Excavation and Peripheral Earth Retaining Solutions at the Convent of Santa Joana in Lisbon

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Abstract

This thesis presents the analysis of an earth retaining and underpinning solution of a centenary wall, located in the former Convent of Santa Joana, at Lisbon center, preceded by a back analysis of the geotechnical parameters used to model the solution, based on the results of the site monitoring and observation plan. The objective is to evaluate the importance of the ground characterization for the design of the geotechnical solutions, based not only on the site geological and geotechnical investigation results, field tests and laboratory tests, but also on the study of similar solutions adopted in the same geological and geotechnical conditions. To validate the main assumptions, the critical cross section was modelled, using a stress – strain finite elements analysis, with the Plaxis 2D software. Later, two alternative solution were proposed followed by its design, allowing a comparative analysis, technical and economic, with the adopted solution. It is concluded that the geotechnical parameters used to design the geotechnical solutions, often based on geological and geotechnical site investigation reports, are sometimes quite conservative, leading to unnecessary expensive solutions.

Keywords: Reinforced Concrete King Post Wall; Geotechnics; Earth Retaining Wall; Numerical Modeling.

1. Introduction

In heavily urbanized cities, the space for construction at the surface is growing scarce. With this comes the need to build in depth to better use the area and to maximize the useful area which requires even more demanding geotechnical solutions.

In this dissertation we will approach a study case of a foundation reinforcement and underpinning of a centenary gravity wall, located at Rua Camilo Castelo Branco, in Lisbon, which is integrated in the expansion work with demolition of the former Convent of Santa Joana, to achieve a hotel purpose.

The excavation was done along the wall in depth, and the main challenge was to ensure that it continues to fulfill its earth retaining function, without suffering any kind of structural damage and without causing damage to neighboring structures.

This dissertation contemplates the study of the solution adopted as well as the design and modeling of two alternative solutions, and for this purpose the results of monitoring and observation obtained with the adopted solution were analyzed. This back analysis allowed to calibrate the numerical model, performed using the finite element program, PLAXIS 2D. Thereafter, this calibrated model was used to design two alternative solutions.

2. Gravity walls

Gravity walls are structures used for land earth retaining and can consist of stone, gabion, and plain concrete. In this type of structures, the gravitational forces, especially the weight of the structure itself, play an important role in the stability of the structure, equilibrating the horizontal earth pressures (Gercovich, 2014).

3. Reinforced Concrete King Post Wall

The reinforced concrete king post wall is a retaining structure that is part of the category of multi-supported flexible earth retaining structures. Basically, it consists in the phased execution of reinforced concrete panels, which are later anchored or shoring. The panels of the reinforced concrete king post wall are excavated (from top to bottom) and executed alternately to take advantage of the "arch effect", that is, the ground where the excavation is performed will lost confinement, which leads to a redistribution of efforts to the lateral and confined ground, which, in this case, it will be the area that is still to be excavated. In this way it is possible to reinforce the excavated area, with the execution of the reinforced concrete panels, without a significant loss of ground confinement, allowing the subsequent excavation of the adjacent panels, taking advantage of the redistribution of efforts generated by the construction phase.

4. Ground anchors

A ground anchor is a structural element installed on soil or rocky mass that will be able to transmit an applied tension load to the ground to equilibrate the ground earth pressures and, mainly, control de ground deformations.

The ground anchors are prestress before they are put into service, limiting the deformation of the structure and in some cases, recovering part of the existing deformation, being therefore called active bracing elements.

There are also the passive ground anchors, or nailing, which are only activated with the displacement of the structure and the ground. They are quite similar to ground anchors, but no prestress is applied.

5. Micropiles

A micropiles is, as its name implies, a small diameter pile, usually up to 300mm, drilled into the ground and injected under pressure with cement syrup being structurally reinforced through pipes, metal profiles and steel rods. The main advantage of using micropiles is that they are carried out with light and small equipment and can be used where conventional pile equipment cannot work in addition to not causing large vibrations or noise. In addition, they can be run on various types of terrain supporting variable loads, usually between 150kN and 2000kN. They have the advantage of working well to both compression and tension, transmitting the loads by lateral friction in the sealing bulb area (Machado).

6. Case study

The accompanying work, case study of this dissertation, located in Lisbon, between Rua de Santa Marta (Northeast) and Rua Camilo Castelo Branco (Southwest). The implantation area is 3.463m^2, having a construction area of 22.271m^2, being the excavation area about 2.600m^2, visible in green in Figure 1. The building to be built will have 2 to 3 floors buried, depending on the area, with a maximum excavation depth of 12 meters.

Each zone has its constraints, and the earth retaining structure is adapted to each, however, in general, the solution adopted will be Reinforced concrete king post wall, braced by ground anchors, struts and slab bands. In Figure 2 it is possible to observe the peripheral retaining solutions adopted as well as the zone that will be analyzed.

