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Analysis of Displacements and Stability of a Colluvium in a Tropical Region in Rio de Janeiro, Brazil

Marcos Fábio Porto de Aguiar
Professor, D.Sc., Technological Sciences Center, University of Fortaleza and Department of Civil Construction, Federal Institute of Education, Science and Technology of Ceará,
e-mail: marcosporto@unifor.br

Francisco Chagas da Silva Filho
Professor, D.Sc., Department of Hydraulic and Environmental Engineering, Federal University of Ceará,
e-mail: fchagas@ufc.br

Márcio de Souza Soares de Almeida
Professor, PhD, Alberto Luiz Coimbra Institute of Post-graduation and Research, Federal University of Rio de Janeiro
e-mail: mssal@globo.com

Francisco Heber Lacerda de Oliveira
Professor, M.Sc., Technological Sciences Center, University of Fortaleza
e-mail: heberoliveira@unifor.br

ABSTRACT

This paper presents a study of the movements and stability of natural colluvium slopes with verification of the capability of the Finite Element Method to determine the Safety Factor, taking as an example the situation of Coroa Grande, Rio de Janeiro, Brazil. It was observed that the slope is moving slowly by creeping with a displacement speed strongly influenced by rainfall. Methods applied in the stability analysis showed consistent results, proving that, in periods of intense rainfall, the Safety Factor is very close to unity. Through the results of analysis with FEM, considering maximum shear strains, it is possible to identify the sliding surface and observe the influence of the geometry and the variation on the inclination, the surface of the natural ground slope, and on the formation of the sliding surface.

KEYWORDS: creeping, instrumentation, finite elements.

INTRODUCTION

The study of movements in natural slopes is of great importance, for excessive movements of earthy or rocky masses can, in certain situations, cause major disasters.

The natural slope that is the object of this work, is located on the coastal hillside of Serra do Mar, in Coroa Grande, Itaguaí county, in Rio de Janeiro, Brazil. At the foot of the hill is highway BR 101. There is a Petrobrás oil pipeline in that area, which carries oil from a terminal located near the city of Angra dos Reis to a terminal located in Guanabara Bay, as shown in Figure 1. Thus, it is quite important to know and monitor the hillside movements in order to take appropriate measures against impending landslides that could endanger people who live or travel in the region, and harm the environment.

The average inclination of the instrumented slope area is of approximately 17%, the subsoil being made of two soil layers over rock: the first and most superficial layer is colluvium, consisting of clayey sand with gravel, while the second layer of residual soil is silty-sandy clay with the presence of gravel. The deep bedrock is composed of sound gneiss rock, altered in some nearby parts of the residual soil.

This article presents an observational analysis of the slope displacement from instrumentation data obtained between 1986 and 2004. It also presents stability analyses carried out not only by limit equilibrium methods, but also through the finite element method. These analyses were carried out based on parameters obtained in tests on Denison-type samples taken from depths close to the sliding area. These samples made it possible to conduct (Freitas, 2004) characterization tests on direct shear and torsion.

