Slope Stability in Road Infrastructures

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Abstract

When trying to define the layout of a road, it’s necessary to take into account several factors, each one of those factors recommends certain orientations, which are not always convergent. Therefore, the solution to adopt it will be, always, the result of the ponderation of a group of various conditionings.

During the construction of a road section in IC9, it was necessary to re-establish the Road of Pêras Ruivas. The road was relocated to the top of an excavation slope. Several episodes of slope failure have been observed since the road was implanted there.

The objective of this dissertation is the study and comprehension of the instability phenomena observed, in the context of slope stability, with the purpose to safeguard the road and the slope, proposing two reinforcement solutions to guarantee its safety.

In a first approach, the analysis and interpretation of the occurred events led to the diagnosis of the instability phenomena. Based on that diagnosis, a model of the problem was defined, which was the groundwork for the definition of the two proposed solutions.

Trough back-analysis the established diagnosis was validated, using the finite element method, with the program Plaxis 2D, comparing the results obtained in the numerical simulation with the readings of the instrumentation present on the slope, and the observed instability mechanism.

The two possible stabilization solutions, modelled using the program Plaxis 2D, were structurally dimensioned by EC2. A comparative study was carried out between both solutions, in order to find the most advantageous one.

Keywords: Slope stability, Back-analysis, Finite elements, Numerical modelling, Instrumentation.

1. Introduction

There are many conditionings to define the layout of a road, in the present theme, the most relevant ones are associated to the characteristics of the region, as its topography, geology and geotechnical features.

The defined topography should adjust as much as possible to the natural terrain, to avoid large earthworks, which in addiction to expansive the project, can give rise to several problems, such as slope stability problems and settlements, among others. It’s known that 30 to 40% of a total cost of a road work can be function to earthworks. Great earthworks also imply the need for the construction of works of art, tunnels, walls, and aqueducts, restoration of interrupted roads and major disturbances of the landscape [1].

It’s also recommended to guide the layout through suitable soils, since stability of slopes depends directly on the present geological and geotechnical conditions. In relation to the foundation of the landfills and other works, the more plastic and compressible the soil, the greater the difficulty to find economic and effective foundation conditions, among risk of undesirable settlements, or even failure. There is also an interest in guiding the layout of a road so that the excavated materials are suitable for using in landfills or pavement layers, if possible, avoiding their transport to a ditch [1].

Therefore, geotechnical engineering has a very strong presence in road works of this type, taking into account the aforementioned constraints, and the fact that a large part of the road network is embedded in embankment or excavations slopes.

The present case is given by an instability situation of a road on the top of a slope, referring to the reestablishment of the Road of Pêras Ruivas, named Reestablishment 10, due to the construction of IC9, located between Ourém and Alburitel. The re-established road was implanted on the top of an excavation slope, which has been experiencing several instability phenomena, since its construction in 2011, until present date.

The main objective of the present study is the analysis and understanding of the instability phenomena present in the excavation slope, in order to find the best possible solution, towards to safeguard the Reestablishment at the top of it. With the results of geological and geotechnical prospections, the characteristics of the soil present in the slope were evaluated and analysed. This analysis was made with the intention to obtain the most complete information regarding its characterization and geotechnical zoning, and,
consequently, better support the instability phenomena observed. It is proposed, as from the results obtained, the creation of a three-dimensional model of the problem so that the possible causes of destabilization are easily interpretable and very visible.

The confirmation of the diagnosis was made through back-analysis method, in the interest to complete a reliable assessment of the slope behaviour, and perception of the instability phenomena, through the finite element program Plaxis 2D. The program, considering the interaction between stresses and strains experienced in the ground, is able to validate, or not, the diagnosis, through a comparative analysis between the results of the instrumentation placed in the slope and the respective displacements obtained. Hence, it is possible to define the strength parameters of the unstable material, compatible with the observed instability, which can be used in the modelling of the stabilization solutions.