Figure 1 – Site location

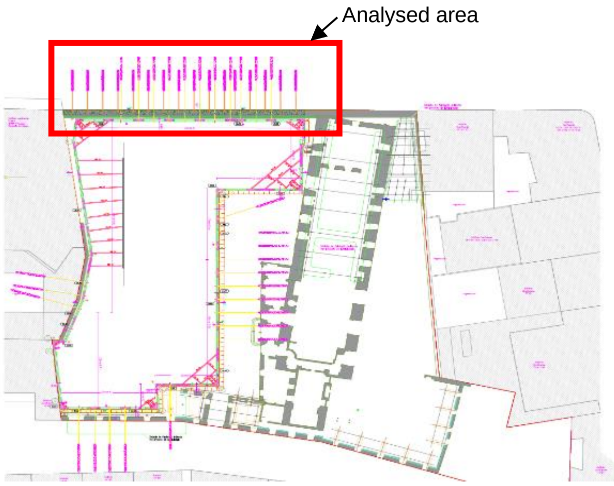

Figure 2 - Peripheral containment solution plant with identification of the study area (JETsj, 2020)

6.1. Geological and geotechnical scenario

It is recalled that, in addition to starting any project related to the construction of an infrastructure, it is essential to know the geological and geotechnical scenario for the choice of the type of peripheral earth retaining walls and foundations to be adopted.

From the results of the geotechnical investigations, it was possible to observe that the ground at the intervention zone is characterized by landfills, which constitutes the most recent layer with a thickness ranging between 2.70m and 10m with 5≤NSPT≤40. Underlying this layer Oligocene materials were found, represented by the unit called "*Formação de Benfica*", mainly silt-clayey and sometimes sandy-silty materials, dated from the Paleogene period. (GEOCONTROLE, 2016) This layer has thicknesses ranging from 8.0m to 13.0 constituting the foundation of the existent centenary wall in analysis.

Finally, at a depth of approximately 18m is the unit called Lisbon Volcanic Complex, from the Upper Cretaceous period, with NSPT≥60, was found.

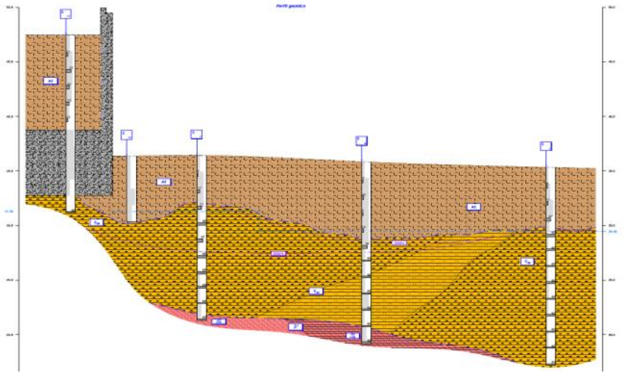

Figure 3 - Geological-geotechnical profile (Geological-geotechnical report, Geocontrole, 2016)

6.2. Characterization of the centenary retaining wall

So that you can get more information about the constitution of the gravity wall under analysis, two inclined core holes and one horizontal core hole were performed on the support wall. Through laboratory tests it was possible to estimate the parameters that characterize it.

The wall consists of stone masonry, more specifically volcanic blocks and/or limestones, assuming a height of 15m, of which 10m make the transition between the Camilo Castelo Branco Street at the crest and the site at the base. It is estimated the base is 5m width, with a trapezoidal section and a shallow foundation over the Paleogene materials (Figure 4). (GEOCONTROLE, 2016)

To model the proposed solution, the parameters that characterize the behavior of the material were estimated and presented in the Table 1.

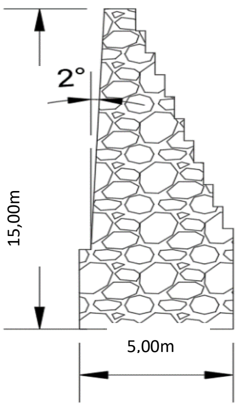

Table 1 - Existent Wall mechanical characteristics

Centenary wall	
γ [kN/m³]	22
E_{ref} [GPa]	7
ν	0,25

Figure 4 – Possible cross section of the existent wall

6.3. Reinforcement and underpinning of the centenary wall solution

The adopted solution was row of micropile at the wall crest, sealed in Paleogene materials with competence for the purpose, together with the execution of the Reinforced Concrete King Post Wall.

7. Monitoring and observation plan

It is extremely important to have a well-designed instrumentation and observation plan so that the demands of the work can be followed in real time. With an instrumentation plan that transmits a good degree of confidence to the designer, it is possible to make a more optimistic project, always with alternatives in mind that allow to react in a timely manner if necessary. For the definition of the alert and alarm criteria, the values obtained through the modeling of the solution in finite element program were used as the basis for its definition.

These values are specified by the designer (Table 2) and are used as guidelines for the analysis of the results obtained through the readings of the instruments.

Table 2 - Alert and alarm criteria defined by JETsj (Descriptive memory of the project, by JETsj)

Level	Criteria		Action
	Topography		
	Deformation rate	Maximum deformation	
1	<1mm/day	δ_H<25mm δ_V<25mm	Stable
2 (Alert)	1-5mm/day	δ_H=25 – 50mm δ_V<25 – 50mm	Communication to the entities involved, special monitoring, verification of readings, preparation of the action plan
3 (Alarm)	>5mm/day	δ_H>50mm δ_V>50mm	Communication to the entities involved, verification of readings, increased frequency of readings and implementation of security replacement measures

8. Numerical model of the adopted solution

The modeling of the proposed solution, in the finite element program Plaxis 2D, version 19, was used to simulate the behavior of the ground during the execution of the work. The section with the highest excavation height was modeled because it was considered as the critical section, since the soil characteristics are constant along the entire wall.

