Proceedings of the XVII ECSMGE-2019
Geotechnical Engineering foundation of the future
ISBN 978-9935-9436-1-3
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doi: 10.32075/17ECSMGE-2019-0523

XVII European Conference
ECSMGE 2019
Reykjavik
on Soil Mechanics
and Geotechnical Engineering

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

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

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

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

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

1 INTRODUCTION

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

Figure 1. Aerial photograph of earth retaining walls

The partial collapse occurred in the central zone of this set of cantilever walls, comprising the walls located behind buildings n. º 108 and n. º106, which experienced severe structural damages result of the walls collapse and consequent landsliding. After the incident the structural stability of the affected buildings was compromised and the overall stability of the remaining earth retaining walls was at risk of imminent collapse, thus the need of intervention was imperious (see Figure 2 and Figure 3).

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

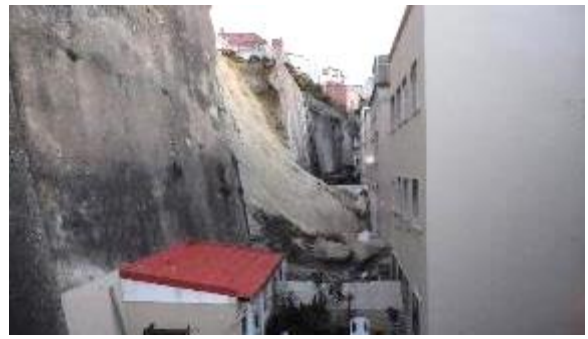

Figure 2. View of earth retaining walls after its partial collapse

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

2 EARTH RETAINING WALLS COLLAPSE CAUSE

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

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

concrete, slightly reinforced, and showing severe corrosion pathologies.

3 URGENT REINFORCED MEASURES

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

Figure 4. View of building n.º108 basement shoring

4 GEOLOGICAL AND GEOTECHNICAL SCENARIO

At the incident area the geological map of Lisbon at a scale of 1:50,000 shows the surface presence of marine sedimentary facies identified as MQB and dating from Miocene period. These materials are mainly composed by sands with some punctual intercalations of clay layers. The MQB layer overlap a limestone unit, identified as MCV. Structurally the geologic units show a monocline with NNE-SSW direction and slope to E-SE, being this structure propitious to the formation of steep slopes.

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

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

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

5 RETAINING WALL INVESTIGATION

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

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

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

Figure 5. Concrete catilever wall samples

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

6 STABILIZATION, RECONSTRUCTION AND REINFORCEMENT SOLUTIONS

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

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

6.1 Solution A

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

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

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

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

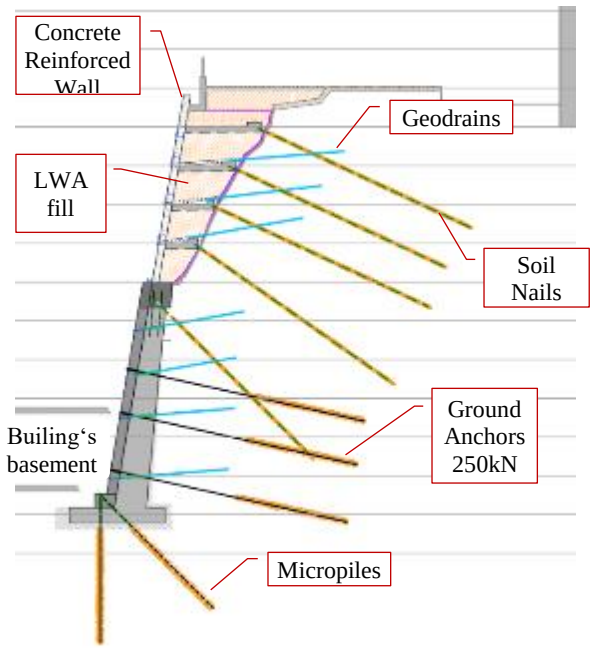

Figure 6. 'Solution A' draft - Cross section

6.2 Solution B

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

Before the implementation of the re-confinement solutions, the existent plaster was removed from the walls face and lined with a high resistance mortar, reinforced with a carbon fibre mesh. Regarding the drainage system, multiple sub horizontal geodrain pipes were installed in order to prevent the accumulation of water behind the wall.