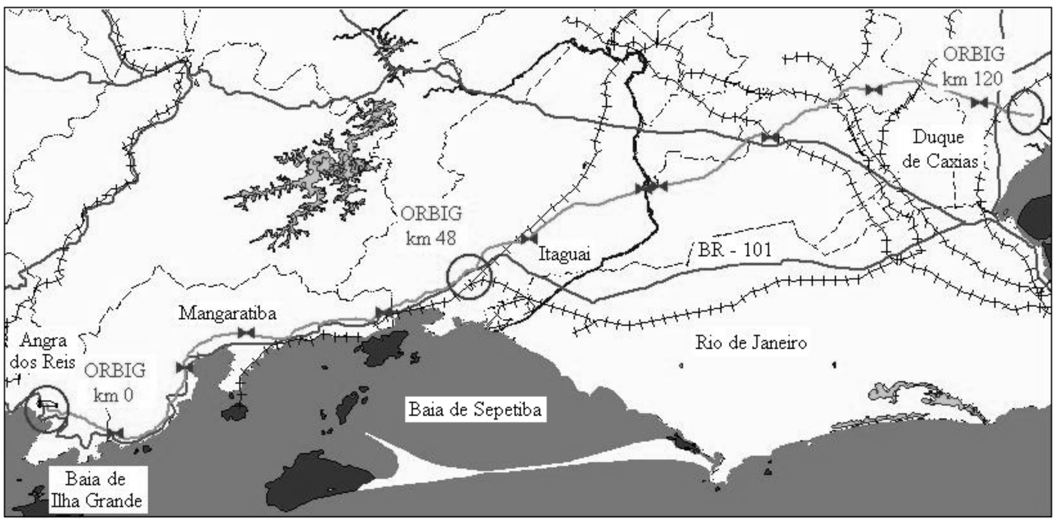

Figure 1: Location of the study area, of Rio-Ilha Grande Bay Oil Pipelines (ORBIG), and highway BR 101.

OBSERVATIONAL ANALYSIS OF MOVEMENTS

An observational analysis of movements was conducted on Coroa Grande hillside through instrumentation performed from 1986 to 1999 and from 2000 to 2004. Figure 2 shows the instrumented areas of the slope. The instrumentation considered in the study, from 1986 to 1999, was composed of 7 inclinometers, 1 Casagrande-type piezometer and 1 water level meter, arranged as shown in Figure 3. Table 1 shows the periods of monitoring of each instrument reviewed here.

The rainfall data used here were provided by the State Superintendent of Rivers and Lakes of the State of Rio de Janeiro (SERLA-RJ) for the rain gauge stations of Mendanha and Santa Cruz, the closest to the instrumented area and located about 50 km away.

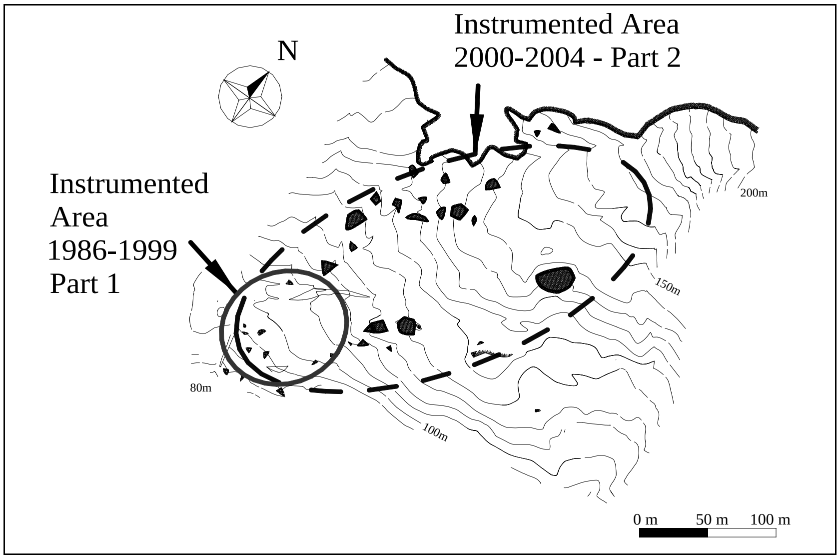

Figure 2: Study area with indication of the instrumented areas in periods from 1986 to 1999 and from 2000 to 2004.