8.1. Hardening soil model

In the modeling developed, soil behavior was simulated through the constitutive model Hardening Soil, which counts with plastic deformations since the beginning of the modeling as well as a nonlinear behavior of the soil. As the material is subjected to increasing cutting stresses, a decrease in stiffness is considered causing plastic deformations (irreversible). In addition, the increase of the deformability module with the increase of normal tension evolves hyperbolically and not linearly, as with the Morh-Coulomb model. Thus, it was considered that it would be more appropriate to consider the Hardening Soil model that uses the theory of plasticity as opposed to the Mohr-Coulomb model that uses the theory of elasticity.

8.2. Model geometry

To obtain results closer to reality it is necessary to try to reproduce the geometry of the elements in the best possible way, as well as the limits of the surrounding. To define the boundaries of the model, approximately 6 times the height of the excavation was taken as a reference to set the side boundary (left) and 3 times the excavation height to set the lower limit. In this way, the border was defined at 50m to the left side and 26m below the dimension of the excavation bottom to the lower limit. For the limit on the right side, it was considered 30m since it corresponds to half the length of the lot in that direction.

Figure 5 shows the geometry of the calculation model that was used to perform the stability analysis of the solution.

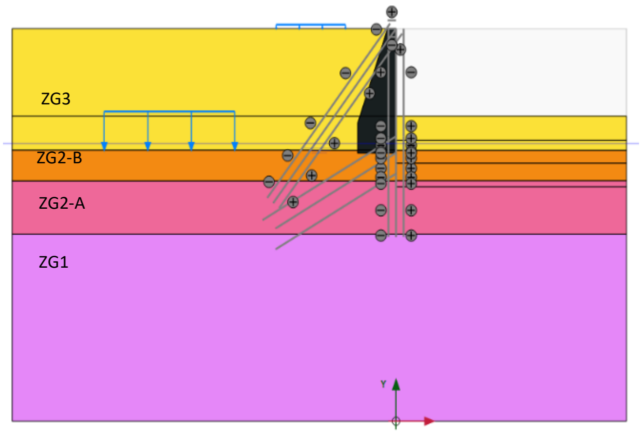

Figure 5 - Calculation model of the adopted solution, in program PLAXIS 2D.

8.3. Finite element mesh

To perform the modeling of the solution, a finite element program, PLAXIS 2D, was used, as described above. The basis of the program consists in the analysis of several points that constitute a previously defined mesh. The more refined the mesh is, more accurate the analysis is, since more nodes are considered, however, a heavier analysis is performed from the computational point of view, which is not always compensated by the quality of the results obtained. Thus, several meshes were analyzed and a fine mesh was chosen, refining only the points in which it was thought that the stress-strain relationship could be more significant and that they deserved special care to proceed with study (Figure 6).

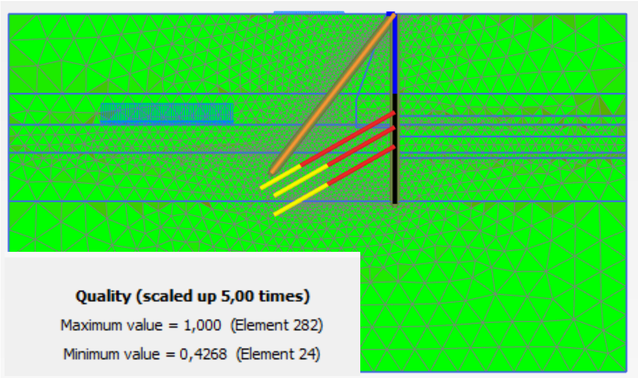

Figure 6 - Finite element mesh quality

8.4. Ground and structural element parameters

In addition to defining the geometry of the model, it is necessary to study which parameters will be used in the characterization of the elements and the soil. Thus, the necessary parameters were calculated, presented in Table 3 and Table 4.

Table 3 - Parameterization of the structural elements.

	Young modulus [GPa]	Inertia [m⁴]	Area [m²/m]	ϒ [kN/m³]	v	W [kN/m²]	EA	EI
Steel	210	-	-	78,5	0,3	-	-	-
Concrete C30/37	33	-	-	25	0,15	-	-	-
Reinforced Concrete King Post Wall (e=0,6m)	-	0,018	0,6	-	-	4,2	1,98E+07	5,94E+05
Micropiles Φ114,3x9mm	-	4,20E-06	2,98E-03	-	-	0,23	2,08E+05	2,94E+02
Micropiles Φ139,7x9mm	-	7,90E-06	3,70E-03	-	-	0,28	2,59E+05	5,53E+02
Anchors	-	-	7,00E-04	-	-	-	1,47E+05	-
Sealing bulb	7,07E+06	-	-	-	-	-	-	-

Table 4 - Soil parameterization – drain parameters

	γ [kN/m³]	P_{ref} [kPa]	E_{50}^{ref} [MPa]	E_{oed}^{ref} [MPa]	E_{ur}^{ref} [MPa]	m	C_{ref}	Φ'
ZG3 – B (Landfill)	18	100	15	15	45	0,8	5	30
ZG2 – B (Formação de Benfica N_{SPT}<60)	20	300	40	40	120	0,7	10	34
ZG2- A (Formação de Benfica N_{SPT}>60)	20	355	60	60	180	0,7	15	34
ZG1 (Complexo Vulcânico de Lisboa)	21	500	100	100	300	0,3	20	35

Since two analyses will be performed considering the behavior drained and undrained soil, it was necessary to define the parameters presented in the Table 5.