There were chosen two stabilization solutions considered pertinent, and modelled iteratively with the aim to obtain the best possible soil-structure interaction. It’s intended for the stability solutions to guarantee the complete global stability of the slope. As the compete definition of the chosen solutions, their structural safety was verified by EC2.

2. Case Study

2.1. Backgrounds and chronological evolution

The IC9 is a complementary route, transverse along the western coast of Portugal, which benefits accessibilities between the municipalities of Nazaré, Alcobaça, Batalha, Porto Mós, Leiria, Ourém and Tomar. Figure 1 show the respective sections of this itinerary, where the section in case is identified, and its construction started in 2011 [2].

The slope under analysis is an excavation slope, located at km 50 + 200 of IC9, on the right side - Southwest, towards Ourém-Tomar, between Ourém and Alburitel. The slope presents approximately 20m of height and in its top is implanted the Reestablishment 10 (Road of Pêras Ruivas), founded on a landfill. In the aerial view of this zone, shown in the Figure 2, corresponding to the construction phase of the IC9, it is possible to identify the slope in question, as well as the section of IC9 where it is inserted [3].

During the beginning of construction of the referred section of IC9, in October 2011, the first landslide occurred in the slope, after the excavation works. At that time, the excavation was carried out with a gradient of 1V: 1.5H, with benches 7m a part [3].

The occurred a landslide is shown in Figure 3.

Since October 2011, several instability phenomena took place, and the slope has been in between repairing works and landslides for 6 years now. In the Figures 4 to 10 above, is identified a resume of the sequence of the problems occurred in the referred slope.
Figure 5 – View of the slope, in November 2013, after reparation works, due to the heavy rain that caused several damages in the winter 2012/2013 [3].

Figure 6 – Sliding occurred in February 2014 [3].

Figure 7 – Gabion Wall prostrated due to the successive landslides, in October 2014 [3].

Figure 8 – View of the metallic pile walls implemented to stabilize the slope, in January 2016, defined in the “Execution Project of Reparation and Stabilization of the Excavation Slope at the km 50+200 (right side)” in June 2015 [3].

Figure 9 – Slide materialized in February 2016, in the top side of the slope [3].

Figure 10 – View of the slope and its successive interventions, as well as some fissures above an impermeable blanket in order to minimize water infiltration, in January 2017.

2.2. Geological Framework

The slope is inserted in the sedimentary basin of Ourém. According to the geological map of Portugal – Sheet 27-A, published by Geology and Mining Institute, the basin is formed by a depression composed by materials of the Cretaceous and Cenozoic periods [5].
The highest levels in the interior of the Sedimentary Basin of Ourém are generally crowned by limestones of the Turonian age, forming well-marked cornices in the topography. Supported by these limestones are sub-structural reliefs, covered by layers of the Miocene, forming a flattening surface, which arranges progressively to southeast and reaches the maximum altitude of 240-250m, in the region of Ourém-Alburitel [6]. These highest elevations inside the basin are almost always associated with the "Marlstones of Ourém and Batalha" - \( C_{OB} \). These limestones of the Turonian circumscribe the small flattened isolated areas, in which one the castle of Ourém is located [5].

The "Marlstones of Ourém and Batalha" - \( C_{OB} \) succeed in sedimentary continuity to the "Conglomerates of Caranguejeira" - \( C_{CA}^{2-3} \) and the separation between both is considered whenever occurs the transition from the siliceous to the carbonated domain [6].

The "Caranguejeira Conglomerates" - \( C_{CA}^{2-3} \) are deposited over Jurassic sediments in angular discontinuity [6] [7]. At NE of the study area occur the "Sandstones of Alburitel" - \( M_{AB}^{5-6} \), from the Upper Miocene, exclusively on the Ourém upland, covering the Cretaceous unit previously described, "Marlstones of Ourém and Batalha" - \( C_{OB} \).

Figure 11 shows a scheme of the described geomorphology and stratigraphy [6] [7].

![Diagram](image_url)

**Figure 11** – Scheme, merely illustrative, for the interpretation of the constitution of the sedimentary basin of Ourém, adapted from [8].