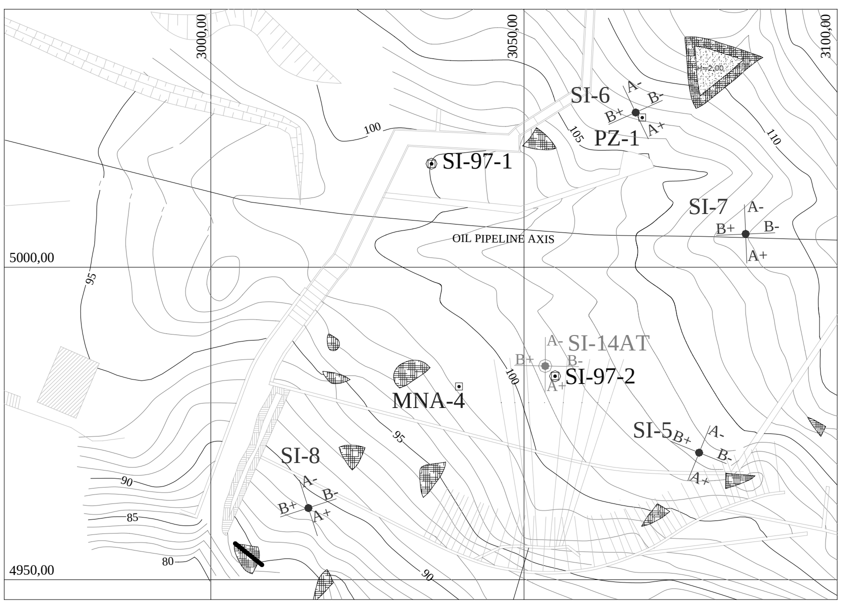

Figure 3: Instrumentation location.

Table 1: Instrumentation considered in the research

Instrument	Period
Inclinometer SI-5	October 1986 to June 1992
Inclinometer SI-6	June 1988 to July 1991
Inclinometer SI-7	June 1988 to June 1992
Inclinometer SI-8	June 1988 to April 1991
Inclinometer SI-14AT	October 1993 to May 1995
Inclinometer SI-97-1	January 1998 to August 1999
Inclinometer SI-97-2	January 1998 to August 1999
Piezometer PZ-01	June 1988 to May 1991
Water level meter MNA-4	June 1988 to May 1991

Piezometers and water level meters

The influence of rainfall was verified in variations measured by the water level meters and piezometers. Figure 4 shows the results of water level measurements and rainfall related with time. Similarly to that reported by Brugger et al. (1997) also for slopes on Serra do Mar, there is a relationship between rainfall, piezometric and groundwater levels, as well as horizontal displacement speed on the slope. It was also verified, in most situations, that the piezometric load and water level values increased with the increasing amount of rain.

Comparisons between data from the rainfall station installed at Coroa Grande from June 18th 2003 to April 10th 2004 (Freitas, 2004) indicate precipitation values in the study site that were twice as high as those found in stations located in Sepetiba and in Campo Grande in the same period. Additional data from 1998 to 2000 (Feijó et al., 2001) show an approximate amount of rain at the Mendanha station that was 30% higher than at Campo Grande and Sepetiba stations. Considering that Mendanha station has the greatest amount of rainfall data available in this study, from 1976 to 2000, it should be considered, through relationships verified by the authors, that in Coroa Grande the amount of rainfall is 50% higher than that in Mendanha.

When considering the relationship between the results of the displacement speed of inclinometers and the amounts of rainfall recorded at Mendanha and Santa Cruz stations, it can be verified that the rainfall peaks are close to the displacement speed peaks on the slope, as will be shown in the analysis of the inclinometer results.

