Urban Slope Stabilization

Case study – Reinforcement and Reconstruction of a Slope Retaining Wall

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Abstract: The exponential population increase and growing soil occupation have prompted the need to find ways to give a better use to the soil, namely the construction of retaining walls, allowing the stabilization of high height slopes and the increase of stable construction areas. This thesis aims to analyze and follow the execution of the reconstruction and reinforcement of a partially collapsed slope retaining wall. The analyzed detailed project comprised the use of micropiles, ground anchors, slab bands and nails. The reinforcement and reconstruction solutions adopted were modeled separately using the software Plaxis 2D. In both models three different actions were considered: a static pressure due to the soil’s weight and a distributed live load representative of the weight of the adjacent building and gardens; a hydrostatic pressure due to an accidental action, where there is no draining which leads to a rise in the water level behind the wall, and a pseudo static pressure due to a seismic action. All of these results were compared with the readings measured during the construction phases and the alert and alarm criteria. With a different software, SAP2000, the steel structure used to reinforce the existing retaining wall was analyzed for both the static and seismic action. The interpretation of the calculation results allowed the conclusion that the adopted geotechnical parameters were conservative, as it was not possible to access them based on a site ground investigation due to the stabilization works urgency. The analysis of the overall stability of the adopted solutions allowed the estimation of global safety factors, for the various loading conditions, higher than the minimum established according to current practice.

Keyword: retaining wall, reconstruction, reinforcement, static pressure, hydrostatic pressure, pseudo static pressure.

1. Introduction

The increase in the search for houses in the urban center and the soil occupation, is leading to the need to explore techniques employed for stabilization and containment of soils in order to allow a larger area of available land for housing.

The case study of this dissertation is located in the center of Lisbon, in an area of great housing density. The present dissertation was designed to accompany the urgent reconstruction and reinforcement of a retaining wall after a partial collapse of the existing wall. The work was carried out under the state of necessity with intervention as a matter of urgency. The collapse occurred in the early hours of February 27th of 2017. During the construction and operation, it was maintained a rigorous monitoring and survey plan in order to control the safety of the retaining wall and adjacent buildings.

The accompaniment of this work was of great interest because, due to its conditioning factors, it uses four different types of supporting and stabilizing auxiliary structures, namely: ground anchors, nails and micropiles applied to a cantilever support wall. This translates into several interesting technical difficulties from the point of view of Civil Engineering.

This dissertation also intends to model the adopted solutions, using the software Plaxis 2D and SAP2000, comparing instrumentation results, the alert and alarm limits criteria with the results obtained with Plaxis 2D, to allow constructive criticism of the applied solution.

The urgency required for the execution of the work led to the work being started without the execution project being completed, making it an atypical work, requiring a constant dialogue between the site director and the design engineer.
1.1. Earth and slope retaining walls

The support walls are intended to allow the modification of the slope geometry, there being several types of support structures, with respect to the constituent material, the shape and the constructive process.

The old cantilever walls are, in generally, reinforced concrete walls subject to high internal stresses, so they are heavily armed, being able to withstand the lateral pressures by bending and shear. These types of walls can collapse by sliding, overturning or bearing capacity and these failure modes are prevented by deep foundations, like pile and micropile, or by shallow foundations, mainly the counter weight of the soil over a large footing.

Cantilever reinforced concrete support walls are executed when the height of the wall allows deformation of its upper end compatible with the design requirements. Cantilever walls can be either L or inverted T shape. The inverted T walls are more common since in the walls in L the footing only develops behind the wall, making it difficult to connect the reinforcement of these two pieces, reducing the stabilizing moments that counteract the overturning.

1.2. Earth retaining structures support systems

The construction of peripheral restraints is sometimes insufficient to withstand the pressure from the soil, so it is normal to resort to auxiliary support structures. Most excavations in urban areas require elements that press the wall against the ground, confining its movement. The following auxiliary support structures are used in this case study: micropiles, ground anchors and nails.