Table 5 - Parameterization of geotechnical zones ZG3 and ZG2 - undrained parameters.

	γ [kN/m³]	P_{ref} [kPa]	E_{50}^{ref} [MPa]	E_{oed}^{ref} [MPa]	E_{ur}^{ref} [MPa]	m	Su [kPa]
ZG3-B (Landfill)	18	100	15	15	45	0,8	113
ZG2-B (Formação de Benfica Nspt<60)	20	300	40	40	120	0,7	270
ZG2-A (Formação de Benfica Nspt>60)	20	355	60	60	180	0,7	350

8.5. Main calculation phases

Constructive phasing is very important in the definition of the calculation model, so the different phases presented in Table 6 were defined carefully. As the analysis was done in two dimensions, to represent the effect of the third dimension which translates into the arc effect that is mobilized by the constructive phasing of the Reinforced Concrete King Post Wall, by changing the value of Staged M=0.5 for the phases in which the panel is opened.

Table 6 – Main calculation phases adopted in Plaxis 2D software.

Phase 0	Generation of initial stresses considering the land share corresponding to the ZG2B, by gravity loading calculation method.
Phase 1	Construction of the gravity wall;
Phase 2	Activation of landfill layers;
Phase 3	Activation of overloads located in the back of the wall;
Phase 4	Reset displacements to zero so that the deformations obtained so far are not considered in the analysis of the final displacements;
Phase 5	Activation of micropiles;
Phase 6	Excavation of the first level considering $\sum \boldsymbol{M}_{\boldsymbol{stage}} = \mathbf{0,5}$, to consider the three-dimensional effect (arc effect);
Phase 7	Activation of the *Reinforced concrete king post wall* and the first anchorage with a prestress value of 600kN;
Phase 8	Repetition of 7) and 8) until reach the bottom slab;

8.6. Analysis of modeling results

From the analysis of the output of the developed model, it was possible to obtain the displacements that results from the proposed solution in each of the phases mentioned and analyze the evolution of them over the execution time.

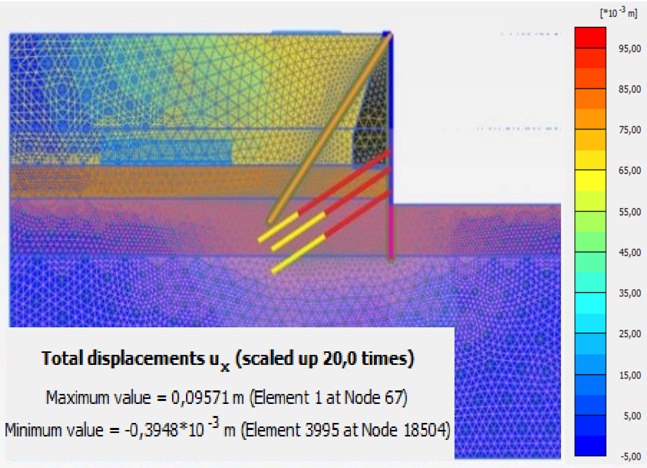

Figure 7 -Total horizontal displacements obtained by modeling the adopted solution (drained behavior) in PLAXIS 2D.

Figure 7 shows the result of horizontal displacements at the end of the excavation (9.5 cm) for the first model that was made. In this analysis, we considered drained behavior from all geotechnical zones, based on the parameters and constructive phases described above.

The area where the largest displacements occur is located at the top of the gravity wall. Through the analysis of the deformed mesh, we saw that there is a rigid body movement in the wall area. In the other hand, the Reinforced concrete king post wall deforms in the less rigid areas, as soon as the excavation begins below its foundation level, i.e. between ground anchors.

The vertical displacements presented in Figure 8, are considered a little high (7.0cm), not only for the settlements but also for the heave. For this fact, in the other analysis was considered a more rigid behavior by adopting the $E_{ur}^{ref} \approx 4E_{50}^{ref}$.

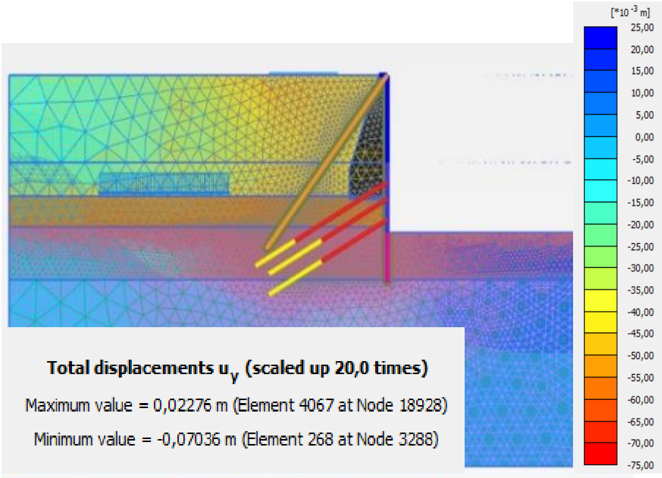

Figure 8 - Total vertical displacements obtained by modeling the adopted solution (drained behavior) in PLAXIS 2D.