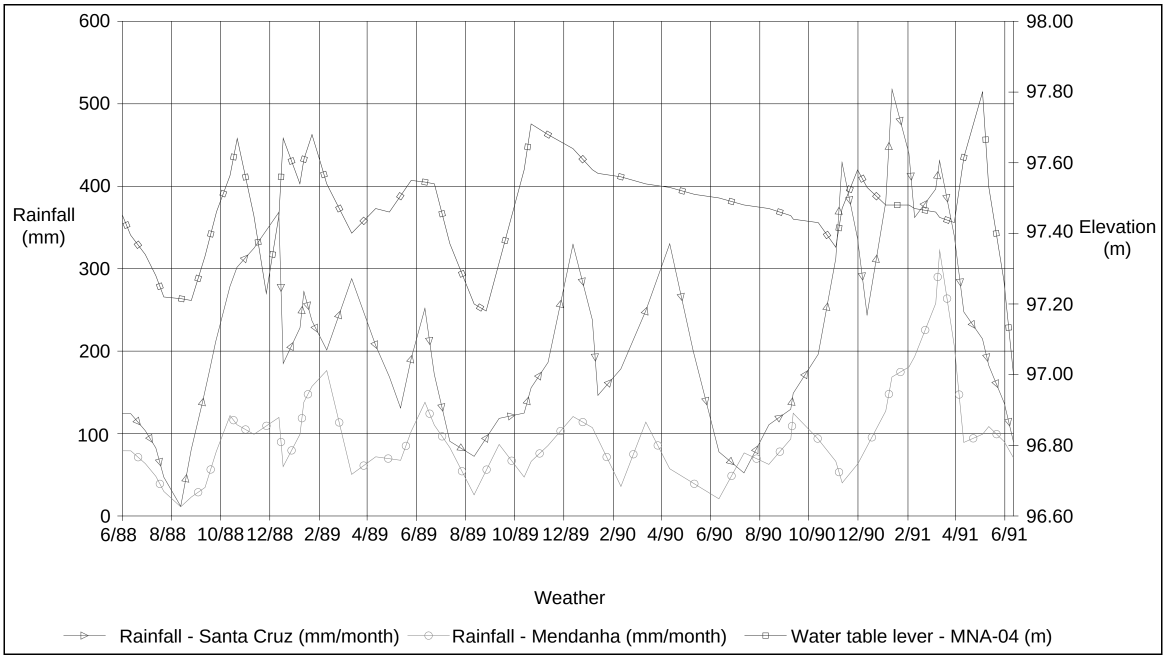

Figure 4: Groundwater level and monthly rainfall at Mendanha and Santa Cruz stations from June 1988 to June 1991.

Inclinometers

With the instrumentation results obtained with inclinometers, the sliding surface was identified and, through the displacement speed, it was possible to classify the movement and analyse the stability of Coroa Grande slope.

Sliding Surface

To reach the critical sliding surface, from the results of inclinometers, the depth equivalent to the maximum distortion was identified for each inclinometer profile, as shown in Figures 5 and 6 for inclinometers SI-6 and SI-8. According to the instrumentation results in the region, throughout the monitoring period, it appears that the critical or sliding surface was verified at different depths, ranging from 4.5m to 11.5m in relation to the ground surface level. Although several critical depths were found at different inclinometers, there is no evidence of two sliding surfaces in the same horizontal displacement profile. We assume, then, that it is the same surface, that can be viewed through a graphical representation in three dimensions or in a topographic plan, by drawing the contour lines from quotas obtained from results taken from inclinometers. Based on the topographic plan of the sliding surface, section M1-M1 was drawn (Figure 7) using the quotas in that section. The sliding surface recorded in the period is located in the saturated soil even in the dry season, and is very close to the contact of the colluvium soil layer with the residual soil layer (Figure 8).

