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Numerical Study of Slope Stability in an Anchored Wall in Belo Horizonte, Brazil

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Abstract- The present work presents a study on an emergency containment work carried out on one of the main avenues of the capital of Minas Gerais in the year 2018. This structure comprises a curtain anchored by rods installed on a containment of the same nature that was instated as a function of corrosion in the head of the rods due to saturation of the slope. The stability of the problem was analyzed by means of the rigorous equilibrium-limit method (Morgenstern-Price), with the help of GeoStudio 2019 software, SLOPE/W module. Through modeling using finite elements in the Sigma module of GeoStudio, A 2D strain study was carried out for the following "In Situ" scenarios (before reinforcement rods and after the application of the new reinforcement lines). Additionally, the Elastoplastic model was considered representative of the Geotechnical massif. In general, the study returned a significant increase in the safety factor of the slope conferred by the anchored wall and acceptable horizontal displacements, of the order of 1, 5cm.

Keywords- *Anchored Wall, Slope Stability, Numerical Analyses with FEM*

I. INTRODUCTION

Anchored wall [1], [2], [3],[4] is a reinforced concrete structure that receives the tensioning of the anchor to contain lateral earth Force on slopes, restricting the excessive displacement and consequently the rupture of the same ones. The anchor provides additional stiffness and acts as tensile-strength elements that are inserted into the ground. According to [5], this type of anchoring system can be subdivided into drilling and clinging of the hole, mounting of the tie rod, filling the hole with injection of cement grout and injection of the anchored stretch links structural elements to the geotechnical mass. After this process, the anchor is subjected to prestressing.

The brief history of the evolution of the use of this technique is reported by [6]. The use of first anchoring techniques in the world dates from the 1930s at the Cheufras dam in Argelia. In 1960s, temporary and permanent anchorages began to be used on rock and earth massifs for containment purposes. In the late 1970s, many advances in anchoring techniques have been reported such as steel improvement, grout injection into the soil, and the creation of hydraulic jacks used to prestress steel. In 1972, the first standard referring to anchorages was created in Germany [7]. The first works using prestressed anchorages in Brazil refer to the construction of the Rio-Teresópolis Highway in 1957. In 1975, the first Brazilian standard concerning injected and prestressed anchor was launched [8]. In 1980s, there was a great use of tie-down anchorage techniques due to the development of the automobile industry and the great need to construct buildings using underground parking. The anchored wall is a well-known technique in Brazil and is widely used today. In 2018, the [9] is canceled and replaced by [5].

In the mid-70s, so before the first Brazilian standard of anchored wall was thrown, the Nossa Senhora do Carmo´s anchored Wall was building (it´s one largest traffic sections of the capital of Minas Gerais) by the GEOTOP company.

In March 2018, the Civil Defense of Belo Horizonte identified the movement of this anchored Wall.

Due to this problem, an emergency work was carried out which included the installation of a reinforced concrete slab (160m and 308 tie-rod) above the unstable anchored wall and foundation reinforcement (64 piles and blocks).

Because of this historical context, the objective of the present work is to verify the stability of the slope under study by limit equilibrium [10] and to perform numerical simulations by Finite Element Method (*e.g* [11]) considering the natural slope with pathology and the reinforced slope after emergency work.

II. CHARACTERISTCS OF THE WORK

A. Introdution

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The slope under study is located in one of the busiest avenues of Belo Horizonte, MG: Nossa Senhora do Carmo Avenue. at the Belvedere clover, near the Morro do Papagaio community, Santa Luzia Dam, Super Nosso and Treatment plant Morro do Redondo (Fig.1).