8.7. Analysis of the solution considering the undrained behavior of the ZG3 and ZG2 zones

Considering that ZG3 and ZG2 presents an undrained behavior during the execution time of the work, the analysis was performed considering the undrain parameters for these zones. The calculation phases are the same for the situation in which the drained behavior was considered, as well as the parameters of the ZG1 zone and the structural elements.

Adopting this new calculation model, a maximum horizontal displacement of 3.5cm and a maximum vertical displacement of 2.0cm are obtained. The results are substantially lower than those obtained when considering the drained behavior of the soil.

When considered a behavior drained from the ground, it is expected that the shear resistance decreased, with the relief of stresses, which happen when the excavation of the land is carried out. On the other hand, considering an undrained behavior, it is expected that there will be no change in the void index during loading/unloading, maintaining constant cutting resistance throughout the process, so the decrease in displacements compared to the previous model was expected.

8.8. Back analysis of the adopted solution

In order to validate the adopted model, a back analysis was performed based on the results of instrumentation and observation obtained so far to date 24/07/2021. The results of the inclinometer closest to the area in study were analyzed, as well as topographic targets to validate, not only horizontal displacements, but also the settlements of the retaining structure executed and the centenary wall itself. The load cells of the nearby ground anchors were also verified to compare with the results obtained in the PLAXIS 2D program.

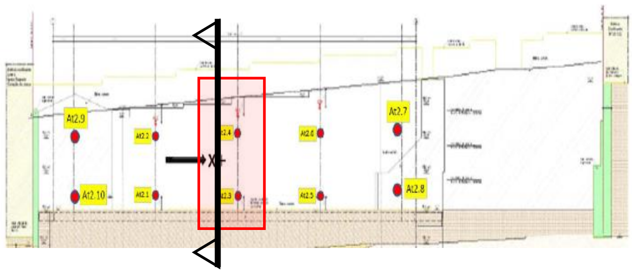

Figure 9 - Implementation of the topographic targets - Zone 2

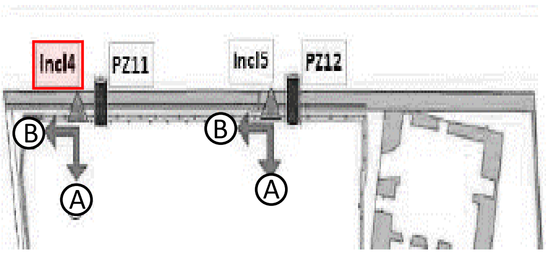

Figure 10 - Location of inclinometers - Zone 2

The differences found between the results observed in the field and the results obtain from the model that was made may be related to several factors: simplifications are assumed in the model; the adopted geometry of the structural elements, as well as in the characterization of their parameters; assumptions of ground parameters based on point tests that can be not representative of the whole zone. In addition, the results of the model are being compared with the results of displacements of topographic targets that are not located at the exact same zones.

With the analysis of the data of the I4 inclinometer, it is noted that the maximum displacement occurs at the top of it, corresponding to a value of 9. 7mm (Figure 11), which is a lower value than the one verified by reading the topographic targets. However, it is again mentioned that they are not located in the same place, just as none of these are exactly at the exactly position of the critical zone that is studied, whereby these values are only a basis for comparison.

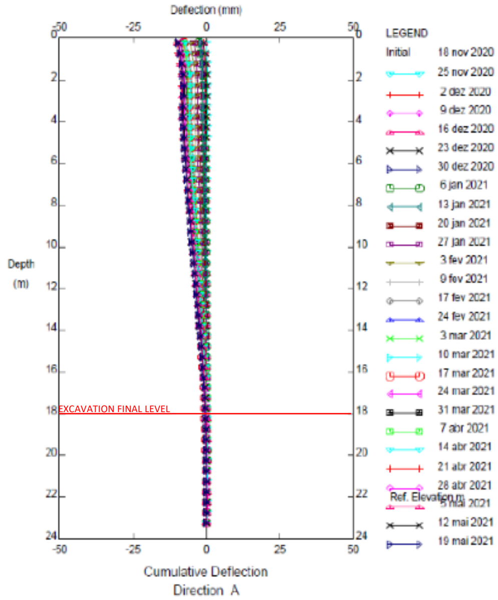

Figure 11 - I4 inclinometer readings.

9. Parametric analysis

9.1. Ground Young Modulus

Given that this parameter increases in depth, the increasement of it was not made in a proportional way for all geotechnical zones. Although it can be unusual, for a landfill, a value of greater than 15MPa. In this case, that was considered due to the centenary character and to the fact that there is a very busy road on top of it, making the landfill well compacted. So, the young modulus has been increased to have a more rigid behavior when stress relieved due to excavation.

Several analyses were performed, changing the deformability module, but only the one containing the final analysis of this parameter is presented.

The parameters adopted in this model are presented in Table 7.

Table 7 – Adopted parameters in the model of the solution, after back analysis of the soil ground young modulus, in the finite element program, Plaxis 2D.

	Su (kPa)	E50 (MPa) (before)	E50 (MPa) (after)	Eur (MPa)
ZG3	113	15	25	100
ZG2-B	290	40	70	280
ZG2-A	350	60	110	440
ZG1	-	100	150	600

The results of this change show that this parameter has a considerable influence on the results of displacements. The horizontal displacement was reduced by approximately 1.1 cm and the vertical displacement by approximately 0.7 cm.

In Figure 12 and Figure 13, it is possible to analyze the evolution of the young modulus adopted in the model, before and after the back analysis performed.

