Slope Stabilization Solutions in Urban Area

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Abstract

Within the theme of slope stability, it is presented the analysis of a case study, associated to a instability phenomenon referring to the slope where Miradouro de São Pedro de Alcântara develops, identified through the observation of various pathologies, both at its platforms and in the retaining walls that support them.

The work developed aimed primarily at clarifying the rupture process and the phenomena involved in its initiation, as well as the establishment and evaluation of the current reference situation, using slope stability analysis programs for that. Based on the developed analysis, it was concluded the need for structural reinforcement of the site, as soon as possible, in order to stabilize the movement of the soil and, consequently, the support structures. In order to do this, and considering the inherent conditioning to the context in which is inserted, stabilization solutions have been defined to restore safety. For the dimensioning and validation, Plaxis 2D program was used, allowing the numerical modeling in finite elements of the constructive phase and stability analysis of the constructive elements. The validation of the solutions also includes a comparative analysis between the presented stabilization alternatives. Based on the obtained results, it can be concluded that the solution that contemplates the decrease of the unstable soil mass, compared to one that only implies the increase of its shear strength, has a superior beneficial effect.

Keywords: slope stability, limit equilibrium, finite element, stabilizing solutions, numeric modeling

1. INTRODUCTION

The increasing population and the development of society have led, over the times, to the demand of new and alternative constructive solutions, in order to promote the exploitation of urban space, which often implies the need to build in more adverse areas. Therefore, the use of retaining walls is assumed as important in the geotechnical context, once it allows to contain the ground and so to build structures below the ground surface. Attending the natural interaction between these structures and the underground, it is required not only concerns about the design of the structure, and its own stability, but also relating to the site. In order to do so, an evaluation of the geological and geotechnical characteristics, and the influence of external agents must be done. Moreover, the possibility of adaptation to new equilibrium conditions is also important to take in account [1].

It should be noted that the importance of the preview considerations increases when the construction develops in areas at risk, such as slopes, which already have a relative susceptibility to instability. As such, the slope stability analysis is a cross-cutting and very important issue in the geotechnical field, regarding the severe consequences that slope ruptures encloses, in terms of damages and losses, particularly in urban context.

Following this theme, this document presents the analysis of a case study, referring to the initiation of a slope instability phenomenon, which induced the movement of the retaining walls that sustain part of the unstable soil mass.

Facing this, it was decided to perform a detailed analysis of the local, in order to understand the problem, which carried out geotechnical prospecting and the implementation of some instrumentation throughout the critical area.

Based on the analysis of the instrumentation results, the developed works aimed to clarify the instability process and the factors involved in its initiation, and to evaluate the current safety factor. Besides, in order to help the slope evaluation performance and the perception of instability phenomenon, as to establish the reference situation, by the definition of soil parameters in line with the identified pathologies, it was proceeded the numerical modeling of the case study, considering two methods commonly used in practice for the evaluation of slope stability, the Limit Equilibrium Method (LEM) and the Finite Element Method (FEM). The first, by the program GeoStudio-Slope/W, which is based on the method of limit equilibrium, and the second by the finite element method through the Plaxis 2D program, which allows an evaluation of the stress and strain experienced by the ground.

Confronting to the results, it was preconized the need of a stabilization solution, as soon as possible. Therefore, this works also includes the presentation and design of two solutions to stabilize the site, which definition regarded the associated constraints. For the design and validation of this solution, the finite elements program, Plaxis 2D was used, allowing the numerical modeling of the constructive phase, and consequently to find the stresses and displacements related to the respective constructive element. It also allows the stability analysis of the site, associated with the solution.

Finally, the validation of the presented solutions also contemplates a comparative analysis between them, which takes into account technical and economic consideration.

2. CASE STUDY

This case study is related to the stability analysis of the slope where the Miradouro de São Pedro de Alcântara (Figure 2.1) sits, near Bairro Alto, Lisboa. This site is one of the most emblematic viewpoints in Lisbon, from which can be seen part of the downtown area of Lisbon.

Figure 1 - Case study plant view

Basically, Miradouro's belonging area is shaped on two levels (platforms), both sustained by masonry retaining walls with relative high thickness (about 3 to 5m).

Although, the beautiful view of the city contrasts with the anomalies that have been detected over the last years. Since 2006, several pathologies, like settlements (Figure 3), were reported on the Miradouro's platforms. At the same time, some cracks were identified on its retaining walls

(Figure 2 and Figure 3), which seems to indicate a scenario of global instability of the slope, that includes the Miradouro's implementation area.

Figure 2 - Cracks in upper retaining wall (R_{sup})

Figure 3 - Settlements on lower platform

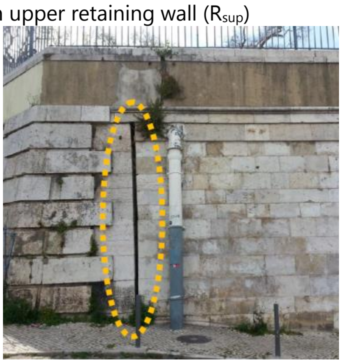

Figure 4 - Fissure in lower retaining wall

2.1. Geologic and Geotechnical Scenarios

To establish the geological and geotechnical characteristics of the site, some auxiliary tasks were carried out: drilling of five mechanical boreholes, followed by Dynamic Penetration tests (SPT), two of them located in the upper platform, near the upper retaining wall, and another three in the lower level.

Considering the reports provided about the slope recognition process, this site develops over the geological marine Miocene formation, usually referred to as "Areólas da Estefânia" (M1II), which are overlapped by "Argilas e Calcários dos Prazeres" (M1I), both attributed to the Aquitanian (lower Miocene). Therefore, according to the results obtain in site survey, reproduced in [2], from the surface it was intersected a landfill to a variable depth of 10.4 to 16.4m, near the two opposite corners of the site, that basically consists in clayey sands, with lithic fragments, bricks and organic materials, with NSPT values between 2 and 25. Below this superficial horizon, it was detected the occurrence of a sequence of strata, corresponding to the Miocene formations. The two geological profiles defined trough this survey are shown in Figure 2.