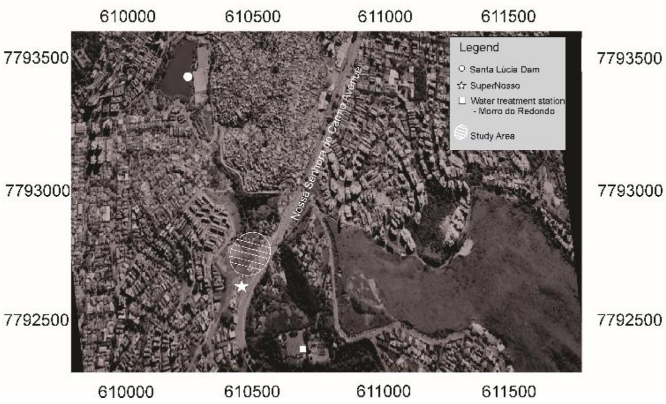

Figure 1. Location study area. [12]

In March 2018, the Civil Defense of Belo Horizonte identified the movement of this anchored Wall (about 10cm) of thrown wall installed in the 70´s in this part the city. The anchored wall was made by GEOTOP and composed by 12 ties (8mm thickness), 2.5m spacing (Fig.2).

Figure 2. Cracks detail on the crest of the slope. (Image courtesy of Thiago Bomjardim Porto)

This movement is accompanied by cracks in the asphalt and repression of the slope crest.

The dignity of the instability of this slope was associated with its saturation promoted by water leakage from the Morro do Redondo Treatment Station (COPASA) and poor surface and underground drainage systems. Due to this saturation, there was corrosion of the prestressed reinforced anchor in the bulb and, consequently loss of performance (loss of prestressing by steel relaxation and loss in the load capacity of the tie-rod.)

To correct this pathology, an emergency work began in April 2018 and lasted for 5 months. During the execution of this emergency work, there was a partial interdiction of traffic in some of the lanes of BR-356, towards Rio de Janeiro.

The main measures adopted to pre-manage the anchored wall´s risk of collapse involved: 1) provisional drainage of the geotechnical massif with the installation of sub-horizontal drains to reduce lateral Earth Force into geotechnical massif; 2) Implementation of techniques for investigation of leakage in COPASA water pipes and consequent repair of them; 3)Implementation of topographic monitoring systems of the retaining wall (total station and geotechnical radar); 4) emergency provisional reinforcement of one panel of the anchored wall with implantation of 3 metal beams anchored in neighboring panels by tie- rods with ϕ47mm with a work load of 833.6KN . After provisional reinforcement of the panel, 8 SPT drillings were performed near the wall and the open cracks in the asphalt were closed (Fig.3).

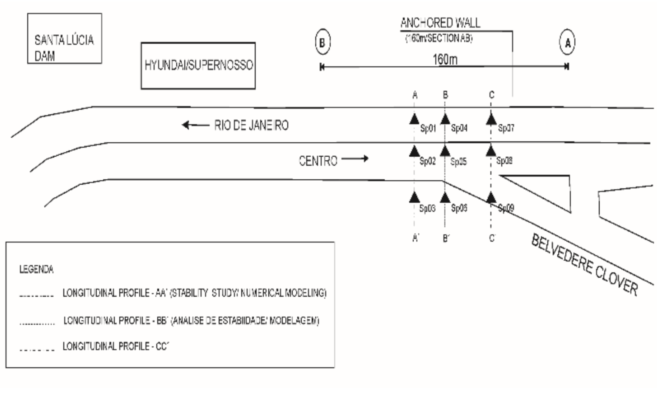

Figure 3. Sketched of anchored wall and geotechnical sections.

Concomitantly with the actions taken above, there was also the removal of families that lived below the wall (Fig.4).

After this emergency stage, a definitive reinforcement was implemented with 308 new tie-rod with a workload of 343.2KN and 64 blocks with two micropiles each (8m in length and 15 cm in diameter). In this stage, there was an injection of cement grout into the massif to improve the resistant properties of the massif (friction angle, cohesion).

Figure 4. Houses in the immediate vicinity of the wired wall.