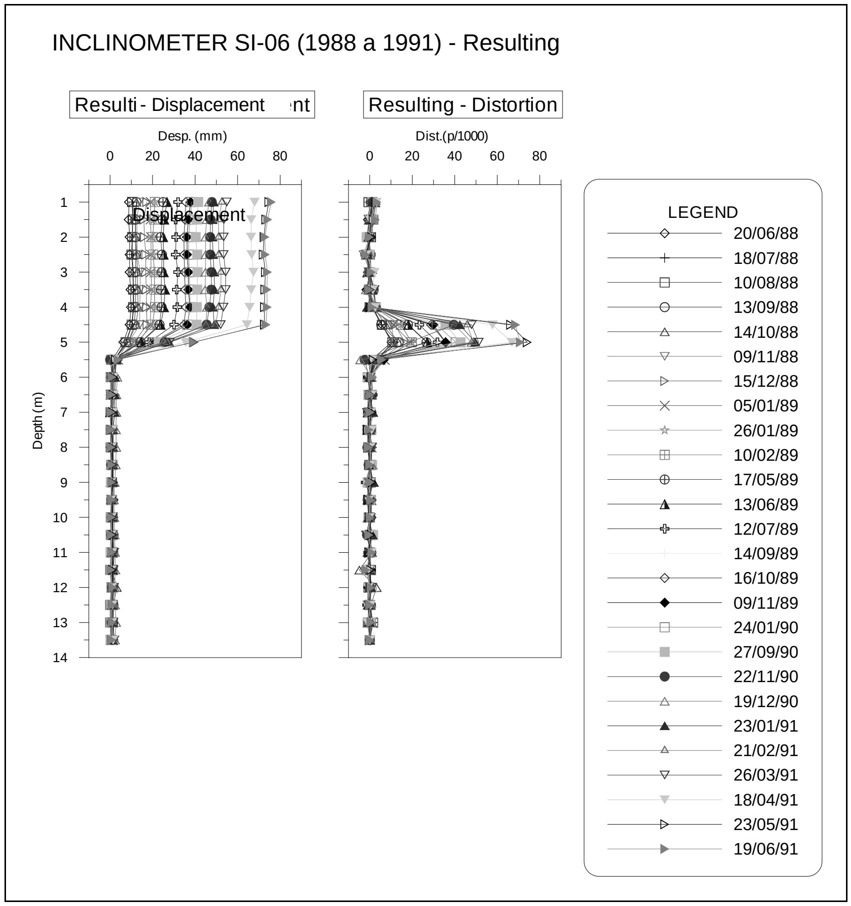

Figure 5: Graphical representation of displacements and distortions recorded at SI-6.

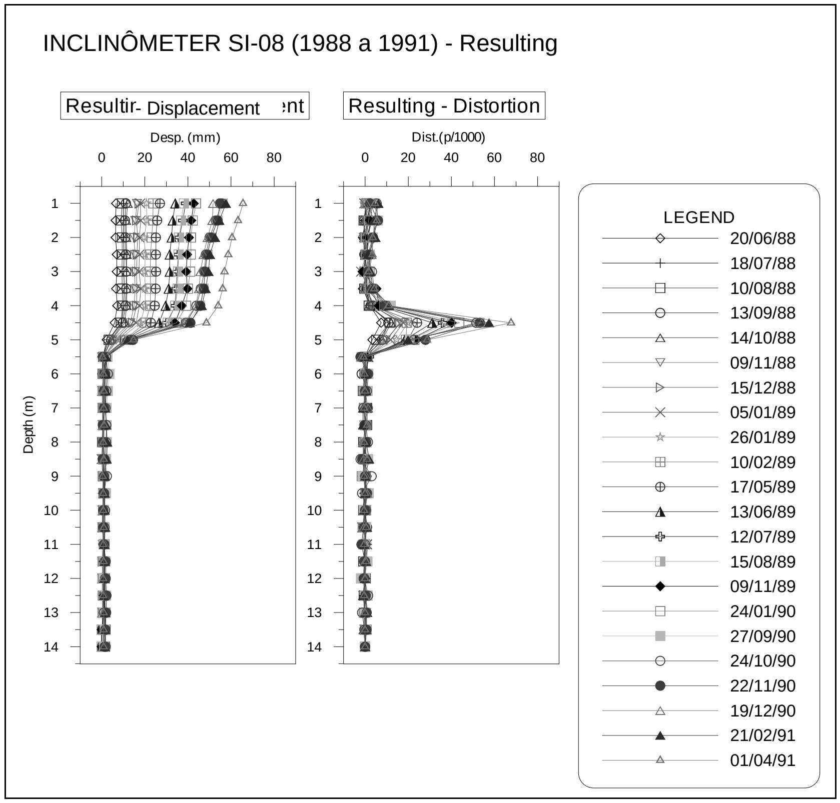

Figure 6: Graphical representation of displacements and distortions recorded at SI-8.