In Figure 12 is presented the location of the study area in an extract of the geological map of Portugal – Sheet 27-A, published by Geology and Mining institute.

![Map](image_url)

**Figure 12** – Location, in an extract of the geological map of Portugal – Sheet 27-A, published by Geology and Mining institute, of the case in study.

### 2.3. Prospection Works

Due to the landslide occurred in February 2014 a program of prospection and monitoring was defined in the slope, allowing a better knowledge about the present materials. The prospection work, carried out in September and October 2014, included the execution of 14 drilling holes, with full sample recovery, with the installation of 8 inclinometers and 6 piezometers, distributed along the slope. The location of the inclinometers installed in the drilling holes is shown in the Figure 13.

![Diagram](image_url)

**Figure 13** – Location of the inclinometers installed in the slope, adapted from [7].

The results of the prospections works confirmed the presence of sedimentary formations in the lands evolved in the landslides, including layers of clay, silty-clay, sandy-clay, and clean sand. In this upper horizon the soil was saturated [3]. The landslides frequently occur after heavy rainfalls, regarding the unfavourable hydrogeological conditions in the present scenario, due to the occurrence of almost impermeable marl layers, in a low depth, and unfavourable inclination (15-20° North), under the sedimentary layer. This interface restrains the infiltration of the water in the overlying detrital sediments, which quickly become saturated and subjected to percolation forces towards North, favouring the landslides [3].

The results of the readings in the installed inclinometers, dated since December 2014 to October 2016, verify the tendency of the cover soils and detrital sedimentary formations (sands, silts and clays), to slide over the layer of marls. This phenomenon is more evident in the inclinometers I3 and I4, where the displacements reach 20mm and 38mm, respectively, in October 2016, as presented in the Figure 14.

![Diagram](image_url)

**Figure 14** – Readings on the inclinometers I3 and I4, adapted from [3].
The interface that constitutes the sliding surface is clearly visible. Soils as detrital coverings formations, coloured in yellow, in Figure 14, resting over a more resistant layer of marls, coloured in grey, give rise to a mass movement, probably due to lubrication processes associated to the effect of water circulation.

In October 2016 a geological-geotechnical study was carried to complement the prospecting and monitoring program of 2014, in the face of recurrent landslides phenomena. Mechanical prospection consisted in 6 drills complementary to those performed in 2014 (SC1 to SC6), executed by rotation, with continuous sampling and followed by dynamic penetration tests (SPT) [7].

These surveys aimed the ground recognition, in situ testing and the sampling of soils/rocks in depth for their visual observation and geomechanical characterization. Several samples of soils collected in the surveys were also selected for laboratory characterization. It should also be noted that inclinometers were installed in all the drill holes.

2.4. Geological and Geotechnical Characterization

Based on the geotechnical prospecting work, it was possible to perform a geological and geotechnical characterization, present in [7]. The main conclusions are presented in Table 1, referring to each layer of soil verified in the prospecting works. With of the results obtained in the prospecting campaign, the strength and deformability characteristics are shown for the different lithological types in question, according to the values of NSPT and RQD, which are presented in the same Table.

Table 1 – Geotechnical characterization of the slope, adapted from [7].

<table>
<thead>
<tr>
<th>Lithology</th>
<th>Nspt</th>
<th>RQD (%)</th>
<th>W/F</th>
<th>φ '(*)</th>
<th>c' (kPa)</th>
<th>Cu (kPa)</th>
<th>E' (Mpa)</th>
<th>Eu (Mpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Landfill</td>
<td>2-21</td>
<td>-</td>
<td>-</td>
<td>25-28</td>
<td>-</td>
<td>-</td>
<td>5-10</td>
<td>-</td>
</tr>
<tr>
<td>Clayey Sandstones, Silts and Sands</td>
<td>&lt; 30</td>
<td>-</td>
<td>-</td>
<td>32-35</td>
<td>-</td>
<td>-</td>
<td>60-90</td>
<td>-</td>
</tr>
<tr>
<td>Marl Clays, Clay Marls and Silty Sand Marls</td>
<td>&lt; 30</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>120-180</td>
<td>-</td>
<td>36-54</td>
<td>-</td>
</tr>
<tr>
<td>Limestones, Marley Limestones and Sandstone Marls</td>
<td>&gt; 30</td>
<td>-</td>
<td>-</td>
<td>35-38</td>
<td>-</td>
<td>180-360</td>
<td>-</td>
<td>54-120</td>
</tr>
</tbody>
</table>