In order to ensure the durability and efficiency of the definitive restraint wall, were executed: 1) Permanent drainage system consisting of sub-horizontal drains and weep holes; 2) A conventional BSTC ϕ1, 5m with 63m extension and a 140m long bullet tunnel located upstream of the anchored wall for rainwater capitation(Fig.5); 3) Flexural wall for slope containment at the end of the anchored wall; 4) Filling an existing drainage system under the mass that was damaged with the risk of internal rupture (BDTC ϕ1, 0m) by means of compacting concrete injection;

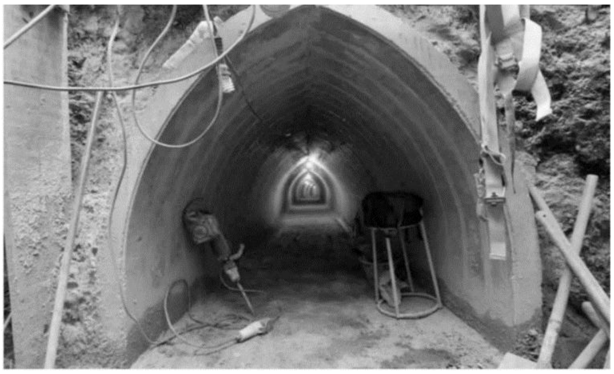

Figure 5. Bullet tunnel(140m in length, 1,6m tall e 1,5m de wide)was intended for rainwater collection. (Estado de Minas, August 2018)

B. *Methodology used for dimensioning anchored wall and permanent staking*

With the purpose of retro-analyzing, the influence of the permanent anchored wall on the stability of the studied slope, is shown below the calculation of this retained structure. The loading capacity of the anchorage of the rods was estimated according to the method of [6], to obtain the anchored stretch (LB), according to equations 1, 2 and 3. While, the free stretch (LL) was obtained based on the recommendations of [13], model presented in Fig. 6. The foundation of the anchored wall was calculated based on the method of [14] and [15], according to equations 4, 5 and 6.

$$q_s = 10K\left[\frac{SPT}{3}+1\right] \quad (1)$$

Where

q_s: Adherence (KN/m²);

K: Coefficient of anchor (Table 2);

N_{SPT}: of medium bulb of anchor;

$$D_s = \beta D_P \quad (2)$$

nde

D_s: Medium diameter of bulb (m);

β : Coefficient of anchor (Table 2);

D_P: Diameter of drill (0,1m)

$$T_l = \pi D_S L_b q_s \quad (3)$$

T_l: Load of bulb (KN);

L_b: anchored stretch (m);

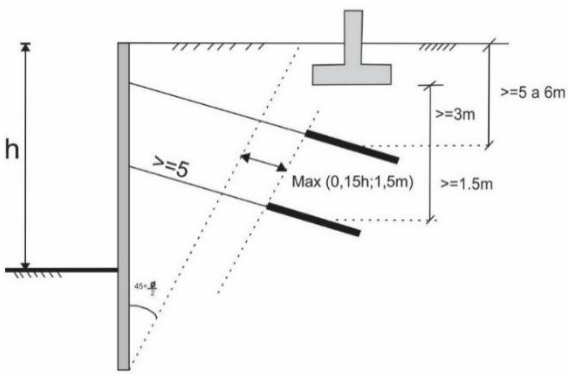

Figure 6. Anchored wall sizing.[13].

$$Q = \frac{\beta Q_l}{1,3} + \frac{\alpha Q_P}{4} \quad (4)$$

$$Q_l = A_l * \left(\frac{\bar{N}}{3}+1\right) \quad (5)$$

$$Q_P = A_p K \bar{N}_P \quad (6)$$

Q: Bearing capacity (KN);

: Lateral strength (KN);

Q_P: Tip strength (KN);

α : Coefficient as function of type of pile;

β : Coefficient as function of type of soil;

$\bar{N}$: Medium NSPT (lateral strength);

N_p : Medium NSPT (tip strength);

C. *Parameterization of massive geotechnical data*

In order to analyze the stability of the slope before, during and after the work, the parameters of the geotechnical massif were estimated. This estimate was made based on two longitudinal profiles performed from eight Standard

Penetration Test (SPT) executed in the study area (Fig.7, Fig.8). Thus, from the top to the base: 1) 0 to 0, 45m of asphalt cover or concrete floor; 2) 1, 25m to 6m of landfill composed of argyl-sandy material with rust canga shafts, brown, soft to hard; 3) 4 to 14m of Silto-Argyl-Sandy landfill (fine sand) to clay-silt with boulders, hard tough, pinkish color; 4) 0 to 1, 15m of residual soil of filito, clayey silt, with little sand, hard rough; 5) Saprolito de Filito, very altered, pink to gray, hard.