Micropiles are a deep foundation element constructed using high-strength, small-diameter steel casing and/or threaded bar. This support system offers several advantages, for instance, it can be applied to any type of soil as it has relatively high load capacity even on weak or impermeable soils, fast and easy execution, it can withstand tension or compression stresses, and there is a possibility of adequate verification/control of performance. Some of the limitations are the need to specialized firms with adequate equipment and labor and limitation of the load capacity [1].

Ground anchors are structural, temporary or definitive elements, installed in the soil that are used to transmit an applied tensile load to the soil which has the main function of locking contention flexible structures. Due to pre-stress, there is a significant decrease of horizontal displacements at the top of the wall. Ground anchors have some advantages, such as a high tensile strength, and is a safe process when compared with shoring. The main disadvantages are the need to specialized equipment and manpower, it can cause damage to nearby structures, limited use by the existence of neighboring basements, delayed execution and a high maintenance cost [2].

Slab bands are a supporting system achieved by reinforced concrete framing executed at each excavation level. This support system offers several advantages, such as the non-utilization of the subsoil and, consequently, less impact on neighboring structures, it has a higher stiffness, and it does not imply drilling of the containment wall. The main limitations of band slabs are: the need to adopt vertical elements of temporary support of the structural elements and the excavation is more conditioned, especially under the structural elements adopted as locking system [3].

The nails are intended to improve the overall structural behavior of the ground by introducing steel bars into the ground. Nails have several advantages, some of them are: the simplicity of application; relatively low cost as they do not need very heavy equipment and the natural soil is part of the sturdy structure; the equipment allows low noise and vibration and it is easily adapted to the soil configuration. The main disadvantages of this technique are: the occupancy of space may not be acceptable, especially in urban areas; it can only be applied to stiff ground and it is not advisable in situations where the permissible displacements of the mass adjacent to the excavation are strongly conditioned [4].

2. Case study

The case study is located in Lisbon’s center, in an area with a large housing density and aggressive topography where a good use of available land is required, as such, a retaining wall was built, in 1955. Later, in 1994 a condominium and a pool were built at the slope crest, near to the edge.

Recently, there has been a rupture of the upper and central part of the retaining wall and there was an urgent need for intervention in order to reinstall the stability of the walls adjacent to the wall that has collapsed. The work that served as a practical case in this dissertation was the urgent reconstruction of the portion of the wall that failed and the reinforcement of the portions of the adjacent walls.
2.1. Damage caused by the collapse of the earth and slope retaining wall

The collapse of the wall under analysis occurred in the upper part of the walls located at the end of the buildings number 106 and 108, followed by the mudslides of the soils contained by it. There were several piers, beams and slabs affected by the collapse of the retaining wall. The basements from the numbers 106 and 108 were destroyed (Figure 1).

![Figure 1 – Plant of the building number 106, red circles represent the elements damage by the collapse of the retaining wall [5]](image)

2.2. Main causes for the instability

The analysis of the available elements related to the analyzed walls and the practical information collected through in situ inspection allowed to point out the following reasons as the main causes for the collapse of the walls [6]:

- Horizontal water pressure behind the walls, caused by rainfall, water action of the watering of the garden area, and probably motivated by the existence of water leaks through a swimming pool;
- Inefficiency of the drainage system of the walls, which prevented or hindered the drainage of the water accumulated behind the retaining wall, resulting in the installation of a hydrostatic pressure, causing an increase of the horizontal pressures;
- Reasons of geological and hydrogeological nature, in particular the existence of a layer of clay, located at the depth where the collapse of the wall’s was observed;
- Deficient structural safety conditions of the walls, with slightly reinforced concrete structure, consisting of a cantilever wall, founded over a spread footing. The walls are made of low strength concrete, according to uniaxial compression tests performed and a low rate of reinforcement already affected by corrosion, as well as several vertical joints.

2.3. Geological and geotechnical conditions

The studied area is characterized by “Areias da Quinta Do Bacalhau” (MQB), dating from the lower Miocene age. To the west of the intervention zone a layer of “Calcário do Casal Vistoso” (MCV) was found, while at the eastern zone the lower age unit called “Areias de Forno de Tijolo” (MFT) was detected.