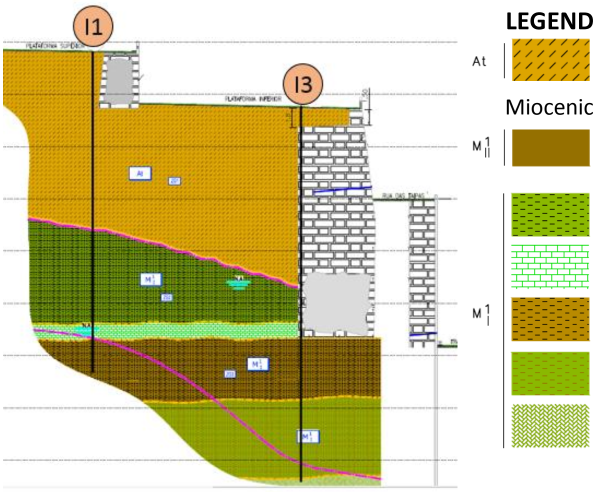

Figure 5 - Geological profile type with inclinometer's location

Considering the obtained SPT values, geotechnical zoning was defined into four distinct zones (GZ), as presented in Table 1, which demonstrates the different mechanical characteristics of the geological units referred. For each zone, to define and quantify the mechanical characteristics, some empirical formulas, that correlate SPT N values and liquidity index with soil strength parameters, like friction angle, φ'.

Table 1 - Estimated soil strength parameters for each geotechnical zone

Zona	Elevation (m) S1	Elevation (m) S3	γ_{unsat} (kN/m³)	Φ' (°)	c' (kPa)	E' (MPa)
ZG1	até 51,4	-	17	28	0-10	5-15
ZG2-1	51,4 a 43	-	18	34	20-60	30-60
ZG2-2	43 a 39,40	34,1 a 28,30		32		
ZG3	under 39,40	under 28,30	19	40	60-100	60-100

γ_{unsat} – undrained volumic weight; Φ' (°) – friction angle; c' – cohesion intercept; E' – drained Young's modulus

2.2. Instrumentation plan

The monitoring campaigns associated with the case study are due to the evolution of pathologies once identified. Thus, a geodetic observation system was implemented in 2009, in order to measure possible displacements in the lower retaining wall.

Based on the information collected over three years, and expressed in [3], it was checked that the instrumented cracks appear to function as an expansion joints that adapt throughout the years. Although the displacements are small and have a reduced rate of evolution, a movement trend was recognized.

In addition, during the Geological-Geotechnical study, other observation equipment was introduced, namely inclinometers and piezometers, in order to detect deep ground movements.

Following the obtained results, it was possible to observe the movement resumption related to Miradouro's site, either at the surface, expressed in its retaining walls, moving towards the Rua das Taipas, as so as in depth, with clear evidence expressed in the inclinometer readings, as can be seen in Figure 6.

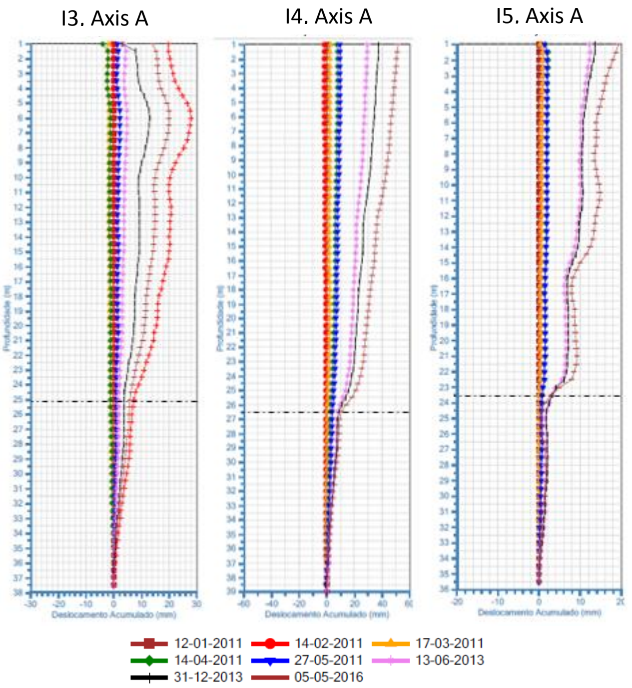

Figure 6 -Inclinometers displacements

Taking into account the three graphs, shown in the figure above, which represents the soil's total displacements, a similar pattern can be observed. This pattern develops approximately 25, 26 and 24m depth, respectively for inclinometer I3, I4 and I5 (indicated in the corresponding figures) which is understood as a slip surface that occurs a few meters below the lower retaining wall foundation, but still extends up to substantially 30 to 33m deep.

The previews results led to conclude the instability phenomenon as a consequence of the overall stability loss from the slope where the Miradouro's develops.

2.3. Causes

Regarding the causes to this phenomenon, some research was carried out, in order to identify the process that changed the balance of the site. From the analysis, it was possible to clarify the evidence of a highly waterproofed area, associated with surface water runoff and infiltration in less waterproofed areas, such as Miradouro's platforms. In addition, the construction of several structures along the slope has been recorded over the years, which constitute barriers to the water flow inside the geological device, and so it may be the origin of an increase in the level of the water table. It should be noted that water plays a very important role in the geotechnical context, since its effects manifest at several levels. This phenomenon is due to an increase in the degree of saturation of the soils, thus a decrease in suction and, consequently, in the effective stresses, resulting in a reduction in the resistance to the cutting of the soil and the existence of soil puffs; By increasing the specific weight of the material, then increasing the unstable Weight; Or by percolation forces, which create hydraulic gradients within the soil and promote internal erosion.

The set of information provided by the instruments seems to prove the slope's instability, with the recommendation of a sliding surface that develops mainly below the lower retaining wall's foundation, which indicates the existence of an imminent global rupture mechanism between the upper platform and Travessa do Fala Só. It is presumed that this phenomenon justifies the movements of the strata inspected, as well as the pathologies recorded in the retaining structures.

2.3. Numerical analysis of reference situation

Facing the phenomena nature, explained above, it was decided to take advantage of the numerical modeling valence to analyze the case study, particularly evaluate the global stability associated with the slope where Miradouro sits. Initially, this analysis aimed to establish a reference situation, consistent with the advocated phenomenon, which presents itself as a basis to the definition and analysis of some solutions that intent to restore the safety of the slope.

The slope security verification to global ultimate limit state, as a result of actions undertaken, was carried out by taking into account the philosophy present in EC7 (2), considering the AC1-Comb.2.

- <u>Slope/W</u>

As a starting point, the GeoStudio: Slope / W (2012) program, based on the limit equilibrium theory, was used, in order to obtain the critical slip surface, corresponding to the lowest safety factor. The geological model analysis take in account the 4 geotechnical zones, the position of phreatic level according to the piezometers readings, and considers the previews estimated soil strength parameters, as exposed in the Table 1.

In order to determine the most probable breaking mode associated with the lowest safety factor, the option "Grid and Radius" was initially chosen, which allows to obtain slip surfaces with approximately circular shape. To do this, a mesh of centers is defined for the potential slip surfaces and, for each center, a set of tangent lines, corresponding to surface radii, in an area considered sufficient to cover the entire geological landscape. In this case, 40x40 increments were established for the mesh of centers, and increments of 40 for the tangent line mesh, whose geometry was invariable throughout the analyzes.