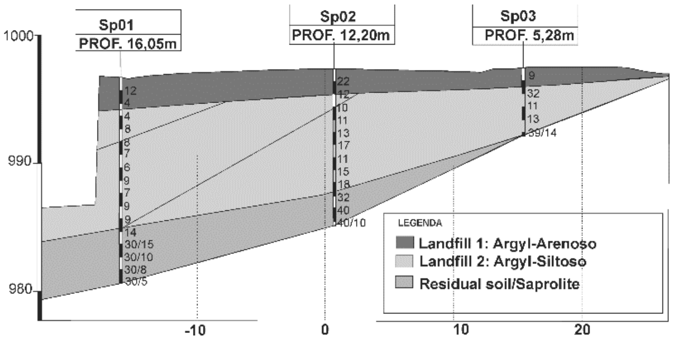

Figure 7. Geological-geotechnical Section AA´ (est 6 + 7,59).

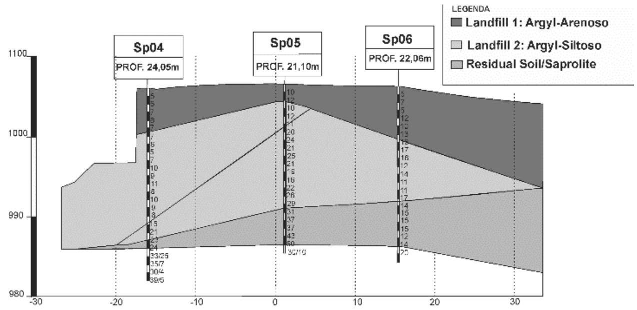

Figure 8. Geológical-geotechnical Section BB´ (est 7 + 10,74).

Based on geological-geotechnical profiles, the values of the geotechnical parameters consolidated for the materials present in the study were presented, namely: specific weight, cohesion, friction angle and modulus of elasticity (table I).

In the development of stress x strain analyses, the elastoplastic model was adopted as a constituent model of the materials.

TABLE I. GEOTECHNICAL PARAMETERS

Soil/Parameters	Table Column Head			
	Specific Weight (kN/m3)	*Friction angle(º)*	*Cohesion (kPa)*	*Modulus of elasticity (MPa)*
Landfill 1: Argyl-Arenoso	19	27	15	20
Landfill 2: Argyl-Siltoso	17	24	14	12
Residual Soil/Saprolite	19	26	11	17
Decomposed Rock	21	29	285	31

D. Stability study via limit equilibrium

The analysis of external stability (generalized global rupture) was performed with the help of computational software GeoStudio 2019, module SLOPE/W, using the rigorous method of Morgenstern-Price. The [16] divides the sliding mass into slices of finite thickness, considers a generic rupture surface and admits that the direction of the resulting interlamellar forces is determined by the use of an arbitrary function, where λ (function factor) must satisfy the balance of forces and moments. According to [5], the safety factor should be greater than 1.5, a conventionally adopted beacon in this type of work.

Another approach that can be adopted for internal stability analysis of an anchored wall considers the possibility of resolution by means of a limit equilibrium with the use of dividing the potential mass in wedges (e.g [17];[18];[19]). However, this approach will not be dealt with in this work.

The study of stability developed had as objective introduced to the result of stability for two sections of the wall-after addition of the new lines of rods, in the condition drained. Since the cadastral survey of the old rods was not available, it was not considered in the studies of this article, although they assist passively in the stability of the massif.