The definition of the geological and geotechnical zoning was carried out based on previously performed surveys and other available information. The information collected estimates the occurrence of following materials in depth:

- Landfills with variable thickness, consisting of very heterogeneous materials, both sandy, and silty-clayey, with low levels of compactness or consistency, and characterized by spt values between 4 and 8 strokes;
- Sand complex consisting of “Areias Quintas Do Bacalhau”, characterized by fine to medium sand, silty-clayey, micaeous, often with coniferous concentrations and fragments of limestone. These sands are superficially more uncompressed to depths of the order of 7 meters, being characterized by SPT values ranging between 26 and 38
strokes. For higher depths, the compactness of the soil increases substantially; at 9 meters’ depth the SPT value is 60 strokes, interbedded with clayey silts of rigid consistency.

2.4. Adopted reinforcement and reconstruction solution

The adopted solution for reinforcement and reconstruction of the retaining wall was divided into two parts: the emergency reinforcement measures and the walls construction and reinforcement. The walls construction and reinforcement was divided into two solutions: Solution A for the reinforcement of the existing walls and Solution B for the reconstruction and reinforcement of the partial collapsed walls.

Emergency measures were of the utmost importance to establish minimum safety conditions. These were applied on the slope behind the partially collapsed retaining wall:

- Decrease of the slope inclination, in particular at the surface zone, consisting of landfill materials;
- Removal of potentially unstable blocks, identified on the slope surface;
- Lining of the slope with shotcrete.

In the Solution A, the retaining wall in the lower area of the terrace was lined with a 0.50 m thick reinforced concrete wall, founded over micropiles with 6 m length and ground anchors with an free length of 6 m and an sealing length of 6 m. Above the terrace, the existing wall was preserved and coated with mortar reinforced with a carbon fiber mesh. Subsequently ground anchors, with the same dimensions as prescribed before, were executed over a grid of steel profiles supported on the existing wall. The drainage system consisted on the execution of five levels of sub-horizontal geodrains with a 12 m length with a negative inclination, spaced 3 m from each other (Figure 2). The water flowing through these drainage devices was directed to the rainwater drainage system through gutters installed at both the balconies and the basement.

Solution B was decomposed into two different solutions: one for the bottom of the wall that did not collapse and one for the top of the wall that was completely rebuilt. The solution recommended for the reinforcement of the existing wall was similar to solution A, with some small differences: the 0.50 m thick reinforced concrete wall was extended to the bottom of the capping beam, not using the steel profiles, and in the drainage solution, the geodrains were spaced 3.5 m from each other instead of 3 m. In the upper part of the wall, a 0.35 m thick reinforced concrete wall was built, over the capping beam. At the end of the wall a landfill of lightweight aggregates was built, to limit horizontal pressure and also to improve the drainage conditions. Four levels of nails with a longitudinal spacing of 3 m were executed, using 12 m rods with different inclinations. The nails were capped by slab bands of 0.30 m thickness (Figure 2). The drainage system in the upper part consisted in the execution of 3 levels of sub-horizontal geodrains, spaced 3 m from each other.

![Figure 2 – Solution A (left) and solution B (right)](image)

The adopted solution for the reinforcement and reconstruction of damaged buildings lead to the disconnection of the first floor slabs (balconies) from the retaining wall, in order to obtain a better behavior against seismic action.
On the damaged buildings, numbers 106 and 108, the execution of a new slab for the first as well as new reinforced concrete columns was necessary. The columns constructed next to the wall for the support of the balconies slabs were sustained by the foundation beam of the new retaining wall. Carbon fiber laminates or sheeting’s were applied on existing piers and beams with a reinforcement deficit.

In adjacent buildings, slabs in a range of approximately 3 m adjacent to the retaining wall were reinforced by the application of a reinforced concrete laminate over the existing slab. Steel columns were built next to the supporting wall of the balconies slab (Figure 3). These were supported over the foundations beam of the new retaining wall.

![Figure 3 – Intervention on the adjacent buildings (left) and intervention on the damaged buildings (right) [5]](image)

2.5. Monitoring and survey plan

During the construction phase, the horizontal and vertical displacements of the buildings at the front and the back of the wall were monitored, as well as the horizontal and vertical displacements of the reinforced walls. Experimental measurements were based on topographic targets, load cells on ground anchors, inclinometers and piezometers.