Through the analysis of the established reference model, it was perceived that the rupture surface had an approximately planar shape, and considering the estimated strength parameters, it was obtained a safety factor (SF) of approximately 1.34.

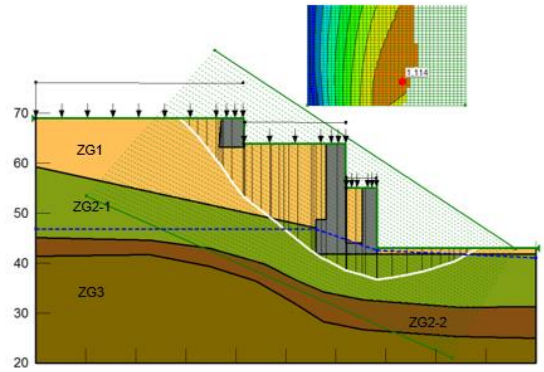

Figure 7 - Geological model adopted in Slope/W analysis, with preliminary critical slope surface representation (SF=1.336), in Slope/W program

Since the safety factor was still high, namely far from a failure mechanism (characterized by a SF near 1), a sensitivity analysis was carried out in order to calibrate the soil strength parameters.

The sensitivity analysis is done in an iterative way, by changing the numerical values of the parameter of interest, within a set interval, while the others remain fixed in the initial values. In this case, the evaluation of the sensitivity to the alteration of the geotechnical parameters considered in the model developed in relation to the reference scenario, represented in Figure 4 and characterized by the values expressed in Table 1. From this scenario, the values of each parameter were individually modified in a series of iterations, and for each one the safety factor was recorded. Each iteration corresponds to the change in the value of a parameter, within a range considered, and assumed as plausible. The results of the sensibility analysis are shown in Figure 8.

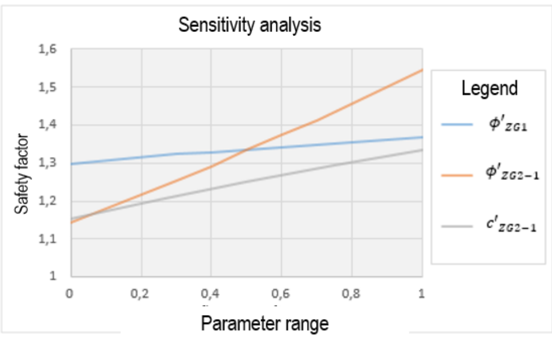

Figure 8 –Sensitivity analysis to strength parameters

From the sensitivity analysis carried out, the resistance parameter of ZG1, namely the friction angle, φ 'ZG1, shows a reduced influence on the change in the safety factor, as well as on the detected critical break surface, which is maintained Unchanged from the base scenario. In fact, the value of the angle of friction is in agreement with the type of material constituting ZG1 and, assuming a zero cohesion, the resistance of the layer is relatively small, having little influence on the instability phenomenon. As regards ZG2-1, and compared to ZG1, there is a greater influence on the overall stability of the slope, since the variation of the corresponding resistance parameters, effective cohesion, c 'ZG2-1, and above all φ' ZG2 -1, have a visible effect on the change in the safety factor, clearly expressed in the steeper slope of the corresponding curves, as shown in Figure 8.

Based on this information, a serial of new iterations took place, so it could be possible to establish a scenario with a SF near 1, and so in accordance with the instability phenomenon. The strength parameters obtained through the analysis are shown in Table 2, which corresponds to a safety factor of 1.114, and has a slip surface like the one represented in Figure 7.

Table 2 – Optimized soil strength parameters

Parameter	Geological zoning			
Zone	ZG1	ZG2-1	ZG2-2	ZG3
γ (kN/m³)	17	18	18	19
φ' (º)	27	31	33	35
c' (kN/m²)	-	10	40	60

- <u>Plaxis 2D</u>

In order to overcome the major limitations of the limit equilibrium methodology, and to ascertain the calibration of the geological model adopted for the case study, it was used a software based on the finite element method, namely Plaxis 2D. This program was developed with the purpose of conducting stress and deformation analysis, allowing the understanding and monitoring of soil's behavior, for a given geological scenario, as well as the interactions between rheological materials and other structures. Besides, this methodology are still a very useful tool in order to help the conceptualization of geotechnical solutions, as so as to model construction phases related to those solutions.

Therefore, the calculation model geometry adopted for the numerical analysis in Plaxis is in accordance with the defined one in Slope /W, as can be seen in Figure 9.

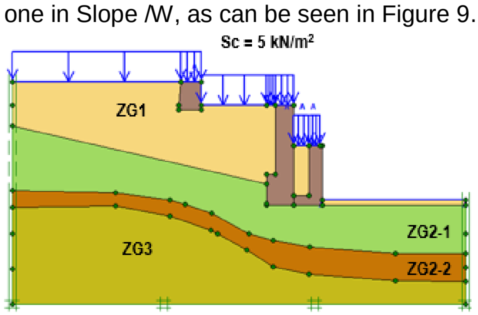

Figure 9 - Geological model adopted to the analysis in Plaxis 2D

The behavior of geotechnical zones considered were simulated by using the Hardening Soil constitutive model for the soil layers, and for the masonry walls the Linear Elastic constitutive model, based on Hooke's law, since both models are appropriate for simulating the soil response. In Table 3 the characteristic parameters for each layer and model considered are presented.

Table 3 - Characteristic parameters to soil and masonry wall for numerical modelling in Plaxis 2D

Parameter			Geological zone			Masonry
Geotechinal zone		ZG1	ZG2-1	ZG2-2	ZG3	
Material type		Drenado				-
γ (kN/m³)		17	18	18	20	22
Strength parameters	Φ' (º)	27	31	33	35	-
	c' (kN/m²)	0	10	40	60	-
Stiffness parameters	E_{50}^{ref} (kN/m²)	5000	42000	42000	115000	2000000
	E_{ced}^{ref} (kN/m²)	5000	42000	42000	115000	-
	E_{ur}^{ref} (kN/m²)	15000	126000	126000	345000	-
	m (-)		0,5			
Advanced parameters	v_{un} (-) (1)	0,2 (*)				
	p_{ref} (kPa)	100				-
	R_f		0,9 (*)			

In order to carry out the stability analysis of the reference situation, three calculation phases were established, consisting basically on the generation of soil stresses, introduction of loads at the platform level, and finally the calculation of the safety factor associated to the established scenario, through Phi/C reduction procedure, where the resistance parameters Φ' e c' are successively reduced until soil rupture occurs. Note that given the complexity of the geometry and slopes problem, it was not carry out the generation of the initial tensions by K0 procedure, as this is commonly applied to model surfaces with horizontal geometries, and so it was considered an additional calculation phase called Gravity Loading.