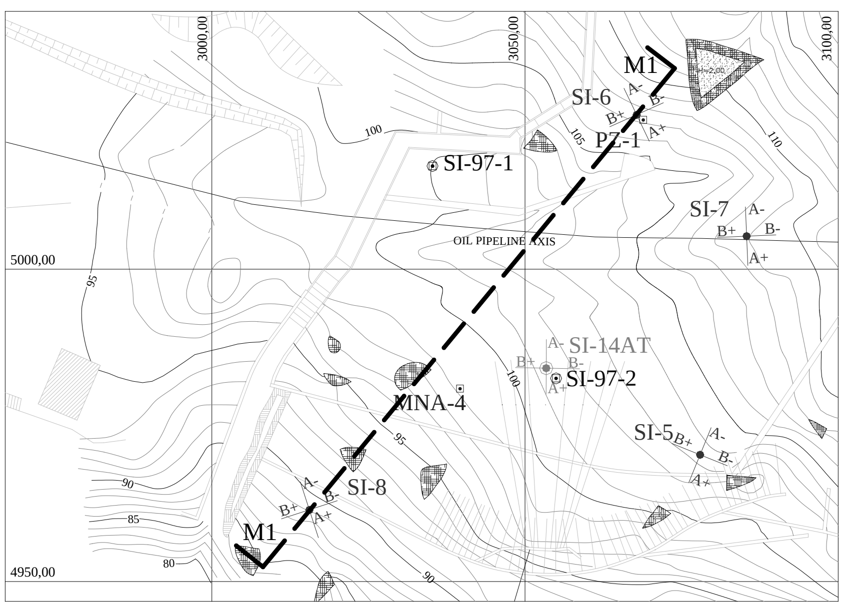

Figure 7: Instrumentation location and section M1-M1.

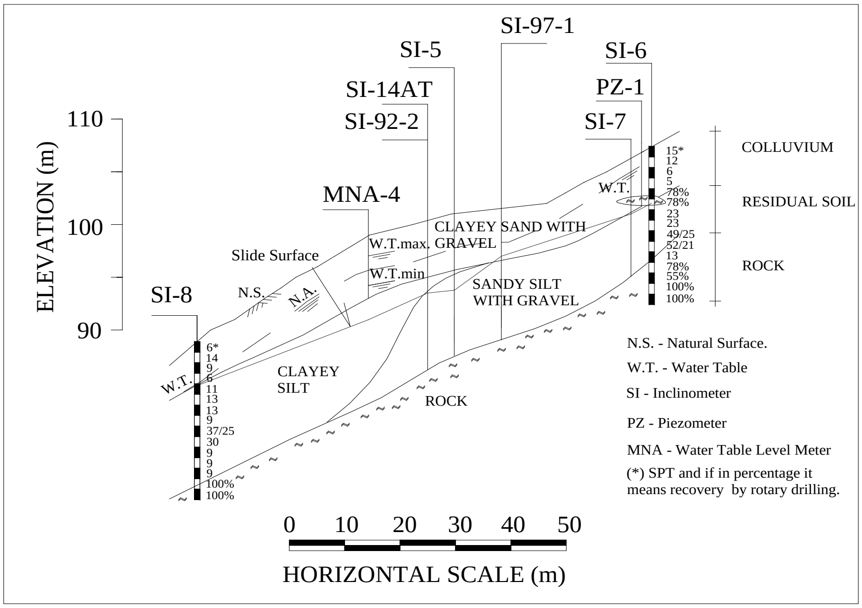

Figure 8: Section M1-M1 of the instrumented region.

Movement Classification

The study on the displacement speed in the period from November 1986 to August 1999 showed that the movement that occurred in the study region ranges from very slow to extremely slow (Cruden and Varner, 1996). Analyzing each interval between measurements individually, it was observed, even in the worst situation, that the soil mass movement is by creep (Terzaghi, 1950). However, considering the entire observation period, there are variations in the horizontal displacement speed, which, according to what is presented here, are influenced by rainfall. Therefore, it is concluded that the soil mass moves by creeping. Significant accelerations were recorded for quotas of groundwater levels observed in MNA-4, above 97.05 m.

Despite changes verified in the horizontal displacement speed, in the situations studied, the behaviour of the "horizontal displacement speed vs. time" curves and the speed levels achieved indicated no trends of rupture (Figures 9 and 10). The graphs showed that the speed had a small oscillation around an average equivalent to 0.05 mm/day. Accelerations of the displacement speed are checked only during rainy seasons, reaching a maximum of 0.66 mm/day in November 1998.

The movement observed in the case shown is, considering its worst situation, of low destructive power (Cruden and Varner, 1996). Some permanent structures may remain intact during the movement. Proven cases show situations of movements within the same speed range and without significant damage (Turner and Schuster, 1996). According to Hunt (1997), for cases of residual and colluvium soils, speeds from 2 to 5 cm/day, during periods of heavy rain, indicate imminent collapse.