The above-mentioned measures shall be carried out by [7]:

- Topographic targets to measure the displacements of the buildings in the front and in the back of the wall as well as the displacements of the wall itself, with a minimum of 32 units;
- Reflection prisms targets for a wall displacement measurement, with a minimum of 32 units;
- Load cells for load measurement installed at ground anchors, with a minimum of 8 units;
- Inclinometers for measuring the horizontal deformations of the ground behind the wall, with a minimum of 3 units;
- Piezometric tubes for measuring the water level and the length of the walls, with a minimum of 3 units.

Based on the proposed solutions for the work performed, as well as the geology of the site, it was possible to estimate standards for the peripheral wall (Table 1). Figure 4 represents the location of the topographic targets and the readings registered for the target located on the retaining wall behind building number 110.

<table>
<thead>
<tr>
<th>Construction phase</th>
<th>Structure</th>
<th>Alert Criteria</th>
<th>Rupture criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Earth retaining structure¹</td>
<td>15 mm</td>
<td>30 mm</td>
</tr>
<tr>
<td></td>
<td>Constructions behind the Earth retaining structure</td>
<td>Horizontal</td>
<td>Vertical</td>
</tr>
<tr>
<td>Exploration phase</td>
<td>Earth retaining structure¹</td>
<td>15 mm</td>
<td>30 mm</td>
</tr>
<tr>
<td></td>
<td>Ground anchors (Load variation)</td>
<td>15 %</td>
<td>30 %</td>
</tr>
<tr>
<td></td>
<td>Deformations of the soil behind the retaining wall</td>
<td>10 mm/10 m</td>
<td>20 mm/10 m</td>
</tr>
</tbody>
</table>

¹ Maximum displacements of the Earth retaining structure¹
3. Solution’s modelling

The behavior of the support wall was analyzed using the finite elements program, the 2D Plaxis V8, that allowed to simulate the nonlinear behavior of the soil as well as the main constructive stages. The solutions A and B were analyzed in detail for a static load corresponding to the weight of the soil and the condominium; for a hydrostatic loading, corresponding to an accidental situation due to the ineffectiveness of the geodrains that leads to the installation of the water level behind the wall, and to a pseudo-static pressure related to the seismic action.

The existing geological and geotechnical information did not allow a sustained characterization of the existing geological formations behind the retaining wall and in its foundation. As such, a back-analysis was performed in order to better estimate the geomechanical characterization of the intersected materials. A global stability analysis was performed using the Mohr-Coloumb Model and a phi-c reduction analysis, where it was possible to determine soil parameters that lead to a global safety factor not bigger than one (Table 2).

### Table 2 - Adopted parameters with Hardening-soil model

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Geotechnical zone</th>
<th>Landfill</th>
<th>ZG1</th>
<th>ZG2</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma_{unsat}$ [kN/m$^3$]</td>
<td>2.9</td>
<td>19</td>
<td>18</td>
<td></td>
</tr>
<tr>
<td>$\gamma_{sat}$ [kN/m$^3$]</td>
<td>2.9</td>
<td>20</td>
<td>19</td>
<td></td>
</tr>
<tr>
<td>$E^{ref}_{50}$ [kN/m$^2$]</td>
<td>8 000</td>
<td>90 000</td>
<td>50 000</td>
<td></td>
</tr>
<tr>
<td>$E^{ref}_{sed}$ [kN/m$^2$]</td>
<td>8 000</td>
<td>90 000</td>
<td>50 000</td>
<td></td>
</tr>
<tr>
<td>$E^{ref}_{ur}$ [kN/m$^2$]</td>
<td>16 000</td>
<td>180 000</td>
<td>150 000</td>
<td></td>
</tr>
<tr>
<td>$c'$ [kN/m$^2$]</td>
<td>1</td>
<td>60</td>
<td>25</td>
<td></td>
</tr>
<tr>
<td>$\varnothing'$ [$^\circ$]</td>
<td>35</td>
<td>45</td>
<td>35</td>
<td></td>
</tr>
<tr>
<td>$K_0$</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
<td></td>
</tr>
</tbody>
</table>