The results of the stability analysis in Plaxis 2D are represented in Figure 10, where can be seen the rupture envelop (darker colors), which associates with a SF of 1.09.

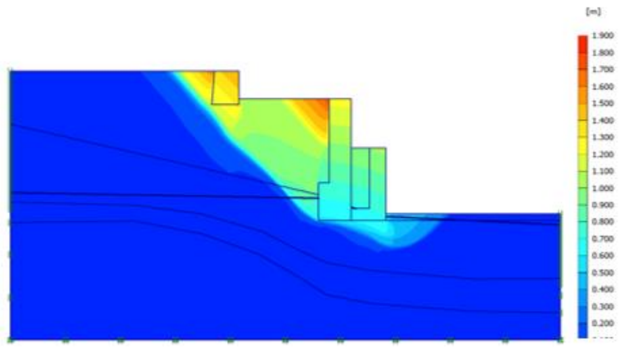

Figure 10 - Critical rupture envelop, obtained in Plaxis 2D

With respect to the comparison of the numerical analysis methods used, confronting the configuration of the critical slip surface obtained in the two methods, which are represented in Figure 7 and Figure 10, it is possible to see the significant similarity between both, interested only in the layer of landfills, ZG1, and ZG2-1, which presents an approximately planar geometry behind the lower retaining wall, and which passes just below the foundation of this structures.

3. STABILIZATION SOLUTIONS

Attending to the nature of the identified phenomena, it was deduced that the existing retaining walls are settled on the unstable soil weight, hence a solution which only contemplates the reinforcement of these structures will be clearly insufficient to solve the problem. Thus, the solutions were defined taking in account the surrounding area, and particularly the soil layers above the platforms and walls, in order to reestablish the safety conditions of the geological device and, therefore, of the existing structures.

3.1. Constraints

Whereas the geological and geotechnical constraints, the Miradouro platforms are developed under a superficial landfill layer, and below are a series of clayey strata that are successively more resistant as depth increases. Nonetheless, deterioration phenomena in the geological device seems to occur immediately below the lower retaining wall foundation, induced by the increase in the water table level, which should justify the global instability scenario. In view of this, the stabilization solution should mainly function in order to increase the shear strength of the strata above, seaming the likely slip surface. It should be noted again that the movements detected by the inclinometers have some expression up to high depths. Therefore, the solution had to be extended to the competent stratum corresponding to zone ZG3 of the geotechnical zoning proposed before.

Besides, in this case study, the unstable area are located on a very particular local, since the Miradouro's area is an integral part of the historical and architectural heritage of Lisbon. In this sense, the recommended solutions had to try as much as possible to maintain the integrity of the elements presented in it, as well as be "hidden or harmoniously integrated in the landscape scenery of the place. Special attention should be directed to the existing fountain in the upper platform and to the fountain located on the lower one.

In terms of solution executability, especial attention was also paid to the topographical framework and to existing accesses, once the slope presents a big declivity. Moreover, once it is situated in an old part of the city, the access to the site is very conditioned, both in terms of road traffic as well as the roads width. The surrounding area is also very small, so there should always be a need for temporary occupation of the public highway, obliging temporary cutting of some of the existing access roads, to mobilize materials and equipment needed for the work. Therefore, the solution adopted should seek to solve the identified problem and at the same time ensure the minimum interference in the neighborhood.

3.2. Proposed solution: Stabilization solution using concrete pile walls and light weight aggregates - Sol.1

In view of [4], the objective underlying the presented proposals was the reduction the deformations observed in the retaining walls and identified in the geological device, throughout the reinforcement of the geological substrate and consequently increase the overall slope safety stability.

The definition of solutions had always assumed a decrease in the unstable weight, with a consequent increase in the conditions of global slope stability, obtained from the excavation of the landfills that are developed in the upper slope area, which are part of the Miradouro. In order to make the excavation possible, it was proposed to materialize additional retaining structures, namely piles curtains, reinforced with piles buttresses. In the design of these peripheral retaining structures, it was considered a minimum length that ensures the embedding in a competent stratum and positioned above the identified slip surface.

In the technical context, the retaining structures develops in the upper and lower platform, and consists on concrete piles, Ø800//1,0m e Ø600//1,0m, as shown in Figure 11 and Figure 12. These structures allow the excavation approximately of 13 meters height, corresponding to about 19300m^3 in the upper platform and 19900m^3 in the lower one. Then, the excavated area is refilled with light weight aggregates.

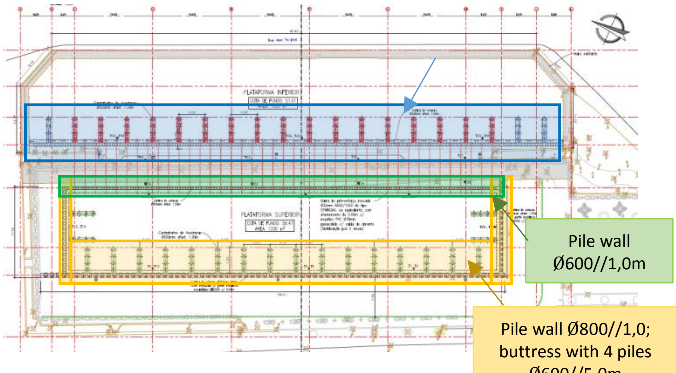

Figure 11 - Solution 1: Pla

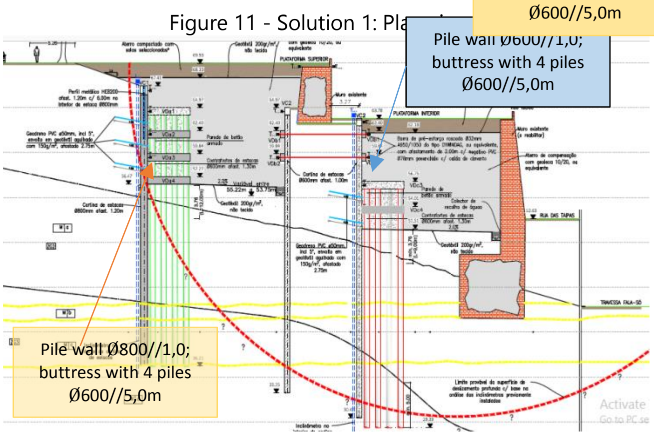

Figure 12 - Solution 1: Cross section

According to [4], the excavation only include part of the superficial landfill layer, both in the lower and upper platforms. This is made possible, as mentioned, by reinforced concrete piles walls, whose function are, at the same time, to intercept the slip surface, which consequently results in increased shear strength at the points of intersection, and to allow the decrease of unstable weight, and so, not only increasing the overall slope safety factor, but also reducing the surcharge on the masonry walls.