E. Numerical modeling (finite element method)

The application of finite element method is quite useful in predicting the behavior of containment structures, such as anchored wall.

Through a stress x strain study, the safety factors obtained through conventional stability analyses have been supplemented, since they present relevant information regarding vertical and horizontal displacements, strain on the wall, plasticizing areas, among others. It can be noted that although this is a very valuable application, there may be situations in which this approach becomes quite complex (*e.g* Top of the slope with inclination exceeding 30 º; complex geometry of the slope; various loads applied to the slope; occurrence of water; Stratified soil with weaker layers, simulation of constructive processes), as described by [20].

In the scope of this work, 2D numerical modelling was performed using GeoStudio 2019 software (SIGMA modules) for the following "In Situ" scenarios (before reinforcement rods) and during the application of the new reinforcement rods lines.

III. RESULTS

The following presented results were obtained from the new line of tie rods that served as reinforcement for the study at Av. Nossa Senhora do Carmo.

A. Sizind of the tie rods

TABLE II. THE DIMENSIONS OF THE RODS ARE EXPLAINED IN TABLE 02.

Line of Tie-rod	Deph (m)	Dip (º)	Bore Diameter (m)	Tie-rod diameter (m)	Anchored Stretch (m)	Total Stretch(m)
1	1,375	15	0,1	32	8,00	22 m
2	4,125	15	0,1	32	8,00	18 m
3	6,875	15	0,1	32	8,00	14 m
4	9,625	15	0,1	32	8,00	12 m

B. Stability analysis

It is shown below the stability analyses conducted for anchored wall in the AA ´ and BB ´ sections, respectively.

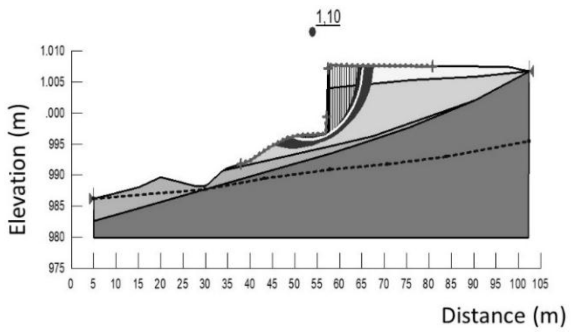

Figure 9. Stabily analysis, without anchored wall- Section AA´..

Through the result of the stability analysis, it is verified that the addition of the reinforcement rods gives a significant improvement in the safety performance to the structure and to the mechanism of external instability of the massif. Such verification is validity by means of the FS value found, which was 1.1 and, after reinforcement, became 1.51 according to the minimum factor recommended by [5] (Fig.9; Fig.10).

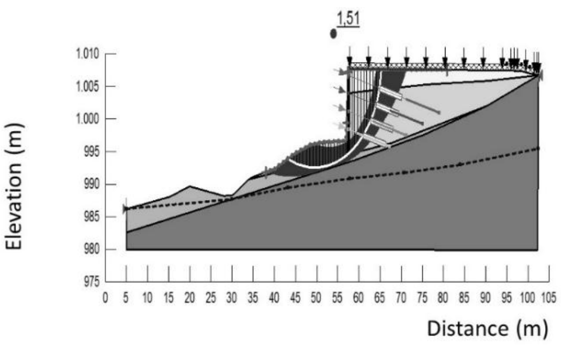

Figure 10. Stabily analysis, with anchored wall- Section AA´..

As verified in the AA' section, once more the result of the stability analysis confirms the conference of improvement in safety performance to the structure as the external instability of the massif, the FS value found was 1.06 and, after reinforcement, passed to 1.46, meeting the minimum factor recommended by [5]. When the result is approximate to 1 decimal place, as presented in the slope Stability Standard [21], Fig.11, Fig.12.