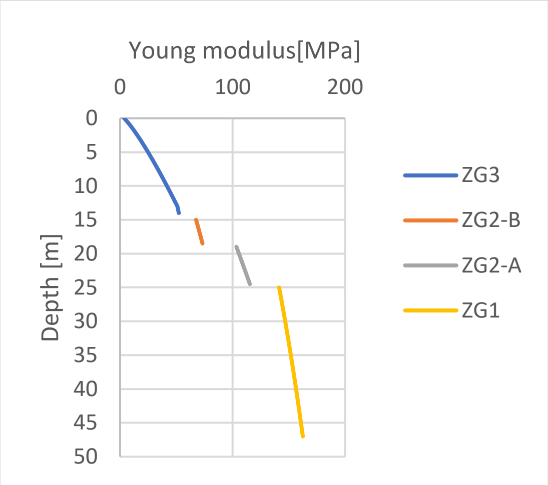

Figure 12 - Evolution of the ground young modulus as a function of depth - before the back analysis performed.

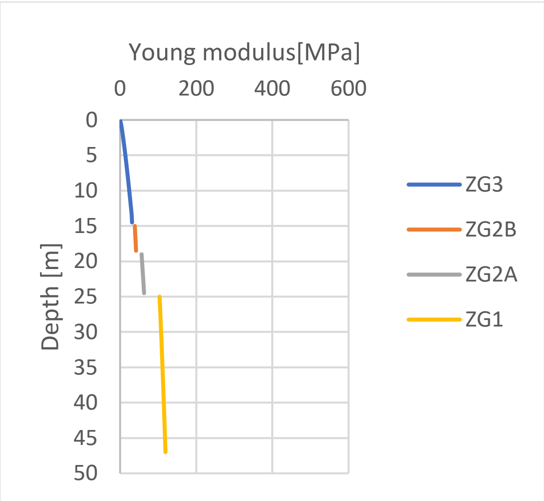

Figure 13 - Evolution of the ground young modulus in depth, after the back analysis performed on this parameter.

9.2. Undrain shear resistance

Since the adopted model for proceeding with back analysis was the model in which is considered an undrained behavior of the geotechnical zones ZG3 and ZG2, and that these layers are the ones that have the most impact on the results of the displacement of the centenary wall in analysis, as well as the retaining wall, it was found that the parameter with the biggest influence on the results, apart from the young modulus, would be the undrained resistance, Su. Similarly, to what happens with the young modulus, the parameters used at the last iteration made for this parameter are presented.

Table 8 – Adopted parameters in the model of the solution, after back analysis of the undrain shear resistance of geotechnical zones ZG3 and ZG2, in the finite element program, Plaxis 2D

	Su (kPa) (before)	Su (kPa) (depois)	E50 (MPa)	Eur (MPa)
ZG3	113	150	15	60
ZG2-B	290	300	40	160
ZG2-A	350	500	60	240
ZG1	-	-	100	400

It should be noted that the analysis performed does not contain the young modulus from the previous study, since it aims to compare the results of the two analyses, to confirm which parameter has the greatest influence on the results.

The results obtained reveal an insignificant decrease in horizontal displacement of 3mm, concluding that the young modulus has a bigger influence on the deformations of the curtain and the gravity wall than the undrain shear resistance, as expected.

9.3. Angle of shearing resistance, young modulus and undrain shear resistance

Finally, after the analysis of the influence of the young modulus and the undrained shear resistance of the soil, it was considered that the resistance parameters representative of the ZG1 geotechnical zone could be optimized.

Table 9 shows the parameters used in the last modeling performed.

Table 9 - Parameters adopted in the model of the solution, after back analysis of the young modulus and the undrained shear resistance, in the finite element program, Plaxis 2D.

	Su (kPa)	E50 [MPa]	Eur [MPa]
ZG3	150	25	100
ZG2-B	300	70	280
ZG2-A	500	110	440
ZG1	Φ=37; c=40	150	600

Comparing the obtained results in the program with the results of the I4 inclinometer, it is observed that the trend is similar, with some differences, especially in the area where are located the ground anchors and the displacement of the retaining wall at the bottom of the excavation. These differences can be justified by several factors as:

i. The inclinometer is not located within the earth retaining wall, so the measured displacements will not be exactly those observed in it.
ii. The parameters that characterize soil behavior as well as the constitutive models adopted are not entirely realistic.

iii. The constructive phases adopted at the site may not have been as defined at the numerical model.
iv. The geometry of the wall can be different from the one adopted in the model.
v. Among others.

The results of this analysis are shown in Figure 14.

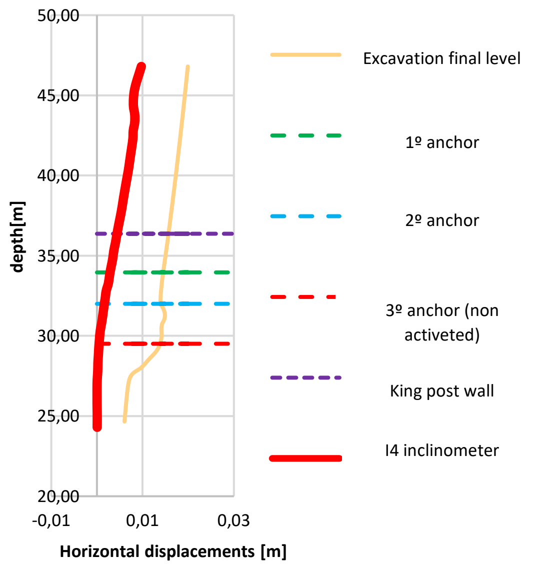

Figure 14 - Comparative analysis between the displacements of the Reinforced Concrete King Post Wall, taken from the model in Plaxis 2D, and the actual displacements of the I4 inclinometer

10. Alternative solutions

By analyzing the instrumentation reports it is possible to verify that the defined solution could have been optimized, since the displacements obtained are much lower than the maximum permissible values.