3.3. Alternative solutions

Next, two new solutions were presented, with the purpose of providing alternatives to Sol.1, in technical and economic terms. The description and characterization of the solutions under consideration, which are then set, were based on all the constraints already mentioned. In addition, there was a concern to define flexible solutions, in order to provide the use of the excavation area, in a second phase, if that is desirable.

3.3.1. Stabilization solution using Cutter Soil Mixing (CSM) panels and light weight aggregates – Sol.2

The first alternative solution aimed as well to reduce the unstable weight and, consequently, the surcharge over the masonry retaining walls. In this sense, solution Sol.2 is presented in order to promote an alternative to the retaining structure in the perimeter of the excavation, using the panels of cutter soil mixing (CSM), reinforced with steel beams, based on deep soil mixing technology.

Regarding the retaining wall, soil-cement panels were developed using CSM technology in rectangular modules of 2.8x0.60m^2, disposed at the excavation perimeter, with a minimum overlap of 0.20m. In order to improve the response to bending and cutting requests, the panels were assembled with HEB200 steel beams, spaced 1.3m. Due to the high excavation height of approximately 13.5m, and the consequent need to control the wall's deformation it was necessary to increase the flexural stiffness of the structure, trough buttresses. These elements, executed with the same technique and with the same dimensions of the panels, were arranged perpendicular to the alignment of the wall, inside the excavation area, spaced 5m, and reinforced with a HEB300. The link between the wall and the buttresses is achieved by a header concrete beam, with a height of 0.80 m and width of 0.6 m, and three levels of distribution beams, with a height of 0.6m and width of 0.4m.

The schematic function model of the retaining structure is represented in Figure 13. The plan view and the cross section associated with this solution are shown in Figure 14 and Figure 15.

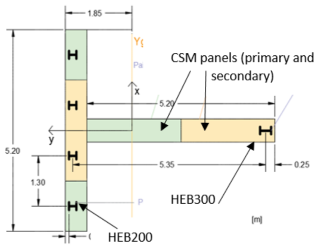

Figure 13 - Schematic function model to CSM retaining walls

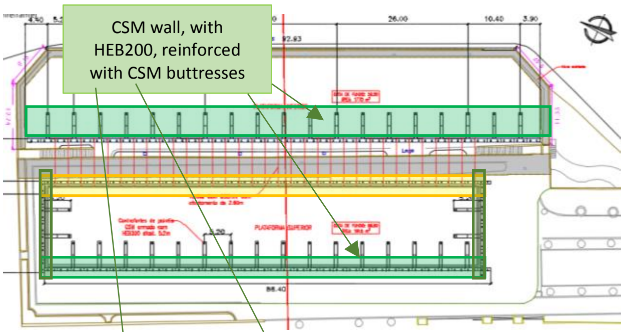

Figure 14 - Solution 2: Plan view

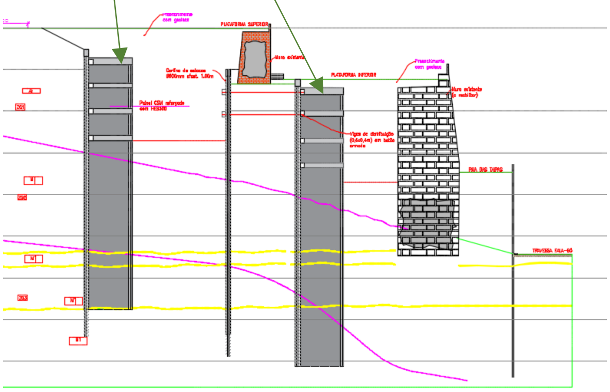

Figure 15 - Solution 2: Cross section

3.4. Second alternative solution – Sol.3

The second alternative solution aimed to minimize the volume of soil excavation, in order to reduce the impact of the solution in the surrounding area. As explained before, the access to the site is very conditioned, since the Miradouro is located in an historical area, characterized by narrow streets, and elevated slopes. In these circumstances, the lorries movement to transport material to the dump will entail considerable interference in adjacent areas. Therefore, this solution aims to overcome this inconvenience, considering to that only the soil reinforcement, through the use of pile curtains extended to the competent stratum.

Thus, the proposed solution involves the execution of two main concrete pile walls, Ø1200 // 1.0m, disposed longitudinally on the lower platform, adjacent to the lower, and upper, masonry wall, spaced approximately 7,5m between axis. Transversely, are disposed alignments of intermediate piles, positioned as buttresses, Ø1000, spaced 3m in the longitudinal direction, in order to increase the rigidity of the structure and the solidarity of all elements, achieved through the materialization of concrete crown beams. Its area of implementation comprises much of the area corresponding to the lower platform of the Miradouro. The base height of the stakes was defined in such a way that they were embedded in the competent strata of at least 5m, thus with a length equal to 34m.

The constructive details and drawings concerning the solution can be found in Figure 16 and Figure 17.

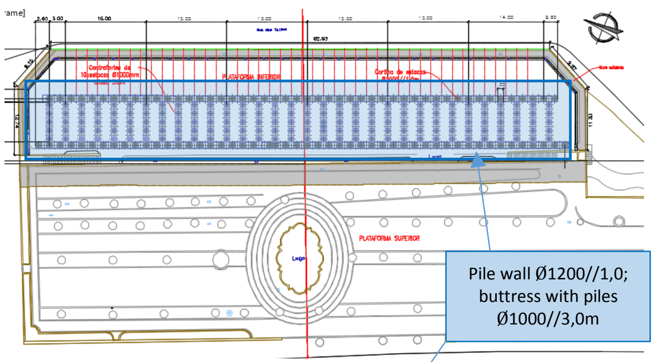

Figure 16 - Solution 3: Plan view

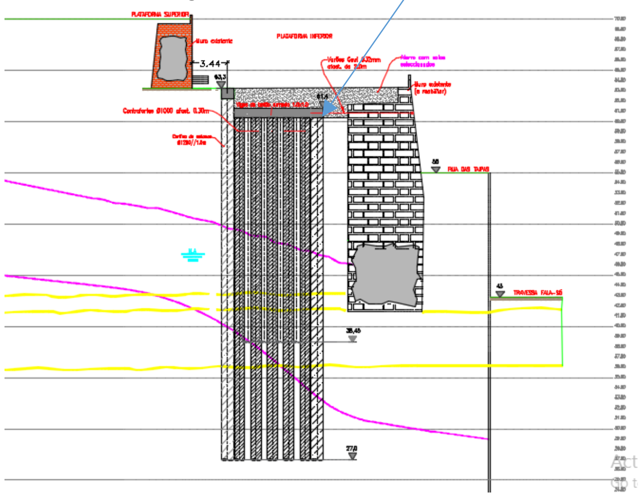

Figure 17 - Solution 3: Cross section

4. NUMERICAL ANALYSIS FOR STABILIZATION SOLUTIONS

A numerical analysis of the three solutions was made, using only the automatic calculation program Plaxis 2D. To accomplish that it was considered in the basis the geological and geotechnical model described before, and the strength parameters corresponding to the reference situation. The goal was to obtain the displacements and efforts of the structural elements and to evaluate the overall safety in the end of the construction process.