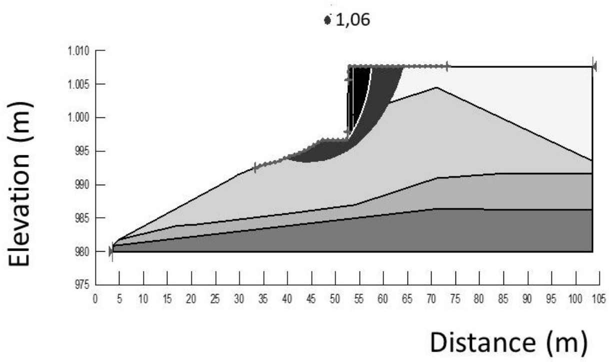

Figure 11. Stabily analysis, without anchored wall- Section BB

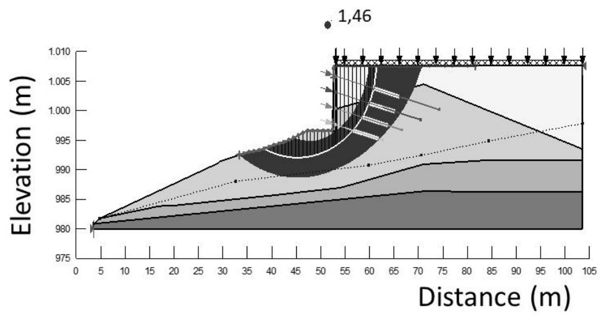

Figure 12. Stabily analysis, with anchored wall- Section BB´..

C. Numerical modeling by Finite element method

1) Seção AA´ (est 6 + 7,59)

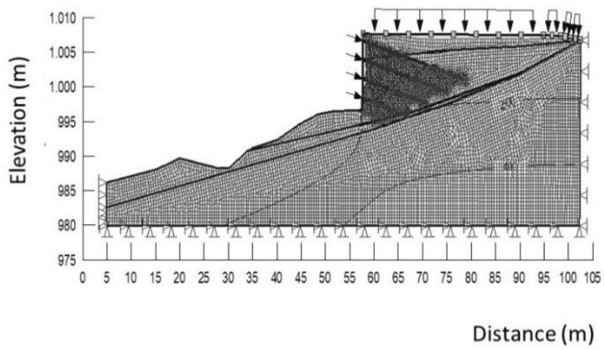

Figure 13. Deformed finite elements mesh containing the representation of the magnified displacements 100 times- Section AA´.

Through the graph of Fig.13, Fig.14, the horizontal displacement predicted on the face of the curtain, after the installation of the rods, is of 1.5 cm – a reasonable value for a structure of this size. The graph of Fig. 15 shows the vertical displacement of the pathway, with an approximate 2.2 cm of movement.

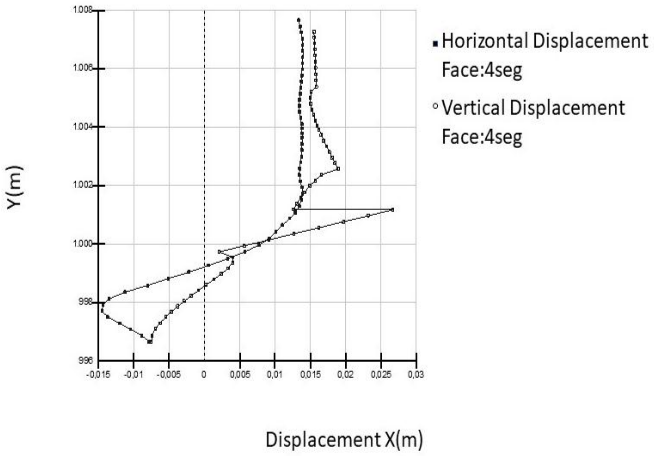

Figure 14. Comparative graph between horizontal displacements on the anchored wall and 1m away from the face, Section AA´.