Thus, it is considered that, within the various alternative solutions that could be considered, there are two considered more relevant.

10.1. Alternative solution 1

To optimize the adopted solution, the following changes were considered:

1. Increased spacing of micropiles on the crown beam executed at the top of the centenary wall to 7m.
2. Reduction of the thickness of the Reinforced concrete king post wall to 0.4m.
3. Increased the size of the primary panels to 3m.
4. Placement of ground anchors only at the primary panels, regardless of the underpinning beam, keep the spacing defined at the adopted solution.
5. Remove the last level of ground anchors, which, in reality, never were executed.

With these changes the numerical analysis results shows that the solution is viable and more economical, saving not only materials but also time.

10.2. Alternative solution 2

As an alternative solution it was considered that it would be a good option to use the top-down system, where part of the structure would be built before excavation was carried out at a minor distance from the centenary wall, so that there is no major disturbance at the centenary wall, and later is executed a solution that consists in a Reinforced Concrete King Post wall, held by shoring, which react against the slab of the floors of the final structure. For a clearer understanding of the solution that is being proposed, it is possible to observe the solution in plan, and a cut section in Figure 15 and Figure 15.

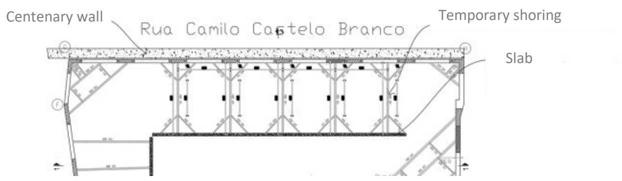

Figure 15 – Plan of proposed solution 2.

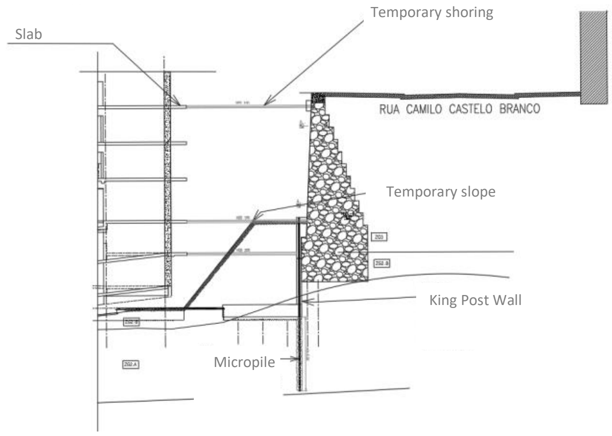

Figure 16 – Cross section of the proposed solution 2.

10.3. Solution structural design

In addition to design the Reinforced concrete king post wall as well as the shoring profiles of the structure, the efforts obtained in the finite element program, Plaxis 2D, were analyzed.

Thus, the values used for the design of the solution were the values taken from the modeling performed, affected by a coefficient of 1.35, since the program does not apply safety factors. The results taken from the model are shown in Figure 17, Figure 18 and Figure 19.

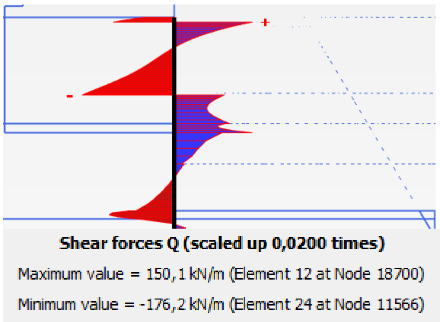

Figure 17 – Shear force in the Reinforced Concrete King Post Wall.

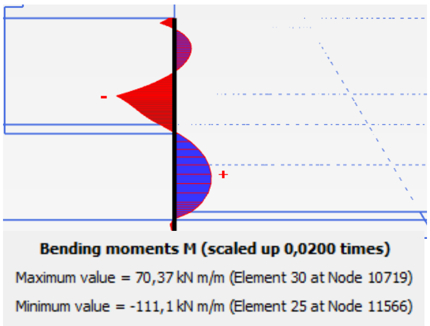

Figure 18 – Bending moment in the Reinforced Concrete King Post Wall.

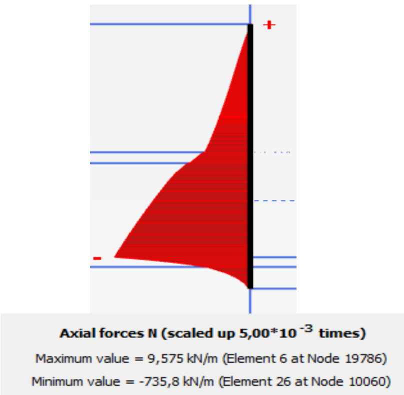

Figure 19 - Axial force in the Reinforced Concrete King Post Wall.