4.1. Solution 1

The adopted model to solution 1 is presented in Figure 18, and it's composed by the piles curtains, modeled as *Plate* element, and the distribution beams, which were modeled as a *Node-to-Node Anchor* element.

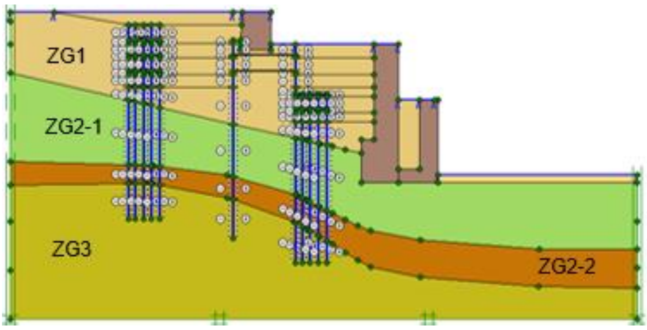

Figure 18 - Solution 1: Numerical analysis model

The constructive sequence includes the soil excavation stages, the execution of the structural elements and finally the refilling with light weight aggregates. The deformed mesh, related to the final of the constructive process is shown in Figure 19. The maximum displacement obtained is about 67mm, related to the top of the upper pile wall.

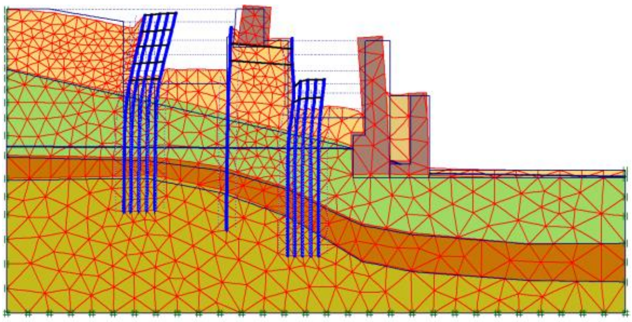

Figure 19 – Solution 1: Mesh deformation at the end of the excavation process (scaled 100 times)

After the refill with the light weight aggregates, the safety factor obtained was 1.63, which corresponds to the rupture envelope of the Figure 20.

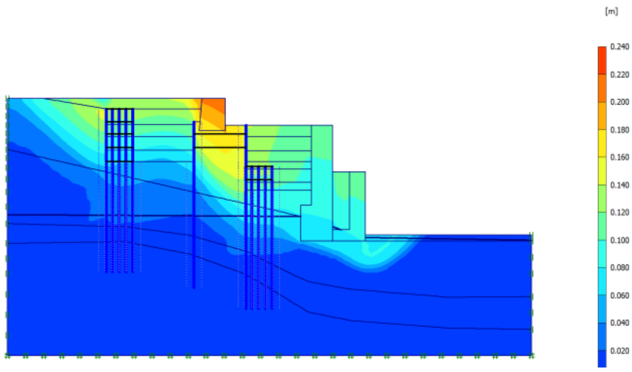

Figure 20 - Solution 1: Rupture envelope after the refill with light weight aggregates (SF=1.63)

4.2. Solution 2

The adopted model to solution 2 is the same one presented to Sol.1, in Figure 18, and it's composed by the piles curtains, modeled as Plate element. The constructive sequence is identical to the previews solution, including the soil excavation stages, the execution of the structural elements and finally the refilling with light weight aggregates. The deformed mesh, related to the final of the constructive process is shown in Figure 21. The maximum displacement obtained is about 47mm, related to the top of the upper pile wall.

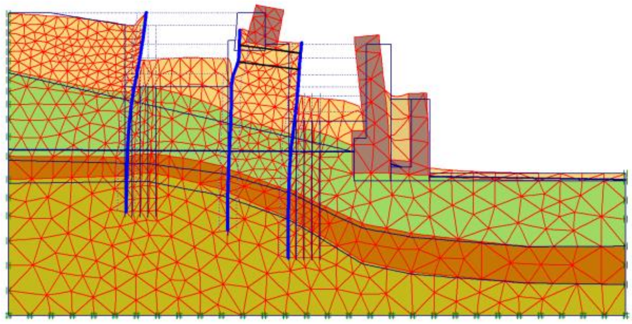

Figure 21 - Solution 2: Mesh deformation at the end of the excavation process (scaled 100 times)

After the refill with the light weight aggregates, the safety factor obtained was 1.55, which corresponds to the rupture envelope of the Figure 22.

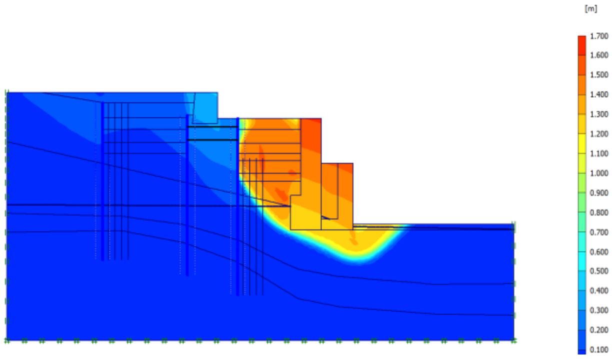

Figure 22 – Solution 2: Rupture envelope after the refill with light weight aggregates (SF=1.55)

4.3. Solution 3

Differently from the solutions presented before, this solution only develops on the lower platform, as can be seen in Figure 23. The constructive sequence of this solution is very simple, comprising only one excavation up to the height of the working level, and the execution of the pile curtains and crown beams.

Figure 23 – Solution 3: Numerical analysis model

The deformed mesh, related to the final of the constructive process is shown in Figure 23. The maximum displacement obtained is about 10mm, related to soil settlement on the lower platform level. The rupture envelope to this solution is represented in Figure 24, to which corresponds a safety factor near 1.4.