3.1. Static pressure

The static pressure is related to the horizontal pressure associated with a distributed vertical surcharge of 200kN/m$^2$ representative of the weight of both the condominium and the fill behind the wall. Figure 5 represents the deformed mesh of both solutions.
In solution A, the maximum horizontal displacement occurs to the inside of the excavation. This kind of displacements occurs in the zone where there are no ground anchors only the soil, which manifest themselves into the interior of the excavation. In the zones of the ground anchors the effect of the pre-stress is represented by an action to the exterior of the excavation, contrary to the pressure of the soil, therefore having a reduction of the horizontal pressure with the depth.

![Figure 5](image)

*Figure 5 – On the left, deformed mesh of Solution A, scaled up 5E+03 times; on the right, deformed mesh of Solution B scaled up 2E+03 times*

In solution B, the maximum displacement is at the level of slab bands and nails which aim to stabilize the structure without introducing any displacements, therefore the displacement felt by the structure is to the interior of the excavation. On a lower level, the ground anchors, due to their pre-stress, apply forces in the ground direction.

### 3.2. Hydrostatic pressure

The hydrostatic pressure situation is intended to represent the accidental situation of possible inefficiency of the drainage systems, which leads to the installation of a water level behind the wall, at a depth of 2 m (Figure 3).

![Figure 6](image)

*Figure 6 - On the left, deformed mesh of Solution A, scaled up 500 times; on the right, deformed mesh of Solution B scaled up 20E+03 times*

For the recommended solution, the displacements for the accidental situation are superior to the static situation.

### 3.3. Pseudo static pressure

The seismic action was analyzed using an equivalent pseudo static analysis. This type of analysis requires that the earthquake be represented by a set of horizontal and vertical static forces equal to the product of the gravitational forces by a seismic coefficient, according to Eurocode 8. This is a simplified but conservative method that allows the analysis of the slope response to the seismic action. Figure 7 represents the deformed mesh of both solutions subjected to the hydrostatic pressure.

![Figure 7](image)

*Figure 7 - On the left, deformed mesh of Solution A, scaled up 1E+03 times; on the right, deformed mesh of Solution B scaled up 20E+03 times*
Table 3 represents the overall stability safety verifications for both solutions and all load cases. Both hydrostatic and pseudo static are considered accidental situations.

<table>
<thead>
<tr>
<th>Analysis</th>
<th>Solution A</th>
<th>Solution B</th>
<th>Minimum SF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static</td>
<td>1.6</td>
<td>2.5</td>
<td>&gt;1.5</td>
</tr>
<tr>
<td>Hydrostatic</td>
<td>1.5</td>
<td>1.9</td>
<td>&gt;1.2</td>
</tr>
<tr>
<td>Pseudo static</td>
<td>1.3</td>
<td>1.6</td>
<td>&gt;1.2</td>
</tr>
</tbody>
</table>

4. Work accompaniment

The implementation of the works mentioned above, namely the execution of Solutions A and B and the reinforcement and reconstruction of the damage buildings is represented by Figures 8 to 10. Figure 8 represents the works done on the existing walls, solution A. Figure 9 represents the works done on the collapsed wall, upper part of solution B, and Figure 10 represents the intervention on the adjacent and damage buildings.

5. Alternative solutions

Alternative solutions to the one executed were briefly analyzed. For each alternative, it was considered its advantages and disadvantages, as well as its execution process. The alternative solutions have two components: the reconstruction of the part that collapsed and the reinforcement of the existing wall.
5.1. Reconstruction of the collapsed wall

The proposed solution involves reinforcement of the soil with strips of galvanized steel, aluminum or polymeric materials, resulting in a cohesive and very resistant composite material. This reinforcement system is intended to serve as an alternative to the joint solution of slab bands and nails adopted for the Solution B. Lightweight aggregates will be used as they provide enough friction with the reinforcement. Due to aesthetic reasons, the wall will be made of reinforced concrete. The connection between the wall and the reinforcement is executed with high resistance bolts, links or hook links. In order to ensure continuity between this reinforcement system and the existing ground, nails will be placed with TerraLink technology (Figure 11). The drainage system is similar to the one used in the adopted solution.