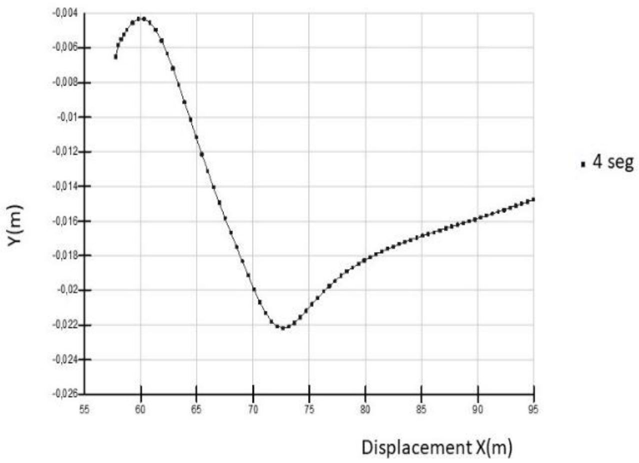

Figure 15. Predicted vertical displacement graphs along the road pavement- Section AA

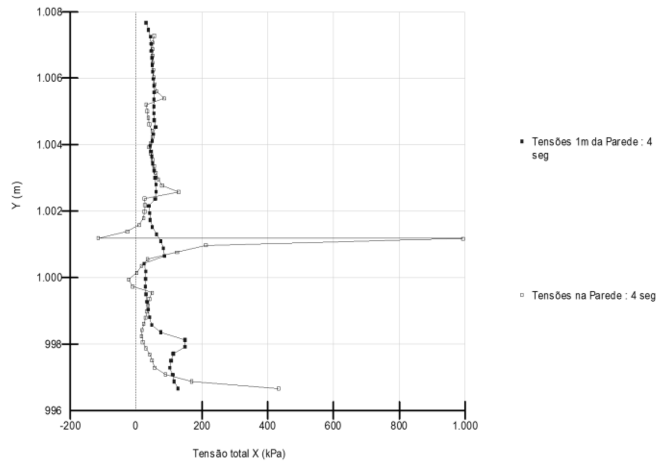

Figure 16. Comparative graph between the acting horizontal tensions on the anchored wall and 1m away from the face - Section AA´.

2) Seção BB (est 7 + 10,74)

The result presented in section AA ´, (Fig.17 e Fig.18), shows that the horizontal displacement predicted on the face of the anchored wall, after the installation of the rods, is of 1.5 cm, reasonable value for a structure of this size. The graph of Fig. 19 shows the vertical displacement of the pathway, with an approximate 2.7 cm of movement.

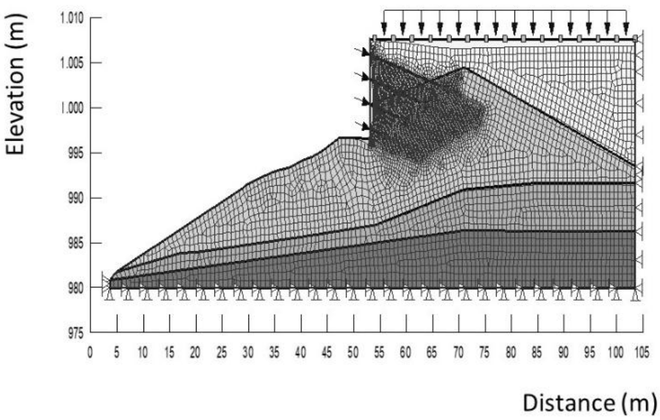

Figure 17. Deformed finite elements mesh containing the representation of the magnified displacements 100 times- Section BB´.

Finally, we have the reference diagram as horizontal tensions acting on the face and 1 m from the face of the thrown curtain (Fig. 16). Note that the average value is around 170 kPa.

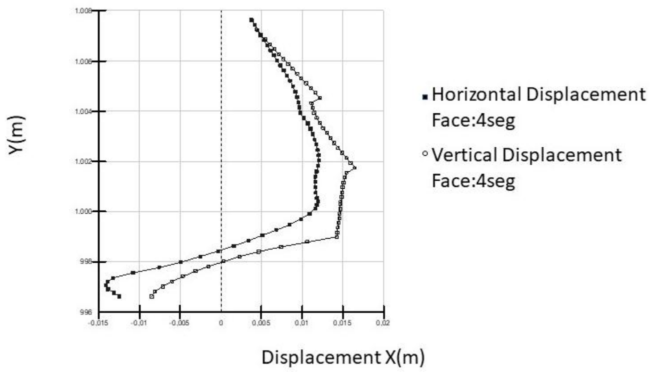

Figure 18. Comparative graph between horizontal displacements on the anchored wall and 1m away from the face, Section BB´.