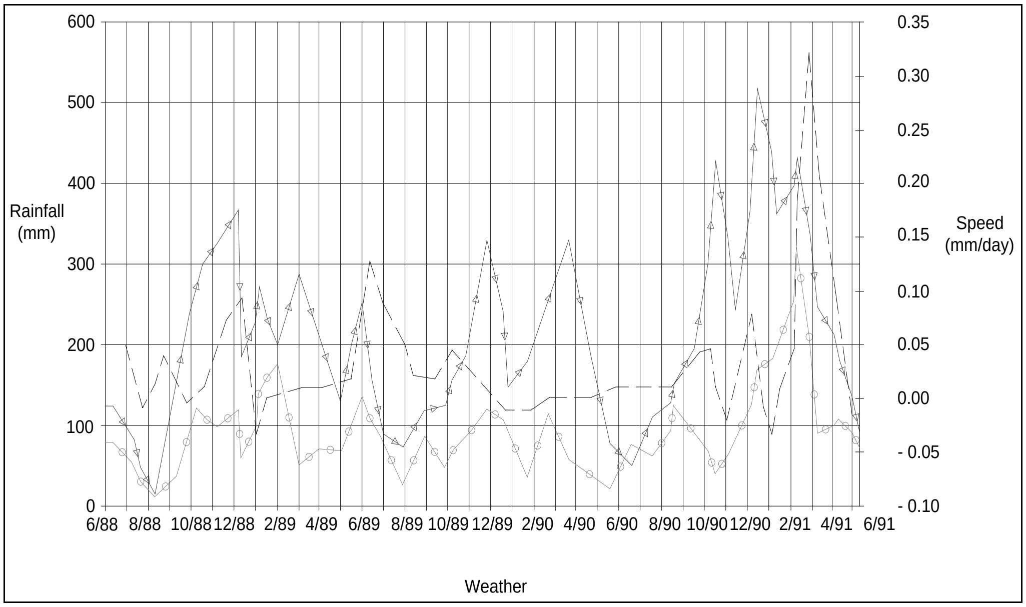

Figure 9: Displacement speed and rainfall from June 1988 to June 1991, at inclinometer SI-6.

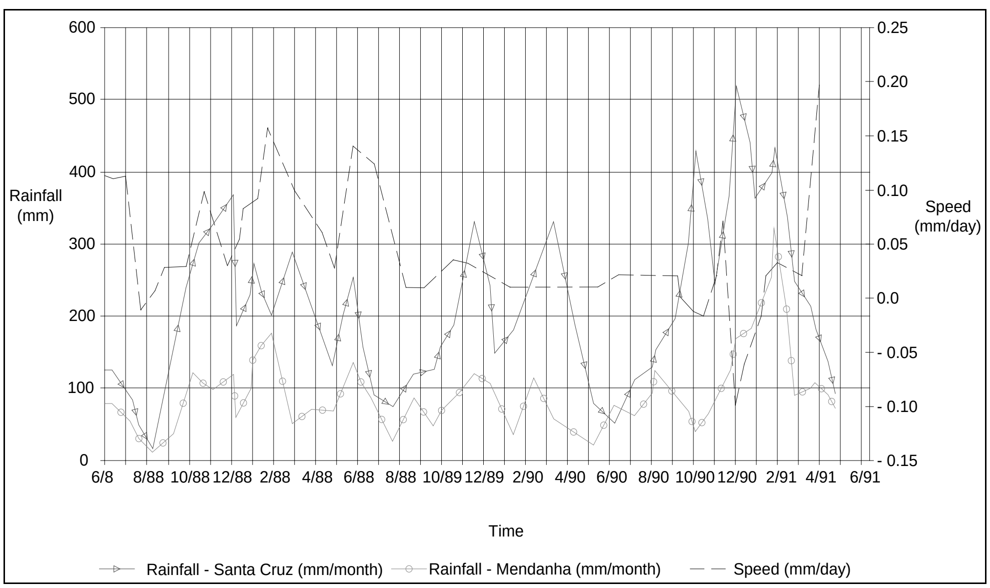

Figure 10: Displacement speed and rainfall from June 1988 to June 1991, at inclinometer SI-8.

STABILITY ANALYSIS

The stability analyses were performed by limit equilibrium methods – Spencer (1967) and finite elements. