last years monitoring campaigns, and specifically the records of inclinometers installed in 2010, it was observed the presence of horizontal cumulative movements at deep soil layers with a progressive tendency, together with the formation of cracks at the retaining walls. The slope and retaining structures overall behavior was investigated, leading to an intervention that included both structural and geotechnical reinforcement for global and local stabilization with the purpose to mitigate the landslide risk. The main implemented solutions included: confined buttress of concrete and reinforced bored piles, covered by a light weight aggregates fill, as well as the refurbishment of the drainage systems.

**RÉSUMÉ:** São Pedro de Alcântara c’est un mirador situe au sommet de une des plus importants collines de Lisbonne. Le talus de cette colline est sous observation et motorisation avec l’objective de suivre le comportement et d’anticiper de possibles problèmes de déstabilisation, pour des actions statiques e dynamiques. Tenant en compte l’évolution historique des déformations horizontaux profondes, mesures avec les inclinomètres installés depuis 2010, bien aussi comme des fissures observes dans les structures de supporte, la stabilité globale du talus a été analysé. La rétro analyse a confirmé qu’il était important le renforcement do talus avec l’objective d’incrémenter la stabilité local et global du talus pour les actions statiques e sismiques. La solution de stabilisation adopte a compris l’exécution des caisses à pieux moules en béton et en béton armé. Sur les pieux un remblai avec de agrégés légères a été exécuté, bien aussi comme le renforcement des systèmes de drainage.

**Keywords:** Landslide; risk mitigation, stabilization, bored piles.

## 1 INTRODUCTION

The acquisition of the São Pedro de Alcântara (SPA) garden by the Lisbon City Council (1732) was done in order to allow the construction of a water supply aqueduct, that would supply the

eastern part of the city, spanning the Liberdade Avenue valley. In the second half of the 18th century a big platform was built with purpose to accommodate the aqueduct abutment, as well as to allow the installation of a fountain and a water

tank. The aqueduct works have never been completed, except a masonry gravity wall, in order to support the embankment and the fountain. Since then, the whole area has suffered several changes until it has become a public garden, from 1835 until now. The site aerial view is shown in figure 1. The zone where the SPA viewpoint is located has several platforms, which are supported by masonry gravity retaining walls. The two main earth retaining structures divide the upper and lower platforms of the viewpoint, and the lower platform and the Taipas street. The levels differences between the upper and lower platforms are accommodated a wall with about 6m high and between the lower platform and the Taipa street by a wall with a maximum height of about 11m with internal buttresses (Figure 1).



Figure 1. Site aerial view.

Between Taipas Street and Fala-Só Bystreet there is a third wall that, with a maximum height of about 15m. The overall slope is identified by the presence of terraces and retaining walls, with an overall height of about 40m, from Liberdade Avenue to SPA viewpoint. At the slope are located several water lines, that ran in the direction of the Liberdade Avenue. Nowadays, due to the intense urbanization, the water percolation through the slope leads to the erosion of the shallow and less compact ground, as confirmed by several repair works.

## 2 MAIN INSTABILITY INDICATORS

The behavior of the earth retaining structures at the SPA viewpoint, as well as the ones between São Pedro de Alcântara, Taipas / Glória Street and Fala-Só Bystreet has been monitoring and checked by several geological and geotechnical site investigations, which allowed to confirm some important ground horizontal movements. The cumulative result of those movements led to several and important cracking, mainly at the wall supporting the lower platform (see Figure 2).

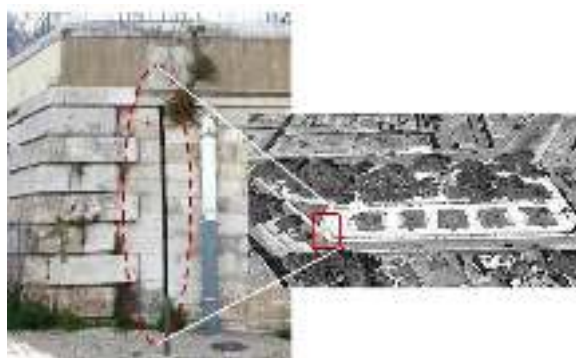


Figure 2. View of cracks at the existent wall's corners.

From the global appreciation of the displacements, it was possible to conclude that, during the last 5 years, horizontal movements were not negligible and showing some evolution with time. The largest displacements occurred at depths approximately coincident with the level of Taipas Street. However, displacements are recorded at lower depths, about 30m (Figure 3). These displacements indicated that a global instability phenomenon could be under development. Following this interpretation, stabilization measures would be needed.

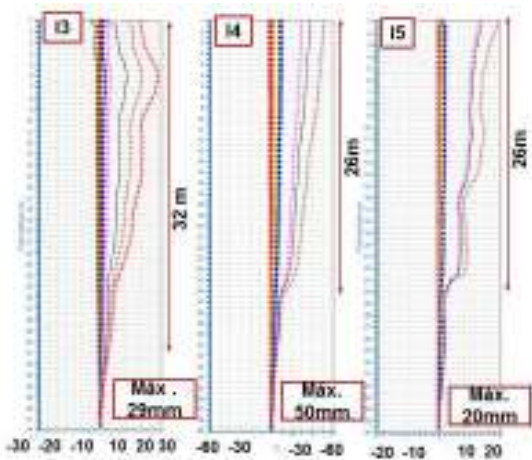
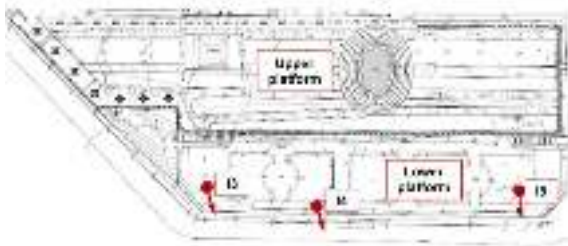


Figure 3. Record of accumulated movements in inclinometers installed in 2011 on the lower platform.

### 3 MAIN RESTRAINTS

As main restraints that would affect the stabilization solutions adopted, can be pointed out:

- Geological and hydrogeological conditions, with three geotechnical zones: ZG3 - heterogeneous landfills, composed mainly by clayey sands; ZG2 – soft Miocene layers, consisting essentially of silty and sandy clays; ZG1 - stiff Miocene layers, with mainly hard silty clays (Table 1).
- Very sensitive neighborhood conditions: the solution to be implemented should minimize the occurrence of any deformations in neighboring structures and infrastructures.
- Conditions of accessibility and car traffic in the surroundings: advising the use of solutions that minimize the volume of earth works, as well as noise and vibrations.

- Conditions associated with the landscaping: the garden at the lower platform should be dismantled and reinstalled.

The main objectives of the stability solution would be to improve the overall stability of the slope as well as the respective drainage conditions.

Table 1. Geotechnical zones and parameters

Geotechnical Zone	Description	$\gamma$ (kN/m <sup>3</sup> )	$\phi'$ (°)	$c'$ (kPa)	$E_s$ (MPa)
ZG1	Landfill: sands and clays	18	25	0-10	3-15
ZG2-1	Miocene: silty clay, silty sands	20	22	10-60	10-50
ZG2-2	Miocene: marl and weathered sandstone	21	24	10-60	10-50
ZG3	Miocene: marl and sandstone	22	28	60-100	60-100

### 4 ADOPTED SOLUTIONS

The adopted stabilization solution consisted on the execution, at the SPA view point inferior platform level, of bored piles walls, Ø1000mm, spaced 0.8m, with reinforced concrete (long) or plain concrete (short), forming several rectangular buttresses, with 5m wide. The piles were capped by reinforce concrete capping beams and cover by a lightweight aggregates (LWA) fill, allowing to increase the global slope stability conditions (Figures 4, 5 and 6).

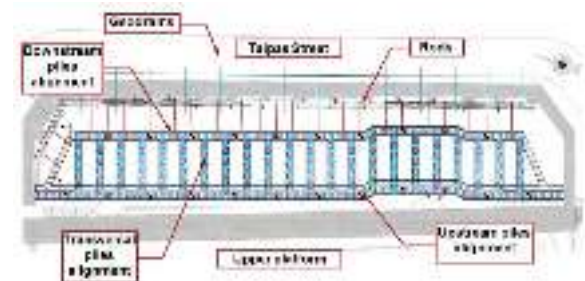


Figure 4. Plan of the adopted solutions.

The longer reinforced concrete piles intersect the previously identified sliding surfaces. The shorter plain concrete piles allow to increase the shallow and less compact ground confinement.

The piles accommodate a significant part of the earth pressures, relieving the existing retaining walls, as well as contributing to increase their local stability

The length of the piles was defined with the aim of ensuring not only that they will rest on a competent layer (ZG1), but also in such a way that they could intersect the previously identified sliding surfaces, based on the inclinometer's readings. For a better control of the deformations of the lower wall between the lower platform and Taipas Street, the outside piles were connected by steel tie rods to a reinforced concrete distribution beam, nailed to the wall the buttresses through sub-vertical permanent nails, Gewi Ø32mm. For a better deformation control the rods, spaced by 2m in plan, were pre-stressed.

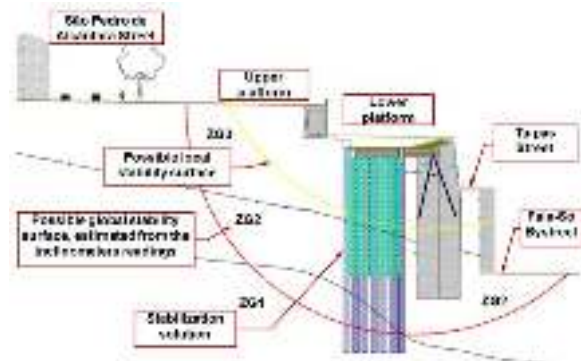


Figure 5. Cross-section of the adopted solutions.

The piles were covered by the LWA fill, expanded clay 10-20mm size, properly confined by geotextile blankets. The use of this type of material allowed either the weight reduction at the unstable zone, increasing the safety factor of the overall slope stability, decrease of the earth pressures over the existing walls, resulting in greater control on the deformations of this structure. The use of LWA fills also allowed to increase the overall shallow drainage conditions.

In order to allow to increase the deep drainage conditions, sub-horizontal geodrains, Ø 50mm PVC pierced pipes wrapped in geotextile, were also installed in the lower wall, facing the Taipas Street.

Complementary to the slope stabilization works, additional rehabilitation works were also carried out at the lower masonry wall. The most important were: cracks sealing using cement mortars as well as the rehabilitation of the wall face wall, including a new coating after the removal of the existing vegetation.

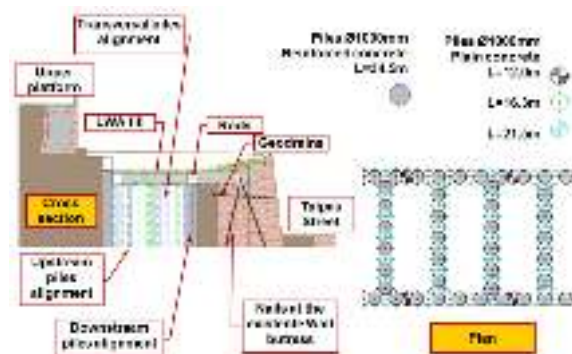


Figure 6. Details of the adopted solutions.

## 5 DESIGN

The initial design phase comprised a back analysis of both the platforms and retaining structures, assessing the overall the slope overall stability. The analysis was performed using the PLAXIS 2D finite element software. The calibration of the geological and geotechnical model was performed in order to try to reproduce the horizontal displacements recorded by the inclinometer's readings. The magnitude of the observed displacements, coincident with a calculation approach under static conditions, led to the formation of the sliding surfaces, which are indicated in Figure 7, corresponding to an overall safety factor of 1.12. The behavior of the slope was also analyzed for the seismic action, leading to a global safety factor inferior to 1.0, confirming the need to design a slope stabilization solution.

After the numerical model calibration through back analysis, it was used to design the stabilization solution. Table 2 presents, comparatively, the values related to safety to overall stability before and after the intervention.

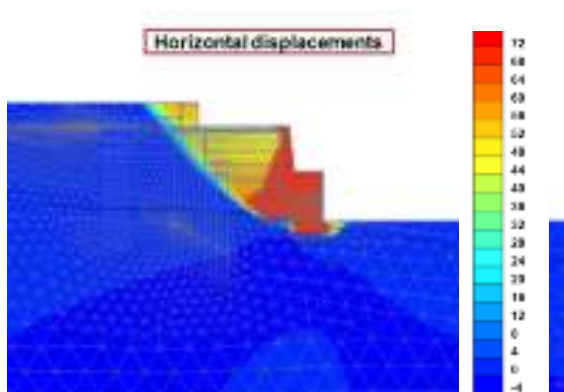


Figure 7. Numerical analysis for the evaluation of global stability to static actions [FS=1.12], horizontal displacements (cm).

According to the numerical analysis it was possible to conclude that the stabilization solution will lead to an important increase of the slope overall stability for both static and seismic actions.

Table 2. Summary of pre and post intervention overall stability safety factors

Situation	Static	Dynamics
Before Interventions	1.12	<1,00
After Intervention	1.70	1.25

## 6 SITE WORKS

The stabilization and drainage works were conditioned by the small space at the lower and working platform, mainly for the bored piles execution, leading for a very demanding organization and activities planning. The works began with the execution of the bored piles, for both the stabilization solution, using guide walls, and for the crane foundations.

During the execution of the piles, soft materials were intersected, as well as the remains of old constructions, which determined the need to use various drilling techniques, from temporary casing to soil and rock augers (Figure 8).



Figure 8. View of the piles works.

Following the execution of the bored piles, the remaining work was carried out with lower restraints, mainly the capping beams, the rods and the nails (Figures 9 and 10).



Figure 9. View of the excavation works.



Figure 10. View of the nail's execution.

At the end the LWA was installed (Figure 11) over a waterproofing membrane, in order to drive the water to the drainage systems, mainly longitudinal drains and geodrains. At this moment,

only the garden reinstatement is missing, leading to the installation of a vegetal soil layer, over the LWA fill, necessary for the replanting of the vegetation.



Figure 11. View of the lightweight aggregates

The last structural intervention was the cleaning and rehabilitation of the lower wall, facing the Taipas Street, including the repair of existing cracks.

## 7 MONITORING AND SURVEY

As already stated, during and after the execution of the stabilization works, several devices have been installed and reinstalled allowing the monitoring and survey of the SPA viewpoint slope and walls, confirming as fundamental risk management tools. After the stabilization works conclusion, the monitoring system includes topographic targets installed at the walls, as well as a inclinometers and piezometers.

The inclinometers and piezometers allowed to evaluate the lateral deformations in depth and the measurement the ground water level. On the other hand, the topographic devices allow the measurement of 3D displacements, at the original walls.

According to the monitoring results, about 1.5 years after the completion of the stabilizations and drainage works it was possible to confirm the stable behavior of the SPA viewpoint slope (Figure 12).



Figure 12. View of the final works

## 8 FINAL REMARKS

Following previous and similar works (Pinto, 2007 and Pinto, 2016) in this paper it was described a case study where the importance of both the geological and geotechnical information, as well as the monitoring and survey works, were key issues in order manage the geotechnical risk, allowing to predict and to anticipate the execution of stabilization and drainage works of a slope located at one of the most sensitives places of the Lisbon Historical Center.

In spite of the complex works, the deadline and the budget for most of the works were fulfilled.

## 9 ACKNOWLEDGEMENTS

The authors are grateful to the site owner, for his permission to the presentation of this paper.

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# The use of BIM technology in geotechnical engineering

## L'utilisation de la technologie BIM en ingénierie géotechnique

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**ABSTRACT:** The degree of uncertainty making part of the geotechnical engineering, associated with the need for constant collaboration from the project to the construction site stages, makes the Building Information Modeling (BIM) a methodology to explore in all the construction sector, including the geotechnical engineering field. In this paper, the case study of the BIM technology applied to an underground parking lot project in Lisbon center, Portugal, will be shown with the purpose to demonstrate the advantages of using the BIM methodology. The paper will point out the BIM use for a geotechnical project, ranging from the design to the pre-execution. With this purpose, a 3D BIM model was created along with the topography and geotechnical / geological layers and then, exploring several tasks, ranging from the structural design to the construction site scheduling, quantities and cost measurement activities, generating the 4D and 5D models. Several software tools were used, testing the interoperability among different platforms.

**RÉSUMÉ:** Le degré d'incertitude inhérent à l'ingénierie géotechnique, associé à la nécessité d'une collaboration constante du projet aux étapes du chantier, fait de la *Building Information Modeling* (BIM) une méthodologie à explorer dans tout le secteur de la construction, y compris le génie géotechnique. Dans cet article, l'étude de cas de la technologie BIM appliquée à un projet de parking souterrain dans le centre de Lisbonne, au Portugal, sera présentée dans le but de démontrer les avantages de la méthodologie BIM. Le papier mettra en évidence l'utilisation BIM pour un projet géotechnique, de la conception à la pré-exécution. À cette fin, un modèle 3D BIM a été créé avec les couches topographiques et géotechniques / géologiques, puis en explorant plusieurs tâches, allant de la conception structurelle à la planification des chantiers, des quantités et des activités de mesure générant les modèles 4D et 5D. Plusieurs outils logiciels ont été utilisés pour tester l'interopérabilité entre différentes plates-formes.

**Keywords:** BIM; Geotechnical Engineering.

## 1 INTRODUCTION

Performance issues in construction has been discussed throughout the 20<sup>th</sup> century and they are still real nowadays. In 1983, a study was done to find out the reason behind the lack of productivity of the construction sector. It referred that more than the half of the time wasted in construction activities was due to the lack of effective management practices and that it was

needed “*a more timely and accurate control over design, planning and scheduling, budgeting, procurement, material logistics, and quality assurance*”

In the design process, the broadly used CAD 2D systems are error-prone, sometimes due to the difficulty in visualizing 3D elements and detecting clashes (Eastman *et al.*, 1974). The reliance on 2D models creates the need for at least two drawings of the same piece of the element to

represent and interpret it in a tridimensional way. In the case of geotechnical engineering, according to the National Economic Development Office (UK), 37% of the schedule overruns had ground problems as a major cause. This kind of projects are based on data obtained by geotechnical investigation and the degree of uncertainty is high. Another characteristic of this sector is the risk associated for both the construction staff and for the structural integrity in case of failure or collapse of this kind of geotechnical structures. This makes the underground construction both a physical and a financial risk (Sterling, 2017). The geotechnical projects need an early constant coordination with the structure and architecture disciplines. This process of interdisciplinary collaboration must be fluent to increase the project's delivery time.

### 1.1 BIM concept

Building Information Modeling (BIM) concept appeared around 1974. Eastman identified some of the weaknesses of the architectural drawings at the time. He considered them to have “many inherent weaknesses” and to be “highly redundant”. He envisioned what he called at the time a “Building Description System” (BDS), with representation based on objects containing information (Eastman *et al.*, 1974). The BIM concept is a methodology and a framework that is based on the use of parametric elements that include, not only a tri-dimensional geometry but also other useful information for the whole building's lifecycle, from the design to the exploitation, and its deconstruction. This methodology promotes an early stage design team-up with an improved collaboration among the different project's stakeholders. The collaboration is guaranteed by a high degree of interoperability – the capacity of seamlessly exchange information between different platforms.

In a first stage of the project, a tri-dimensional parametric model is created. This model is a visual representation of the project, very close to

the real one yet to be built. It characterizes spatial relationships, geometry, and geographic information. A BIM based software is used to make the integration among disciplines of the project and to navigate inside the model. This stage is designated by 3D/BIM. Adding the time variable to the 3D objects generates the 4D/BIM model. This time information makes possible the visualization of the construction processes through the project schedule, allowing a simulation of the project and a better control of allocated resources and the necessities at different stages of the execution. With the cost information a 5D/BIM model is obtained and it is possible to simulate and compare different constructive solutions or materials and to have a tighter control over the budget along the execution plan. The dimension 6D/BIM concerns the energetic and sustainability performance and the 7D/BIM is usually related to facilities management capability.

## 2 3D/BIM MODEL

### 2.1 Case study

The research is based on a public underground parking lot project in Lisbon that includes the excavation and construction of four underground floors and the corresponding exit ramp. Figure 1 shows the project's deployment area and some of the main construction restraints that include the surrounding streets and infrastructures, especially the future Learning Center of Lisbon University and the Lisbon's Metro line.

The geological and geotechnical conditions were assessed by four Standard Penetration Tests (SPT) and a piezometer, to determine the groundwater level position. After the SPT test were completed, the site ground area was divided into four geotechnical zones (table 1) that were also used to create the surface representation in the 3D model.

Table 1. Geotechnical zones and parameters

Geotechnical Zone	ZG3	ZG2A	ZG2B	ZG2
	Fills	Miocene		
Unit weight $\gamma$ (kN/m <sup>3</sup> )	18	19	20	20
Internal friction angle $\phi'$ (°)	24	32	34	36
Cohesion (kPa)	-	5	10	20
Young modulus (MPa)	3	10	20	50
N <sub>spt</sub>	0-7	6-26	30-45	60

confinement and protection of this soil. Geodrains were placed with an average spacing of 3.60m at all floor levels to ensure the suitable drainage conditions.

The BP were braced by two levels of temporary ground anchors and steel props, at the corners. At each level, the bored piles were braced by distribution beams and, at the top, by a capping beam.

Regarding the general execution process, a survey to neighboring infrastructures and preparation of the site was done with diversion of the traffic and services affected and the instrumentation plan is set up. The peripheral BP



Figure 1. Excavation site and surroundings. Source: adapted from Google Maps

The increase in displacements from the excavation works must be controlled and monitored throughout the execution stage by the application of an instrumentation plan.

The designed solutions for the retaining structure were a peripheral retaining structure, with the execution of bored piles (BP) and by the execution of king-post walls in the alignment adjoined to the future Learning Center. The BP system adopted included piles with 600mm diameter with 1.20m spacing. These piles have a length of approximately 16.0m to cover the depth of the excavation and an embedment length of 4.0m. The soil between the piles was lined after the excavation by a 4+4cm layer of shotcrete for

piles and underpinning of the “Gare do Arco do Cego” columns were executed followed by the cleaning of the piles top and execution of the capping beam. The temporary ground anchors and the corner props were installed after the concrete hardening and the excavation till the level of the first distribution beam can be performed. Geodrains were installed along with two layers of shotcrete (4+4cm) and the distribution beam is executed with the next level of ground anchors. This process was repeated until the bottom of the excavation is reached and the internal structure could be built, from the base to the top, deactivating the anchors and props.

## 2.2 Parametric modeling

The software used to produce the 3D model was the Autodesk Revit 2018 that guarantees IFC interoperability with other applications. It comes with a library of parametric elements and allows the modeling of new ones.

The preparation of the model is important and can avoid time losses and increase the modeling efficiency in the long-run (Thakur and Rao, 2014). For this project, the units, project phases, and survey points were early defined. The definition of phases gives the model a time variable for the project. Having this information associated with the parametric elements will allow a quantity take-off associated with each phase and ease the graphic representation of the model for each defined construction stage. In this case it was used for the measure of quantities for the earth works.

### 2.2.1 Retaining structure

The model for the retaining structure was created based on the project documentation available in 2D CAD drawing files and the descriptive memory. The modeling started by importing CAD drawing to the modeling pane.



Figure 2. Rebar intersection representation

Then, the bored piles were modeled, following the distribution and capping beams along with the slabs and the reinforcement details.

The objects automatically adjust their length according to the defined constraint's position and their geometry adapts according to it. These

changes are spread across all views and representations of the project.

Some objects were not available in Revit library and had to be created. This happened with the ground anchor object and some annotations to represent the instrumentation devices.

The ground anchors are an important element to represent. Its geographic position should be accurate and the characteristics clear for the contractor to execute. To create the ground anchor, an object was found in an online BIM library although with some characteristics missing such as the anchor bulb – an important feature to represent. This element was thus edited, creating the required parameters and geometry associated with its material information. The parameters created were the bulb length, the bulb diameter, as well as the total anchor length, defined by the free length plus the bulb length. It was also required to define the material of the grout bulb created.



Figure 3. Complete model of the retaining structure

With a BIM approach, any kind of geometric element can be defined and associated with parameter values to meet the user's requirements. These parameters can then be used for quantities take-off, construction planning or facilities management. This flexibility allows several kinds of project stakeholders to take part in BIM methodology and be a support for the study of design changes, as well as for the decision-making process.

### 2.2.2 Topography surface

In geotechnical projects, the topography and subsurface are especially important to support the design solutions and estimate earthwork quantities. The surface was modeled using Revit tools by importing a 2D CAD file with elevation points. The subsurface and the geotechnical zones were defined using the *Autodesk AutoCAD Civil 3D*, that has a module specific for the purpose.

With the surfaces added to the model, it was possible to confirm that the length and the vertical angles of the anchors are compatible with the soil characteristics, and some optimizations were possible.

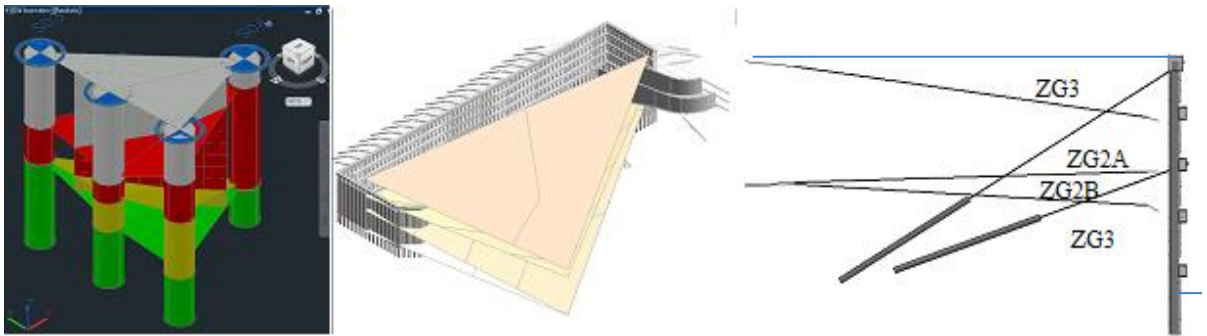


Figure 4. Geological zones in Civil 3D (left), in Revit (middle), and compatibility analysis (right)

### 2.2.3 Connection with a structural design software

Since the modeling in BIM is done using parameters that represent geometry and material properties attributed to elements, this information can be used to support the structural design. It avoids the reduplication of work when modeling the structure in the respective analysis software. Revit creates an analytical model along with the geometric one, which should be prepared to be sent to the structural analysis software. It is important to ensure that the elements are properly linked and that there are no unconnected nodes. The used software was the Robot Structural Analysis, from the same software provider of Revit.

The interoperability is improved if the analytical model is prepared beforehand, what

can be time consuming. It is although an advantage to have the information regarding the sections, the grids and the materials of the elements automatically ready after the export. Defining the names of the materials in Revit according to the ones used in the Eurocodes, provided by Robot software, will save time adjusting the section properties in this software. After optimizing the solution in Robot, the changes in the sections are assumed by Revit, as the models are linked. So, when the cross-section of an element is changed due to optimizations or to ensure design code's requirements, the changes are reflected in the 3D model and in all created views and sheets.

## 3 CONSTRUCTION MANAGEMENT BASED ON BIM

The software used to create the 4D and 5D models was the *STR Vision CPM* from *TeamSystem*. It does guarantee interoperability with several programs used by AEC sector including IFC and scheduling files types. This software allows the budget management of several projects simultaneously with several tools to support construction managers' activities.

### 3.1 Construction planning based on a 4D/BIM model

The process of planning a construction project is a complex task that includes several interrelated variables, sometimes difficult to predict. The site construction manager usually receives data from the different disciplines (geotechnical, structure, architecture, mechanical, electrical and plumbing, etc.) and must combine those avoiding clashes and by promoting health, safety, and quality of the finished works. Traditionally, the site planning process is done based on two-dimensional drawings, what makes it a difficult task. The schedule needs to be interpreted by the construction managers sometimes just based on abstraction.



Figure 5. 4D model

Some of the advantages of the 4D-ready process are an improved team collaboration, improved control of logistics, acknowledge of the manpower allocation, sequencing of operations and facilitated re-scheduling, what reflects in an increased value of the proposals. It is also useful for other stakeholders in the project, for example, to show the project owner different options in the construction processes that can be adjusted according to eventual set-backs found during the construction, usual to happen in geotechnical works.

To create the 4D/BIM model, the construction plan was done based on the project documents available and using reference values for the activities' production. The Critical Path Method (CPM) was used and the main activities were

then assigned to the estimated production rates and the respective duration was determined. The critical path determined was composed by the execution of the piles, followed by the excavation and the execution of the capping beam and the distribution beams with the anchors at the different levels. The excavation must be coordinated with the execution of the piles, beams and corresponding anchors to avoid an excessive reduction of the soil's passive impulse and the risk of collapse of the retaining structure. The excavation should then be made just until the bottom level of the last beam executed. The excavation activity was broken-down in levels from 0 to -4. It was considered that the excavation could be done in a sloped shape in the direction of the center of the exasite. The first level of excavation and the execution of piles' activity had an end-to-end relationship. This means that, for the excavation to be done till the level -1, was necessary the execution of piles to be also completed. Besides, a lag was added to some activities to guarantee, for example, the hardening of the piles' concrete.

The standard IFC file of the 3D model was used by exporting it in Revit. The IFC file type recognized correctly the parameters attributed to the objects. After uploading the 3D/BIM model to the software, the scheduled activities were connected to the objects. To do this, the planning created in MS Project was also imported to by the STR Vision CPM software. When this step was completed, it was possible to generate a simulated animation of the construction process (figure 5).

### 3.2 Budget management based on a 5D/BIM model

With BIM technology, the information regarding the cost and suppliers can be linked to the 3D objects. The budget management and the cost evaluation are useful tools for contractors and designers. The quantities take-off capabilities can help avoiding mistakes, common in the traditional CAD 2D approach.

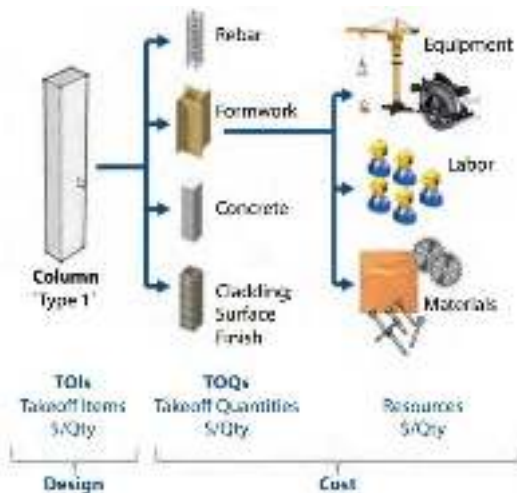


Figure 6. 5D/BIM data flow. Source: Vicosoft

### 3.2.1 Quantities measurement based on BIM

To create the 5D/BIM model, the resources that each parametric element needs to be executed should be first specified. Then, when this element is measured, all the materials, labor, equipment, time, and other relevant cost variables will be associated with it.

To make the quantity take-off, the procedure was the following: definition of different price categories; creation or import of the WBS; creation or import the items price lists; items analysis in terms of resources needed; measurement of quantities and creation of measuring rules.

When these steps are completed the measurements are done by selecting the 3D objects in the software visualizer and creating measurement rules that are saved for a future use. This can be particularly useful for an almost automatic quantities take-off. Also, if design changes happen – what is usually probable – the saved rules can be useful for a fast evaluation of the design changes impact in the global cost. The cost can be analyzed in customizable dashboards, for example, to relate the cost of each WBS. With the increase in the schedule detailing, the cost information will also be more accurate. To increase the detailing, it is necessary to reduce the length of the timeframe considered and associate the

parametric objects executed in this amount of time.

### 3.2.2 Procurement based on BIM

One available tool helps to facilitate the procurement and the evaluation of the supplier bids. By using the measurements done, a spreadsheet file can be exported with the prices for the supplier or the subcontractors to fill. After the file is filled in, it can be imported and associated to specific price categories for analysis of the best proposals. The price estimation can then be updated with the chosen proposal.

### 3.2.3 Cost analysis with design changes

Another useful tool is the capability to evaluate design changes in the 3D model and perform a budget comparison. This is done by using the previously set measuring rules. The two models are then displayed, showing the measurement differences between them. Then, a dashboard can be created to analyze impact of the changes in the budget (figure 7). This analysis can be performed by selecting different parameters to be the target of analysis such as the budget difference by WBS, by floor level, or by item.

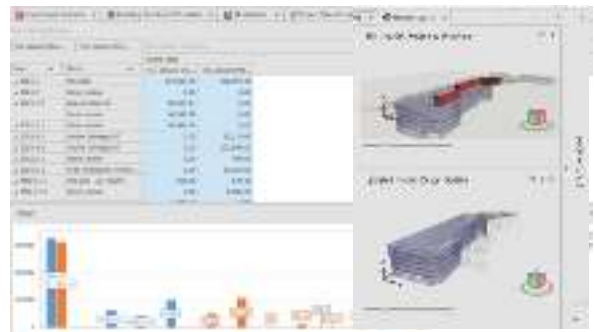


Figure 7. Design change cost analysis.

## 4 MAIN CONCLUSIONS

The purpose of this paper was to demonstrate how BIM methodology could be applied to a geotechnical project. To do so, a tri-dimensional model of a retaining wall structure was created

and then, it was used to explore the scheduling and construction management tools (Figure 8).



Figure 8. View of the final excavation works.

The interoperability was tested with interactions between different software platforms. Not always the interaction between different software was done smoothly. The exchange between Autodesk Revit model to the STR Vision CPM was done using the standard IFC file format. Some versions of IFC file type were not fully compatible, neither was found information regarding this topic from the software makers. Also, the connection between Revit and Robot needed some extra modeling time spend to adjust the analytical connections between objects. The creation of the analytical model by Revit seems to have a margin for improvement. Modeling needs some training experience to improve the models' quality and reduce the time required to complete the project's documentation. The drawings production through BIM had benefits when design changes were needed. Because the information is centralized, the changes in the 3D model are transmitted across the different drawings pieces. This advantage has a positive implication in the optimization of the engineering solutions, especially when project's delivery deadlines are short. The quantities take-off through BIM helped to verify and improve measurements done in the traditional way. The centralized

information gives a global overview for construction managers, that will reflect in a faster and more accurate decision-making.

Upstream changes in the project's design or downstream changes coming from product price changes could be quickly evaluated. With the data centralized in the BIM parametric elements, the information could be switched and updated, giving the project's manager an increased operational control. This allowed a consistent support for decision-making when analyzing multiple construction processes, design options or when evaluating the impact of price material changes.

The AEC industry can find in BIM, a route for its so needed increases in productivity because it is able to promote a leaner construction process, by reducing the reduplication of work and some traditional error-prone activities. By encouraging an early stage design collaboration, avoiding risks and increasing operational control over the whole project's lifecycle.

## 5 ACKNOWLEDGEMENTS

The authors are grateful to the site owner, for his permission to the presentation of this paper.

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# AR53 - Ground improvement and earth retaining solutions in Lisbon's downtown

## AR53 - Solutions d'amélioration et de rétention des sols dans le centre-ville de Lisbonne

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**ABSTRACT:** The presented case study takes place in one of Lisbon's main avenues, Almirante Reis 53 (AR53). At the site area there was an old habitation building, which was previously demolished. The building to be built, has 7 raised floors and 2 basement floors, each with approximately 225m<sup>2</sup>. Some of the most relevant restraints are the 60 years old tunnel of the Lisbon Metro, which is approximately 10m away from the excavation pit, the surrounding structures, as well as the geological conditions. In this scenario, it was decided to build a peripheral earth retaining structure, using a Berlin type wall, braced by steel props and slab bands. Prior to the excavation works and considering the characteristics of the surficial and very soft landfill deposits, these materials were treated with soil-cement columns. A monitoring plan was designed as a risk management tool, aiming to predict the behavior of all neighbor structures and infrastructures during the excavation works. The design was supported by a numerical analysis, using a Plaxis2D model, which aimed to predict deformations and stresses at the Berlin type wall and Lisbon Metro tunnel, for all the main construction phases. At last, a seismic analysis was developed, in order to guarantee the safety of the new building and its nearby constructions.

**RÉSUMÉ:** Le projet présenté se déroule dans l'une des principales avenues de Lisbonne, Almirante Reis 53 (AR53). Avant le projet il y avait un ancien bâtiment d'habitation, qui a été démoli avant le projet. Le bâtiment à construire sur comprend 7 étages surélevés et 2 sous-sols, chacun avec environ 225m<sup>2</sup>. Les obstacles plus importants, sont le tunnel du Métro de Lisbonne, âgée de 60 ans, à environ 10 mètres de distance, les structures environnantes et les conditions géologiques. Dans ce scénario, il a été décidé de construire une paroi périphérique, utilisant la technologie d'exécution du type Berlin, soutenue par des buttons et des bandes de dalle. Avant les travaux d'excavation et en tenant compte des faibles caractéristiques des remblais à la surface, ces matériaux ont été traités avec des colonnes de sol-ciment. Un plan de surveillance a été élaboré comme un outil de gestion des risques visant à prévoir le comportement de toutes les structures pendant les travaux d'excavation. La conception a été calibrée par une analyse numérique utilisant un modèle Plaxis2D, qui visait à estimer les déformations et les contraintes au mur de Berlin et bien aussi à la galerie du Métro de Lisbonne à toutes les principales phases de construction. A la fin, une analyse sismique a été développée afin de garantir la sécurité du bâtiment et de ses constructions voisines.

**Keywords:** Berlin type wall; Soil-cement columns; Micropiles; Monitoring, Deformations.

## 1 INTRODUCTION

As a consequence to the rise of the construction and touristic market in Portugal, numerous new constructions and rehabilitations are being held for accommodation purpose. The presented project takes place in Almirante Reis (Figure 1), one of Lisbon’s main avenues, located over an old water stream, in a dense urban area. The architectural design of the building to be built at the site, “Almirante Reis 53” (AR53), has 7 raised floors, intended for hotel purpose, and 2 basement floors, destined for restaurant and technical areas. The building provides an approximately square plan shape, with a total area of about 225m<sup>2</sup> per floor.

At the site, there was currently a three-raised floor habitation building, with an approximately square implantation area of about 15×15m<sup>2</sup>, which was previously demolished. The geometry of the building, the topography of the site, the geotechnical and geological conditions, lead to an excavation with a maximum depth of 9m. In this scenario, it was decided to build a solution for the earth retaining structure, using a Berlin type wall, braced by steel props and slab bands, avoiding the use of temporary ground anchors.

## 2 MAIN RESTRAINTS

Some of the most relevant restraints of this project are the 60 years, plain concrete, Lisbon’s Metro tunnel, which is approximately 10m away from the excavation pit, the surrounding structures and infrastructures, as well as the geological conditions, which are below described.

### 2.1 Geological and geotechnical conditions

A site investigation campaign was carried out in order to allow the geological and geotechnical characterization. Four boreholes, with SPT tests, were carried out, as well as two shafts, for confirmation of the neighbour buildings geometry and foundation levels, were executed.

Based on the information provided by the ground investigation campaign, geotechnical and geological zones and parameters were defined (Table 1). Regarding the site geology, landfill deposits, characterized by silt sand with fragments of variable dispersed nature, were intersected between 1.5 and 3m depth. Below the landfills, a layer of silty clay, occasionally sandy (N<sub>SPT</sub> blows between 7 and 44), due to the old water stream, with an overall thickness ranging between 1.5 to 14.5m, was observed. At the



Figure 1 - Site location and conditioning (source: adapted from Google Maps)

boreholes base, the Miocene sandstones were observed, at a depth ranging from 12.5 to 14.5m. It should be pointed out that the groundwater table was not observed above the final excavation level.

*Table 1. Geotechnical parameters and zones*

<b>Geotechnical Zone</b>	<b>N<sub>SPR</sub></b> [-]	<b>φ'</b> [°]	<b>γ</b> [kN/m <sup>3</sup> ]	<b>E</b> [MPa]
ZG1 - Landfills, silt sands	<5	26	16	6
ZG2 - Silty clay, sometimes sandy	7-44	31	18	30
ZG3 - Miocene sandstones	>60	38	20	100

## 2.2 Demolition

Demolition in urban areas are always one of the most conditioning and dangerous part of the project. Given the age of the building, as well as the lack of information about the construction materials and its degradation status, additional prevention measures had to be considered in order to guarantee a controlled and safe demolition. In this case, the damaged structural status of a neighbour building at Álvaro Coutinho Street, demanded the need to install a façade retention structure, founded over micropiles, in order to allow the building stabilization during the future excavation works. In Figure 2, an example of a prevention measure can be observed.



*Figure 2 - Shoring of the arches and installation of steel props (September 2018)*

The shoring of the existent arches and the installation of steel props, in order to guarantee the execution of the façade retention structure and the micropiles foundation cap.

## 2.3 Neighbourhood conditions

The excavation enclosure is in an urbanized area, being delimited by buildings (to the north and west, with two and five raised floors), streets (Av. Almirante Reis at east, including the Lisbon Metro tunnel, and Álvaro Coutinho Street at south) and various infrastructures (Figure 1). As said, in order to control possible deformations due to decompression of the old and damaged building facing west, at Álvaro Coutinho street, previous to the demolition works, two facades retention towers were installed and founded over micropiles.

Due to the location on a dense urban area, the risk of intersection of utilities networks, mainly: Lisbon Tunnel, rainwater collectors, gas network, telecommunications, electricity and water, lead to the design of earth retaining structures with the objective to avoid temporary ground anchors.

Other key restraint was the lack of space on site to use earth retaining solutions demanding big and heavy equipment's, as pile walls or diaphragm walls.

## 3 ADOPTED SOLUTIONS

In this scenario, it was decided to adopt a solution for the peripheral earth retaining structure, using a Berlin type wall, with reinforced concrete panels, braced by steel props and slab bands and vertically supported by micropiles. This type of solution has the advantage of allowing the final wall to be executed during excavation. The main design scopes were the following:

- Deformations control: ground and neighbour constructions and infrastructures;
- Minimize possible interference with the Lisbon Metro tunnel;
- Guarantee ease, speed and safe execution.

Before the beginning of the excavation works and considering both the Berlin type wall and the weak characteristics of the ZG1 and ZG2 materials, a ground improvement solution with soil-cement columns was executed.

### 3.1 Earth retaining solution

The architectural project includes two basement floors, which require, due to the above cited restraints, a vertical excavation, supported by an earth retaining wall. In this scenario, the Berlin-type wall (king post with reinforced concrete cast in situ panels) is one of the most suitable techniques, as it takes advantage of the staged construction, allowing to minimize walls thickness and back structures and infrastructures displacements. However, it is crucial that the design guidelines are respected, mainly, the excavation stages. This technique consists in phased retaining wall execution, from top to bottom, of 0.30m net thickness reinforced concrete panels, supported by steel vertical tubular micropiles. The panels are casted directly against the excavated soil face and braced by slab bands and temporary corner steel props at two levels. Temporary ground anchors were avoided given the proximity of the Lisbon Metro Tunnel and the neighbouring buildings shallow foundations. At the final stage, the new building basement slabs assure the stability of the retaining walls.