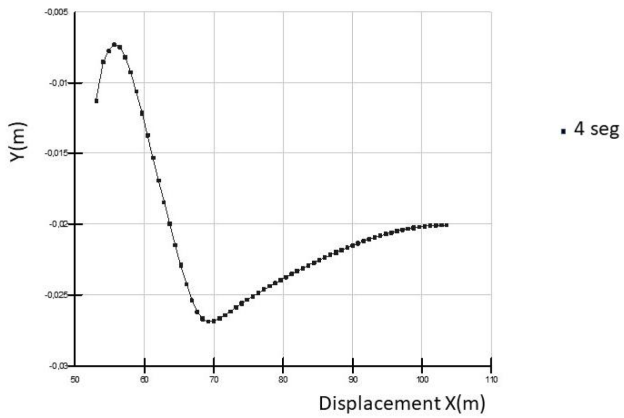

Figure 19. Predicted vertical displacement graphs along the road pavement´, Section BB.

Finally, the diagram refers to the horizontal stresses acting on the face and 1 m from the face of the anchored wall (Fig. 20). It is noted that the average value is around 170 kPa.

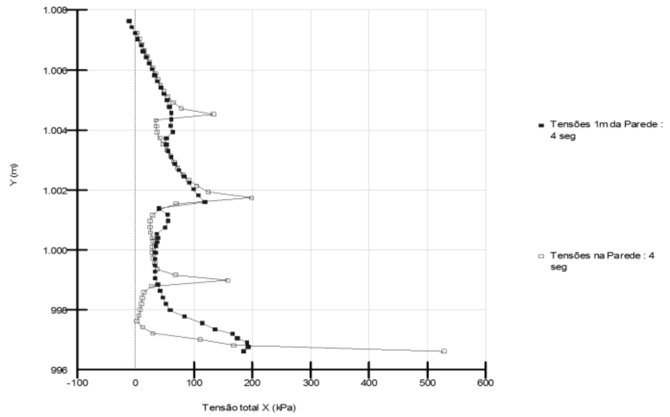

Figure 20. Comparative between the acting horizontal tensions on the anchored wall and 1m away from the face - Section B.

IV. FINAL CONSIDERATION

The stability analysis of the reinforced structure with rods returns a FS that meets the NBR-5629 and NBR 11682. However, it is a reinforcement structure implanted above a curtain of the same nature that was not withdrawn during the work in 2018, thereby understanding that a passive security system potentializes, in practical, the security of reinforcement. Additionally, in this analysis, the increase in geotechnical resistance was not recorded due to the injections of cement grout (cohesion and friction angle).

The use of numerical tools to predict horizontal displacement with reinforced slope returned values of the order of 1,5cm. These values are considered reasonable and inferior to the horizontal displacement (of the Order of 10cm) associated with the formation of cracks on the crest of the slope of Avenida Nossa Senhora do Carmo. This shows the efficiency of the containment dimensioned at 2018.

As mentioned in the [22], all pathologies in shot curtains are critical because failures can cause loss of human lives, compromise the safety of people and vehicles in their surroundings and generate high social and economic costs. Thus it is essential to ensure a good execution of the new works by paying attention to the anti-corrosion protection of the rods due to the finding of soil-steel aggressiveness, to carry out periodic inspections and eventual maintenance over the life of the contentions, Geotechnical triggering if pathologies are found in the rods and perform adequate drainage to the local reality because the water besides generating an extra load in the containment, also facilitates the development of corrosive processes in the steel bars, mainly near the paring

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