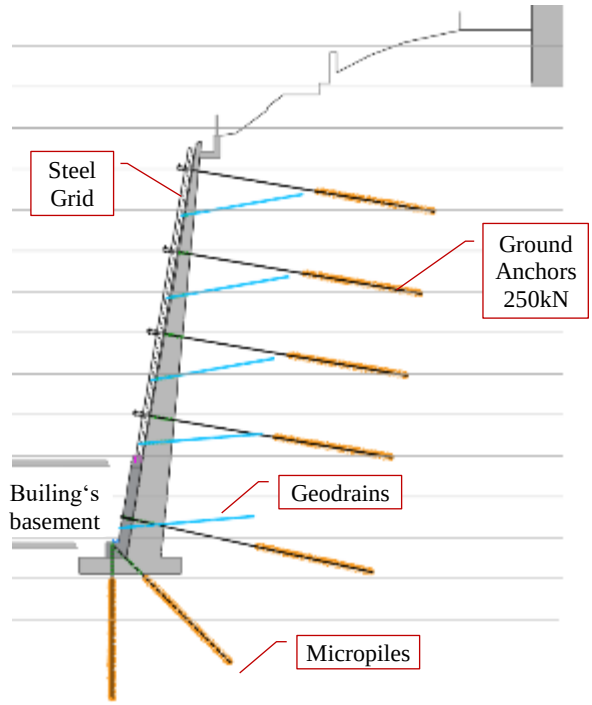

Figure 7. 'Solution B' draft - Cross section

7 SAFETY FACTOR INCREASE DUE TO INTERVENTION

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

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

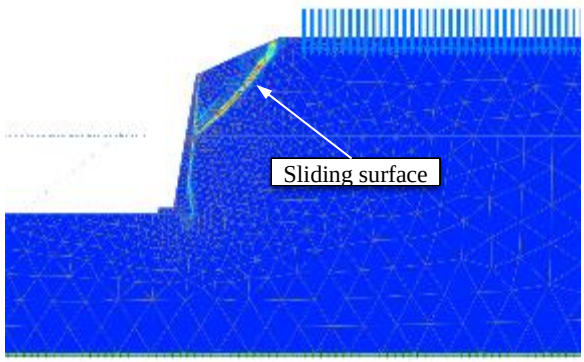

Figure 8. Back analysis - Plaxis 2D output

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

Table 1. Global safety factors

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

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

8 CONSTRUCTIVE CONDITIONS

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

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

Figure 9. Micropile equipment inside building basement

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

Figure 10. Climbing scaffolding and works bellow

9 MONITORING PLAN

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

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

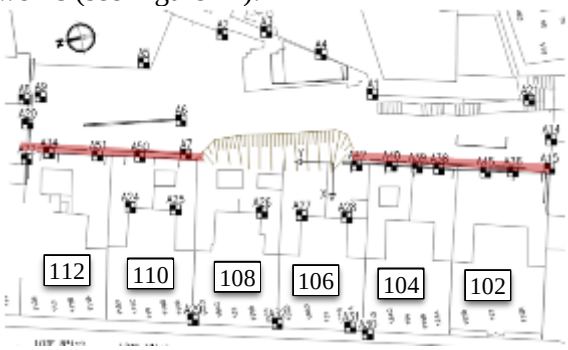

Figure 11. Initial topographic target location on plan

During the reconstruction works the topographic targets were monitored once a week and allowed the site risk management. Since the targets first installed on the walls needed to be replaced during local intervention the replacement targets needed to consider the previous movement history.

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

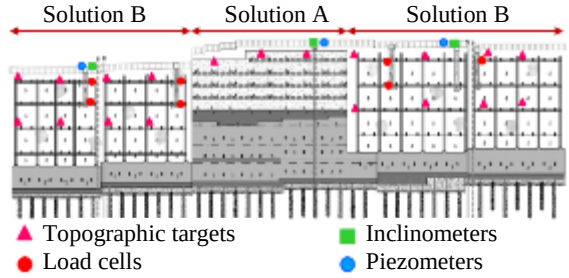

Figure 12. Service life instrumentation and monitoring elevation view

10 FINAL REMARKS

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

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

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

plan is also vital since it will allow the confirmation of the expected behaviour of the retaining structures and reinforcement elements, as well as to assess the need for future interventions.

Figure 13. View after final works

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

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

11 ACKNOWLEDGEMENTS

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

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