2.5. Model of the problem

It was possible to model the surface of the terrain, from the topographical information collected. The surface is presented in Figure 15.

Combining the surface presented in Figure 15, with the results from the prospection works along the slope, it was possible to define a model in depth, regarding all the geometric nature of the problem, where is possible to assess the unfavourable inclination of the marls, causing the sedimentary formations above to have the tendency to slip. The model is composed by two longitudinal cuts and eight transversal ones, with the corresponding geotechnical zoning presented.

![Figure 15 – 3D model of the surface of the slope.](image)

From the 3D model it was chosen a representative section to model the problem. The starting point for the choice of the section were the readings obtained from the inclinometers installed in 2014, since it is possible, from those, to assess the results obtained in numerical modelling, through back-analysis.

The S3-S4 cut, referring to the results of the inclinometers I3 and I4, installed in September and October 2014, was considered as the representative cut to study the stabilization solutions. The inclinometers I3 and I4 present the higher displacements, as already mentioned.

Figure 17 shows the section in question, with the identification of the extracts and their thickness. It should be noted that the gabion wall is merely representative, because although it is in the slope, it has already been overturned,
thus confirming that its presence does not have a significant influence on results to come.

Figure 17 – Chosen Section to model the problem.

3. Numerical Modelling

**Framework**

Classical soil mechanics addresses slope stability through limit equilibrium method. Its application implies the definition of a collapse mechanism. The equilibrium is calculated by considering the forces and/or moments applied to the block or set of blocks, defined by the mechanism. The commonly adopted procedure uses the slices method, where the potentially unstable mass is divided into slices, each individual slice being balanced, and the sum of the contributions of the various slices is taken at the end. These methods allow studying a variety of situations, from circular, planar and composite surfaces, in drained and undrained conditions, to slopes with more complex geometry and geological conditions, as external loads [9].

By defining a failure surface, the ratio between the actually shear strength available and the shear strength given by the destabilizing forces, provides a stability index, called safety factor. This factor compares the stabilizing and destabilizing forces/moments along the failure surface [10].

According to [10], calculated safety factors are meaningless, except for those that imply failure, that is, when they are less than unity. They should be used only as a relative stability index, useful when considering slope corrective measures, e.g. "If this is done instead of that, which solution will have the best results?" The fact that the shear stress evaluated in the soil is uniformly mobilized along a potential failure surface, and when a single SF is calculated on that same surface, it does not take in to account the soil behaviour at the level of its relation between generated stresses and strains.

Stability analyses performed by the limit equilibrium method, based on the slice method, are based purely on static principles: sum of moments, horizontal and vertical forces. This method nothing contemplates regarding stresses and deformations, resulting in non-satisfaction of the compatibility between them. This is the key piece for the loss of physical meaning in this method, overlying the many difficulties experienced by it [11].

In this way, the finite element method allows the overcoming of limitations inherent to the formulation of the limit equilibrium method.

The finite element program, Plaxis 2D, is used to perform deformation and stability analysis for various types of geotechnical applications [12].

### Material Models

Both soils and rocks, when subjected to loading, preform a highly non-linear behaviour before yielding, unlike many structural materials. Soil characterization starts with the choice of the constitutive model that best suits its mechanical behaviour [13].

One of the main differences between the plastic and elastic responses is that the plastic deformations are not recoverable when the stress state returns to its initial value, only the elastic deformations that contributed for the material to the yield are recovered. The perfect plasticity aims the description of the non-elastic behaviour of the soil, meaning the accumulation of irreversible deformations. The hardening plasticity additionally allows describing the non-linearity before failure [13].