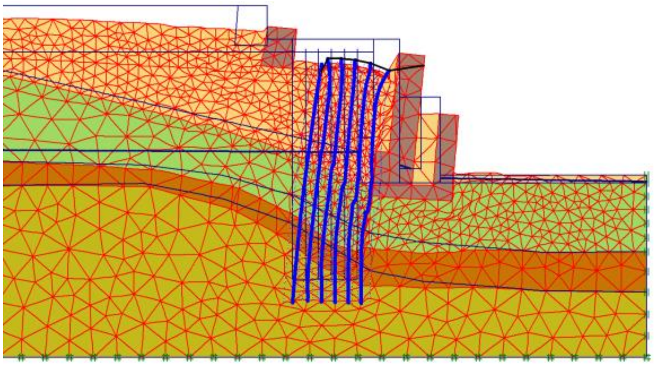

Figure 24 - Solution 3: Mesh deformation at the end of the excavation process (scaled 100 times)

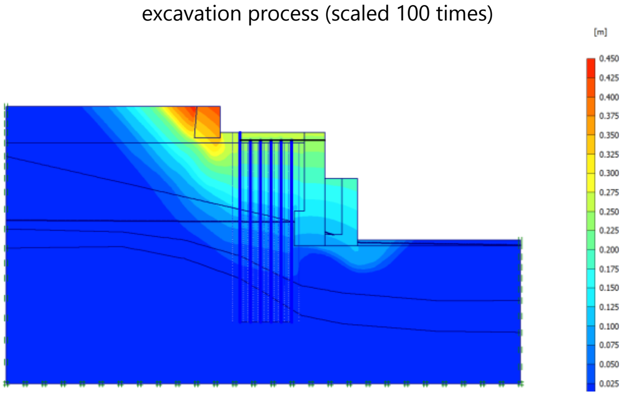

Figure 25 - Solution 2: Rupture envelope after the refill with light weight aggregates (SF=1.39)

5. TECHNICAL AND ECONOMICAL COMPARISON

- **General considerations**

As previously mentioned, the solution using the CSM panels is configured to respond as an alternative method of retaining structure, despite being able to meet the objectives expressed in the base solution, i.e., also enable the reduction of the unstable weight. In theoretical terms, the application of soil treatment technologies, as the one used in Sol.2, with the execution of soil-cement elements, presents many advantages. Briefly, it is possible to emphasize the versatility and speed of execution, besides the fact that spares the excavation process to the panel's execution, with technical and economic advantages. One of the specificities of this technology is that it allows to incorporate the soil in the structure itself, reducing the need for excavation and material transportation. In fact, there is a use of existing soil as building material, and even a natural material that is not competent for a particular engineering work, when treated and decontaminated, can fully integrate the final solution. It should be noted that in the specific case of the present solution the "spared" excavation volume, compared to the excavation volume of the two platforms is insignificant, but this does not imply that the incorporation of the natural material does not constitute an advantage over other methodologies. Another of the advantages is the drilling equipment, which allows the excavation of various types of materials with the same element and, therefore, presents a high work efficiency, which translates the decrease in the working time. It should also be noted that during the construction process there is a constant control of the geometry of the panel. Also, taking in account the overlap between the panels, it is guaranteed a final product with good mechanical properties. However, CSM panel's characteristics are conditioned by the type of soil that composes the mixing product, contrary to what happens for the piles, whose final characteristics are known from the outset.

Based on the numerical analysis results, it was guaranteed the suitability of both the solutions to the context, which achieved good structural results and constructive adequacy. Considering the scenario under study, and analyzing the results of the numerical modeling, it is verified that at the end of the corresponding constructive phases of the solutions, it was obtained safety factors approximately 1,5 times higher than the one established for the current reference situation, 1,09 to about 1,6 in the two stabilization solutions. As expected, the replacement of natural material, with poor strength properties, by lightweight aggregates results in the relative increase of the overall stability of the slope where the Miradouro sits. In addition, the structural elements also play an important role in the stabilization mechanism, once it intercepts the assumed slip surface, increasing as well the soil's shear resistance.

With relation to third solution, regarding the results of the numerical modeling, it was verified that, despite the robustness of the elements that constitute it, the increase of the security in the global slope's stability falls short, comparing to the obtained results for the two another solutions, which are characterized by the excavation of the platforms level. To this last solution, the obtained safety factor in the end of the piles execution was about 1.4, which is considered as a low value to guarantee the long-term safety of the site. In addition, it should be noted that an analysis was not carried out under dynamic conditions, but because the static value is so low, the proposed solution is not expected to be sufficient to withstand in the event of an earthquake. Given this, it is recommended to reinforce the proposed solution at the wall of Travessa do Fala Só.

- **Economical evaluation**

In order to compare the proposed solutions in terms of economic feasibility, a small cost analysis was carried out. The costs associated with equipment mobilization, shipyard assembly and instrumentation and monitoring equipment were neglected at this analysis.

Considering the high number of elements that constitute the solutions, it was took into account the major constructive operations for its execution, being adopted some approximations and simplification. In the table below, Table 4, is represented the estimated total costs to the solutions.

Table 4 - Estimated costs to the proposed solutions

Stabilization Solution	Nr.	Estimated costs
Piles wall with light weight aggregates - Sol.1 (4)	(1)	1 939 764 €
CSM wall with light weight aggregates - Sol.2	(2)	1 634 312 €
Common elements to Sol.1 and Sol.2	(3)	471 154 €
Excavation process (common to Sol.1 and Sol.2)	(4)	1 345 177 €
Piles wall – Sol.3	(5)	2 627 724 €

In relative terms, the higher costs related to the first solution described above are assigned to the reinforcement rods required to ensure the strength and ductility of the piles. Considering only the elements relating to the retaining structure, it is estimated, to its construction process, a cost of EUR 2.41 million (sum of cost (1) and (3) of Table 4).

To the second solution, it was verified that the cost of CSM panel's execution is almost equal to the cost of the steel beams, necessary as reinforcement elements to the retaining structure. Based on Table 4, it is estimated that the monetary cost associated with the first stabilization alternative (Sol.2) is approximately EUR 2,105 million (total cost results from the partial costs (2) and (3)). Comparatively, it was noted that this last solution is more economical than those who uses the piles as a structure element. Thus, if we consider only the costs related to the execution of the structure, there is a price decrease of around EUR 305450, corresponding to a saving of around 16% compared to the Sol.1. It should be noted that even if the unit price for the excavation process using CSM technology was 90 € / m^3 (it was considered 80 € / m^3 on this analysis), the associated solution was still cheaper than the previous one (approximately total cost of EUR 2.196 million, in this case), and therefore we can conclude that this is a viable alternative.