### 3.2 Ground improvement

Prior to the beginning of the excavation works and considering the weak characteristics of the soft landfill deposits, these materials were treated with soil-cement columns, in order to allow the execution of the Berlin wall solution (Figure 3).

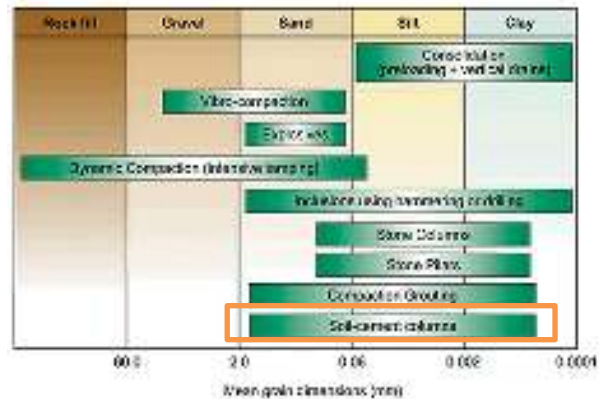


Figure 3 – Main ground improvement techniques (source: Soletanche Bachy)

## 4 DESIGN

### 4.1 Numerical Analysis

The design was supported by a numerical analysis using a Plaxis2D model. Geometry input, soil and interfaces properties were chosen to replicate, as best as possible, the local conditions. The main scope was to predict deformations and stresses at the Berlin type wall for all the main construction phases, as well as to calibrate the Monitoring and Survey Plan.

The most representative and conditioning sections were analysed, including the Lisbon Metro tunnel (Figure 4), in order to evaluate stresses and strains, as well as the stability of the retaining walls, the ground settlements, the forces at the temporary steel props and slab bands, as well as the micropiles axial loads.

Nonetheless, elements such as capping beams were analyzed considering simplified models, using concepts of the classical theory of elastic bars as well as strut and tie reinforced concrete models.

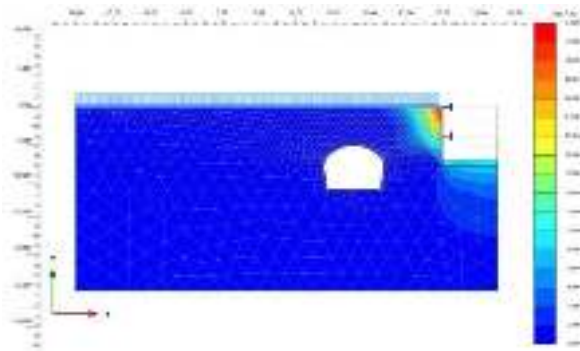


Figure 4- Plaxis 2D model of the Berlin wall and tunnel cross section (total estimated displacement of 33mm)

Taking into account the geological conditions as well as the location of the building foundations, a seismic analysis was developed, in order to guarantee the safety in terms of overall stability of the earth retaining structure, as well as the safety of the new building and its nearby constructions. Using the Plaxis2D software and considering the seismic action according to EC8 and the National Portuguese Annex, the following analysis were made:

- Loss of overall stability;
- Safety checking, ULS and SLS, for all the sections considered as conditioning and representative.

#### 4.2 Micropiles

In this project, micropiles were used for several foundation purposes, namely the earth retaining structure, the new building foundations, the tower crane and the façade retention towers.

The micropiles were designed to transmit their loads to the Miocene layer mainly by lateral friction. The safety verification associated with the ground bearing capacity was carried out through the Bustamante method (Bustamante, 1985), which allows to quantify the sealing length. The credibility of this method is proven by its wide application, as well as the fact that it was developed based on a large number of experimental results, even at the Lisbon Miocene soils.

## 5 MONITORING PLAN

A monitoring plan was designed as a risk management important tool, in order to ensure the execution of the excavation works in safe and economic conditions. The plan aims to confirm, on time, the predicted behaviour of the earth retaining structure, as well as of neighbouring structures and infrastructures during and after the excavation works.

Regarding the Lisbon Metro tunnel, a specific monitoring plan had to be developed, given its proximity (about 10m) to the excavation pit area (Figure 5).

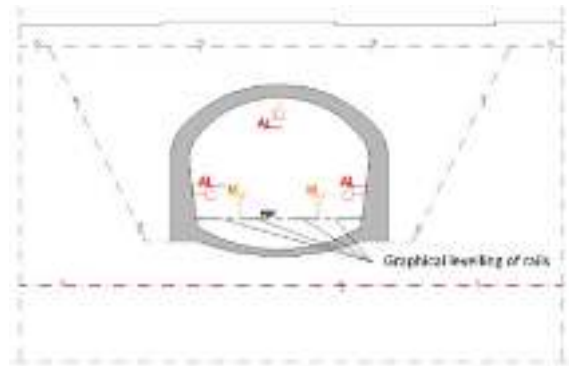


Figure 5 - Monitoring of the Lisbon Metro tunnel

The monitoring plan will be able to provide mainly the following data:

- Vertical and horizontal displacements of:
  - The retaining wall;
  - The neighbouring buildings;
  - The Lisbon Metro tunnel.
- Horizontal displacements of the retained ground;
- Ground water table depth.

In order to obtain the above data, the following main devices were installed: topographic targets, topographic levellings and inclinometers.

## 6 MAIN CONCLUSIONS

Considering the restraints faced, mainly the nearby constructions, such as Lisbon Metro tunnel, it was possible to overcome them by using a Berlin type wall solution, associated with the previous ground improvement, using soil-cement columns, braced by slab bands, which demonstrates the efficiency and versatility of this technique (Pinto, 2017 and Pinto, 2015).

In spite of at this stage just the demolition works have already been done (Figure 6), the adoption of the described earth retaining solutions has many advantages, mainly: the use of equipment with small dimensions and high versatility, the possibility to perform the final wall simultaneously with the excavation, as well as the possibility to apply ground improvement techniques with the same small equipment, when intersecting softer soils, as the soil-cement columns solution (Aleixo, 2018).



Figure 6 – Current phase of the site works: facade retention of the neighbour building and demolition (March 2019)

As usual in this kind of projects, the developed model will be calibrated and validated on time by the monitoring plan results, during the excavation works.

## 7 ACKNOWLEDGEMENTS

The authors are grateful to the site owner, for his permission to the presentation of this paper.

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