The behaviour of the rock material differs from soils given that rock is generally more rigid. The stiffness dependence by the level of tensions in rock materials is practically negligible, therefore, the rigidity of rock can be taken as constant [14].

The program Plaxis version 8.2 has some limitations in modelling rock masses, and the failure criteria of Hoek-Brown, the best approximation for the modelling the non-linear type of resistance of the rocks, is only present in more advanced versions the program. A reasonable approximation for modelling the rock shear strength is by the Mohr-Coulomb criteria [14]. However, the layer that includes limestone rock formations, named GZ5, has a very heterogeneous character, assessed by its geological and geotechnical characterization, able by the prospection works performed, and it presents itself from moderately altered to decompose, sometimes whit soil behaviour. In this way, and also for consistency matters, the Hardening-Soil model was adopted for both soil layers - ZG1, ZG2, ZG3 and ZG4, as well as ZG5. This model is suitable for modelling different types of soils, from sands and gravel to silt and clays [15].

The Hardening-Soil model geotechnical parameters of the different substrates that characterize the slope are exposed in Table 2.

<table>
<thead>
<tr>
<th>Table 2-Table 1: Soil parameters (Hardening Soil Model)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parameters</td>
</tr>
<tr>
<td>Type of material</td>
</tr>
<tr>
<td>( \gamma_{\text{unsat}} ) (kN/m(^3))</td>
</tr>
<tr>
<td>( \gamma_{\text{sat}} ) (kN/m(^3))</td>
</tr>
<tr>
<td>( c' ) (kN/m(^2))</td>
</tr>
<tr>
<td>( \phi ) (°)</td>
</tr>
<tr>
<td>( \Psi ) (°)</td>
</tr>
<tr>
<td>( E_{50}^\text{ref} ) (kN/m(^2))</td>
</tr>
<tr>
<td>( E_{\text{ref}} ) (kN/m(^2))</td>
</tr>
<tr>
<td>( E_{\text{ref}} ) (kN/m(^2))</td>
</tr>
<tr>
<td>( m )</td>
</tr>
</tbody>
</table>
It should be noted that the parameters present in Table 2, were not the initially considered according to the geotechnical characterization, presented in Table 1, but rather those obtained through retro-analysis, guaranteeing the unit safety factor, subject that will be dealt afterwards.

3.1. Back –Analysis

Stability analyses are performed not only in order to obtain a safety factor, once the properties of the soil are known, but also to establish field shear strength from the study of a failure. In order to do this, it is necessary to perform the reverse analysis, obtain the properties of the soil from a given safety factor, knowing that the safety factor is unitary at the imminence of failure. This process is called back-analysis [10].

The strength and deformability parameters of the different extracts presented in Table 1 were considered as a first estimation. It’s intended to impose to the program the failure surface observed and the displacements obtained by the inclinometers I3 and I4, satisfying the condition $FS = 1$, iteratively changing the resistance parameters initially considered.

Stability analysis made by Plaxis are performed through a type of calculation called $Phi-c$ reduction, this is done by reducing the soil strength parameters. Instead of the traditional way, in which the calculation of the safety factor is done from limit equilibrium, in the program it is calculated by gradually reducing the $\tan \phi'$ and $c'$ strength parameters, until there is no distribution able to satisfy the Mohr-Coulomb criteria and the global equilibrium, until the soil failure. Thus, using the $Phi-c$ reduction calculation in combination with more advanced models to simulate soil behaviour, such as the Hardening Soil Model, these will behave according to the Mohr-Coulomb model, excluding the analysis of the stress-dependent stiffness and hardening effects [12].

With this procedure, the resistance parameters presented in Table 2 were obtained, as an estimation of the ground properties as it behaves according to the instrumentation. The obtained deformed mesh is presented in Figure 18, generated by the calculation phase in which the weight of the soil and the road were activated. It is possible to have a perception of the instability phenomena occurring in the slope observing this mesh.