Table 10 - Summary of the design efforts of Reinforced Concrete King Post Wall_ Alternative Solution 2

Design actions	
Bending moment (-) [kNm]	150
Bending moment (+) [kNm]	95
Shear force [kN]	238
Axial force [kN]	993

10.4. Safety verifications

To verify the retaining structure safety, the following checks have been carried out.

Reinforced Concrete King Post Wall:

- Bending moment resistance.
- Shear resistance.
- Punching resistance.

Micropiles:

- Structural and buckling resistance.
- Ground bearing capacity.

Temporary ground anchors:

- Structural and buckling resistance.
- Ground bearing capacity.

11. Solutions comparison

After the analysis of the solutions in the finite element program it was possible to verify that the proposed alternative solutions would also be a possible solution to implement, leading to satisfactory results in terms of maximum displacements achieved.

It is possible to see in Figure 20 and in Figure 21 the displacements obtained at the end of the excavation for the alternative solution 1 and alternative solution 2, respectively.

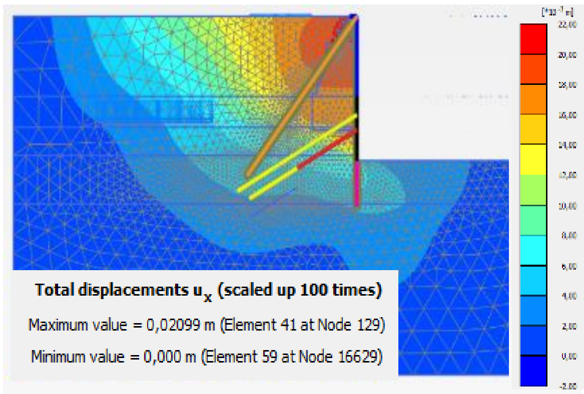

Figure 20 – Horizontal displacements - alternative solution 1.

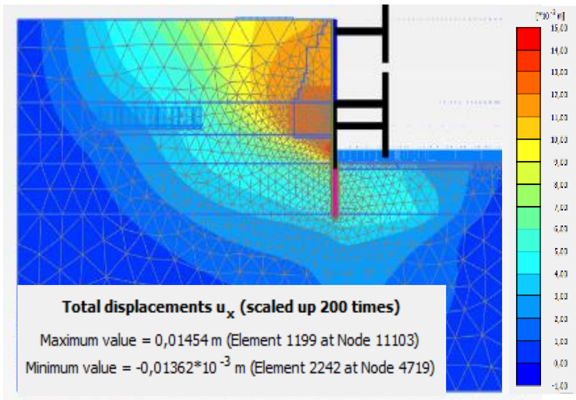

Figure 21 - Horizontal displacements - alternative solution 2

Note that the rigid body movement of the wall is different from solution 1 to solution 2. The solution with the active ground anchors - solution 1 - there is a greater control of the displacements at the base of the centenary wall, because there is a load that is being applied against the ground, restricting the tendency of slipping into the site direction when the ground stresses are relieved due to excavation.

In adiction, an economic analysis will be carried out in a simplified way between the solution and the alternative solution 2. The comparison between the solution adopted and the alternative solution 1 will not be addressed, since, in the case of an optimization, in which part of the bracing elements are removed, it is certain that there will be a reduction in costs.

With this analysis, it is concluded that the cost reduction, from the adoption of the alternative solution 2, is 46 174.65€, corresponding to 42% of the cost of the solution that was adopted. In fact, the reduction would be slightly lower since the cost of excavation was not considered in this analysis.

In fact, the excavation process of the solution 2 will have a higher associated cost due to the conditioning caused by the shoring of the retaining structure, which make it difficult to perform the work. Nevertheless, the solution proves to be more economical.

12. Conclusions

After the end of this work, it is concluded that the defined objectives were achieved. The main objective was the analysis of the case study of a retaining wall structure in a challenging scenario.

To evaluate the stability of the solution adopted, a model of the solution was made in a finite element program, Plaxis 2D. From this analysis it is concluded that to make this type of projects in an economical and safe way, it is quite important to have a critical thinking in the definition of the parameters of the model, assuming some knowledge from the geotechnical point of view. It should be noted that by adopting the parameterization defined in the geological-geotechnical report the displacements obtained for the modeling of the same solution are substantially higher, not corresponding to reality.

This reinforces the need of a good characterization of the ground, based on an effective geotechnical and geological investigation, field tests and laboratory tests to collect as much information as possible that leads us to a better understanding of what behavior can be expected from the soil.

However, despite the critical thinking and geotechnical knowledge of the designer, it is indispensable a good plan of instrumentation and observation, since the results obtained in the modeling will hardly correspond exactly to reality, being necessary to control the displacements and to react in time when unforeseen situations are observed, or even to the optimization of the solution, when the results are better than the expected.

The solution adopted could have been optimized if the parameterization of the adopted model was not overly conservative, however, in such delicate cases as the present case study, it is not possible to take too positive an attitude, but the proposed solution can be optimized later, in the construction phase.

The proposed alternative solutions aim at reducing the cost of execution, not significantly changing the behavior of the structure, ensuring verification to the USL and SLS. However, it is understood that in the project phase the alternative solution 2 was too optimistic because it has a passive performance, being only activated with the movement of the centenary wall. This could, however, be duly compensated by increasing the stiffness of the temporary steel struts.

13. References

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