![Figure 11 – Alternative solution for the reconstruction of the collapsed wall [8]](image11)

The main advantages of this method are: it allows slopes with big inclination; it allows a fast execution and it has a certain tolerance to settlements due to its structural flexibility. The main disadvantages are: rupture in the connecting areas of structures, especially in areas of greater rigidity such as drainage structures and adjacent walls; corrosion of the reinforcement and eventual loss of frictional resistance with the ground. It must be applied to a soil that is able to mobilize enough friction in order to counteract the pressures coming from the soil.

The execution process starts with the projecting concrete on the slope and the execution of nails with TerraLink technology, followed by formwork, shoring, concreting and decasting the first level of the wall. With this done it is necessary to level the landfill materials to the first level of lashing hooks, placement of the first level of reinforcement and its anchorage and repetition of this process up to the prescribed upper design level.

5.2. Reinforcement of the existing walls

The existing wall is of poor quality and it is necessary to increase its resistance, and reduce its cracking and deformation. This alternative consists on the construction of a reinforced concrete grid resting on the front of the existing retaining wall and to place nails on the horizontal elements to allow better soil behavior (Figure 12). The drainage system remains the same as recommended for the Solution A.

The main advantages of this method are: it needs less space to store materials; it leads to lower displacements due to temperature variations and a lower cost. The main disadvantages are: the great need for formwork; the higher labor force; the longer construction process. The execution process starts with the pickling of the existing wall, with total removal of existing plaster, and coating with mortar reinforced with a carbon fiber mesh, execution of the holes necessary to the execution of the nails; formwork, concreting and decasting the first level of vertical elements and then to the horizontal elements, repeating this process. The nails will be executed at the end.

![Figure 12 – Construction process for the execution of the reinforced concrete grid [9]](image12)
6. Final remarks

The work that was the object of the developed study represented a great challenge in terms of civil engineering, not only for its complexity, but also for its execution in an emergency situation, imposing innumerable technical constraints, difficulties and uncertainties of various natures. Accompanying such a complex work, carried out in an emergency regime in which the project was drawn up in parallel to the execution and in which there were numerous impediments or setbacks, had to be done with great frequency. In addition to the accompaniment of this work, it was also analyzed the design elements of the reinforcement and reconstruction of the wall in question, allowing a broader view on the world of engineering and the different entities responsible for a work.

Due to the uncertainty of the soil geomechanical parameters, a back-analysis was performed that leads to a global unit safety factor leading to new geomechanical parameters. These were used for the analyzed models, leading to conservative results. Considering that 3D finite element calculation models could be explored, it would be interesting to model these solutions in these programs since they lead to results that are closer to the reality.

It is important to monitor the behavior of the work during the exploration phase in order to verify the effectiveness of the adopted solutions. There is even a rule: slope that was operated after signs of instability should not fail to be under observation. As such, it is important to adjust the frequency of piezometer readings and inspections to drainage systems as a function of the seasonality of rainfall to obtain greater control over the possible variation of the ground water level. The accidental rise of the water level, either slowly due to regional transfers or abruptly due to rupture of pipelines, can lead to the collapse or settling of foundations and support structures. Measurements of piezometers and the results of inspection of deep and surface drainage systems should be analyzed together. The values obtained must always be compared with those of the previous readings, in order to analyze the trend of its evolution.

It is advisable to update the geotechnical risk map of the Lisbon area due to its high housing density and seismic risk, in order to avoid the occupation of risky areas or, in the case of areas already occupied, greater control over the behavior of the earth retaining structures and slope stabilization solutions. It is extremely important to monitor these structures in order to prove their effectiveness and to verify if any reinforcement element is deteriorated, requiring maintenance or repair works. These preventive measures must be implemented in order to prevent failure mechanisms and consequently very expensive reinforcement solutions.

References