Figure 19 shows the horizontal displacements, obtained in the same calculation phase as the deformed mesh presented in Figure 18. Notice that the maximum horizontal displacements in the program are very similar by the ones given by the inclinometers I3, 20mm, and I4, 38mm, evidence that the result is reliable, according to the displacements of 28 and 36mm, respectively, given by Plaxis in this phase.

During Phi-C reduction calculation phase, additional displacements are generated. The total displacements in this phase do not have any physical meaning, as they are related to the program non-numerical convergence, in which the plastic condition can present unlimited displacements [12]. However, the incremental displacements in the last calculation step, can give an indication of the most probable failure mechanism, presented in Figure 20. The obtained safety factor is 1,099.

3.2. Stabilization Solutions

3.2.1. Solution 1 – Bored Pile Wall

The first considered solution was a bored pile wall in reinforced concrete. This retention wall aims the protection of the Reestablishment on the top of the slope, and it will be designed in order to intercept all the possible failure surfaces in a greater depth, eliminating the unstable material between the future wall and the IC9. Figure 22 shows a transversal view of the wall.

The bored discontinuous piles, with 800mm diameter, spaced 1,5m apart, have variable lengths from 18 to 30m,
according to Figure 21, in order to ensure the embedding at a proper depth and passive impulse mobilization. As structural solutions designed to be in contact with terrain, a concrete C30/37 and A500 NR SD steel were adopted. In order to promote the clamping of the wall, to limit its deformation and to guarantee its equilibrium, it was defined the installation of 2 levels of micropiles in steel N80, Ø127x9mm, with a free length of 16m, 44° incline, and 2,5m distance apart in plant. The loads transmission originated in these elements is achieved through 8m long sealing bulbs. The micropiles must subsequently be attached to a crowning and distribution beam, allowing a better distribution of stresses in the wall, and avoiding excessive concentration of stresses.

![Figure 21 – Schematic front view of solution 1.](image)

![Figure 22 – Schematic side view of solution 1.](image)

The bored pile wall was modelled with the control Plate, in the program Plaxis 2D, characterized by a significant bending stiffness and normal stiffness [12]. The micropiles were modelled by a combination of two different controls, the Node-to-Node Anchor tool to simulate the free length, and Geogrid tool to simulate the sealing bulb. In reality, there is a complex three dimensional state of stresses around the bulb, and it’s not possible to represent that precise state in a two dimensional model, but it’s possible to estimate the stress distribution, strains and global stability in this way. The incremental displacements showing the most probable failure mechanism are presented in figure 23, as the global factor of safety of 1,55.

![Figure 23 - Results of the last Phi-C reduction calculation step, with identification of the failure mechanism and safety factor, for Solution 1.](image)

The deformed mesh is presented in Figure 24, as well as the total maximum displacements of 18,85mm. The maximum horizontal and vertical displacements obtain were, respectively, 16,9mm and 14mm.

![Figure 24 – Deformed mesh for solution 1.](image)

3.2.2. Solution 2 - Viaduct

The second solution considered was a viaduct type solution, in order to safeguard the reestablishment. The reinforced concrete structure, executed against the ground, would be composed with 600mm in diameter bored piles, functioning as pillars, spaced apart from each other 6m lengthwise and with variable lengths, from 18m to 30m, according to previously mentioned wall, in order to ensure an embedding that reaches a suitable depth. The piles will support a platform 0.4m thick and 6.6m wide, over which will be implanted the road. The aim is for this viaduct to connect directly to the existing overpass in IC9, as shown in Figure 25. Figure 26 and Figure 27 show, respectively, schematic front and side views of the solution.

![Figure 25 – Air view of the composition of solution 2.](image)

![Figure 26 – Front view presenting Solution 2.](image)
The bored pillars and platform were modelled with the control Plate. The safety factor, of 3.56, and most probable failure mechanism are present in Figure 28. Figure 29 shows the deformed mesh. The total maximum displacements obtained were 5.7mm, and the horizontal and vertical ones, respectively, 2.71mm and 5.62mm.