Based on the results expressed in these tables, it is estimated that the remaining elements that comprise the containment solutions have a total cost of approximately 471 thousand euros, corresponding to an approximate increase of 35% to 40% in relation to the execution cost of the retaining structure itself. These elements allow solidifying all the elements of the structure, reason why its influence in the good operation of the structure justifies the total cost.

Adding the partial values obtained, (1) + (3) + (4) for the solution with piles, and (2) + (3) + (4) for the alternative solution with CSM, it was obtained a total cost of EUR 3.76 million and EUR 3.45 million, respectively. It should be noted once again that the estimated values are only related to the containment structure itself, and subsequent backfill, neither the costs associated with the replacement of street furniture on both platforms nor the various constraints associated with the solutions were accounted for.

Notwithstanding the above, what is most important to clarify, in terms of costs related to this solutions, is the significant influence of the excavation process on the total cost of these solutions, corresponding to approximately 40% of the estimated total value for the solutions. In addition to these costs, there are still others that must be considered, which are related to waste management, whose costs are directly related to the volume of excavated material.

As mentioned, the second alternative comes in the direction of increasing the stability conditions of the Miradouro, while avoiding the excavation at the level of the platforms and consequently the costs associated with the movement of land, still conditioned by the difficulties of logistics and transportation of the same in the surrounding area.

Thus, the costs associated with the construction process are shown in Table 4. From the results obtained, it turns out that the third alternative solution is cheaper than the previous ones, with a total cost of approximately EUR 2.6 million, which is about 31% and 25% less expensive than the other solutions.

6. CHOSEN STABILIZATION SOLUTION

According to the established objective, which was to increase the overall stability in situ, the chosen slope stabilization solution is very similar to the third solution (Sol.3), and it actually comprises the same type of elements, as such as two main concrete pile walls, in which between are developed alignments of intermediate piles, positioned as buttresses. All these pile elements have a 1m diameter, are spaced 0,80m, and are embedded in the competent strata of at least 5m; the buttresses are spaced about 4m between axis. The connection between elements is carried out by rigid crown beams.

In order to increase the stability of the lower masonry wall, it is prescribed a set of steel rods, attached to the lower curtain's beams, whose function is to tie this wall to the pile grid.

Considering the low safety factor value associated to the third solution, the effective stabilization solution prescribed to the site also comprises the execution of micropiles, as schematically shown in Figure 26. These elements provides the increase of the masonry walls stability, once it nails the wall to the ground.

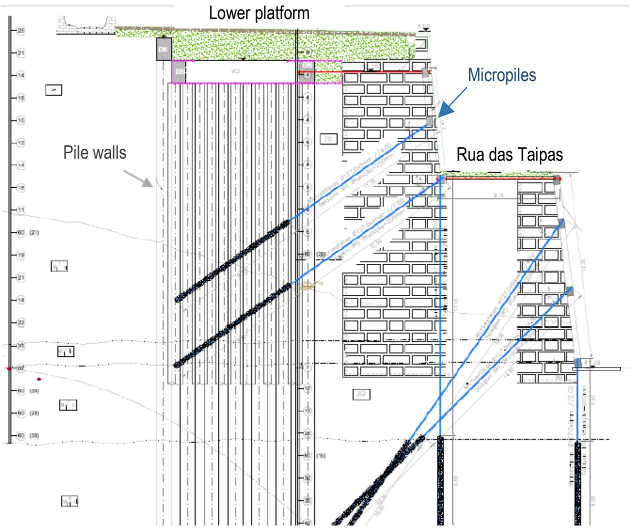

Figure 26- Chosen stabilization solution (schema)

The construction work's related to this solution can be seen in the follow photos, Figure 27 and Figure 28, where is possible to identify the pile grid placed in the lower platform, in accordance with the previews considerations, as so as the further pile's execution (excavation process in Figure 29).

Figure 27 - Stabilization solution: Pile grid location in lower platform

Figure 28 - Stabilization solution: Pile grid location in lower platform; beginning of excavation process

Figure 29 - Pile walls excavation (drilling machine)

7. FINAL CONSIDERATIONS

In conclusion, given the identified phenomena, it is recommended the structural reinforcement of the Miradouro's area, as soon as possible, in order to control the wall's displacements and to cease the geological device's evolution, since in the current situation the overall safety factor is reduced (1.1).

To this achievement, it is possible to ensure that all the studied solutions acted towards the relative increase of the slope's global stability, where the Miradouro de São Pedro de Alcântara sits.

By a particular comparison, in relation to the solution using the CSM panels (Sol.2), it was found to be a viable alternative to Sol.1. Its viability is related either to the technical field, once it exhibited an appropriate behavior, complying with the necessary safety checks, and induced an increase in the global safety factor by approximately 1.5 times compared to the reference situation. Besides, this solution demonstrated to be a viable solution in economic terms, once it is relatively less expensive than the solution using the piles as retaining walls. As such, the use of deep soil mixing technology proved to be adequate taking into account the existing constraints in the place, namely its compatibility with the geological-geotechnical conditioners.

With regard to the solution that minimizes the excavation, Sol.3, compared to the previous ones, it was proved to be a relatively expensive solution, which, despite filling the problem of the soil movement, presented less relative efficiency in the stabilization solution of the Miradouro.

In this sense, and apart from the economic questions, it may be concluded that the solutions that presuppose the decrease of the unstable weight has a relatively greater effect on the restoration of safety conditions, producing higher safety factors, compared to the solution that minimizes the volume of excavation. Besides, the decrease of unstable mass at platform level, and consequently, and particularly, behind the lower retaining wall, promoted the reduction of the surcharge on this structures, and then the decease of its movement, acting on the support structure and the increase of its own stability.

Regarding the third solution, is important to mention that, in order to obtain a safety factor with the same value as the ones obtained to the solutions that contemplate the excavation, and the one that minimizes the excavation, the lower platform should be "armed" with a very dense structure. In view of the fact that the solution under consideration includes stabilization curtains for cuttings, and that for its execution there is excavation volume proportional to the area of the stake, this solution would not serve the purpose for which it was thought, and therefore would no longer be valid as an alternative.

Finally, it is also concluded that the solutions that contemplate the excavation of the platforms has the additional advantage of allowing to incorporate throughout the area of intervention several drainage devices, which manifest significant importance in reducing the harmful effects of water, either by minimizing the hydrostatic impulses on the retaining structures, as by the control of the confinement state of the soil.

8. REFERENCES

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