Both solutions involved a high and complex component of earthworks, although in the second solution it’s significantly superior to the first one. These earthworks will have to be carried out with strict quality control and in a phased form, ensuring the safety of the reestablishment, and the excavated soil must be taken to the ditch, whose availability and proximity should be accounted.

An approximate economic analysis between the constructive solutions considered was made, in order to evaluate the economic viability of their application. Only the most important constructive operations were considered in this analysis, disregarding drainage devices and instrumentation and monitoring equipment.

For the economic analysis of the slope in question, it was considered the area of influence of its destabilization, according to all the prospections effectuated, this area is shown in Figure 30.

![Figure 27 – Side view of Solution 22, showing the construction phase before removing the unstable soils.](image)

![Figure 28 - Results of the last Phi-C reduction calculation step, with identification of the failure mechanism and safety factor, for Solution 2.](image)

![Figure 29 - Deformed mesh for solution 1.](image)

4. Comparative analysis between solutions

Both solutions were structurally designed from the stresses given by Plaxis, according to EC2, regarding the ULS in question, and from the strains obtained by the program the SLS were verified to.

A comparative study between both solutions was made, analysing the guarantee that each one offers in terms of safety, regarding the slope stability, and verifying their economic viability, through estimated values for each solution, from the constructive operations considered more relevant.

The foundation of the two solutions resides in the inefficiency of the landfill, in which is founded the reestablish road. But their character differs essentially in the fact that, the first solution takes advantage of the landfill, by applying retention, and the second one completely eliminates it.

The final costs of the stabilization solutions show a very similar order of magnitude, since they are carried out, mainly at the expense of earthworks. The estimated values for the pile wall and the viaduct are, respectively, €865,745.76 and €800,987.77. The first solution considered to stabilize slope, is then considered to be the least economical, as would be expected, since it is a more robust solution, in the sense that it consists of a number of linear meters of piles that is about

\[
V = A_p \times h = \left(\frac{17 + 3}{2}\right) \times 112 \times 55 = 61600 \text{ m}^3
\]

![Figure 30 –Three dimensional models, regarding the volume of soil considered.](image)
the double of the second solution, as well as a considerable number of linear meters of micropiles. However, it is verified that in the viaduct solution the cost of the platform has a significant weight in its final price.

According to the above, regarding the increase of the safety factor by the eliminating the embankment, in the viaduct solution, considering that its estimated final cost is economically more favourable, the viaduct solution is considered to be the most competitive solution for the present case study.

5. Conclusions and Future Works

As epilogue is highlighted the extreme importance of the elaboration of a Geological and Geotechnical Prospection Plan, considering it as an investment other than an added cost, in order to limit the possibility of occurrence, through its anticipation, of destabilization phenomena. An adequate Instrumentation and Observation Plan is highlighted as well, in order to allow a timely interpretation of the behaviour of the soil through its readings, and to take action in case of excessive deformations or failures.

It should be noticed that, if it had been done prior to the construction of the section of IC9, an adequate prospection plan, and the correct interpretation of the existing geological-geotechnical scenario, very likely it would have been spared various resources mobilized to resolve the situation in question.

Once again, underlining the importance of an Instrumentation and Observation Plan, in order to validate the criteria assumed in the project, trough back-analysis, which was verified in this same document, to be a very useful tool.

In order to understand the destabilization phenomena developed in the slope over the 6 years, an analysis of the case scenario was made, based on reports and studies carried out to date, as well as results of geological surveys and geotechnical data, which served as a basis for diagnosing the instability phenomena, and for modelling the problem. It was possible to compare the results obtained in the modelling with those provided by instrumentation, and to validate the diagnosis. Finally, two stabilization solutions, and their respective technical and economic analysis, were presented, verifying that a viaduct kind of solution will be less expensive at the outset, allowing a better control of quality, being more advantageous that a bored pile wall solution.

References


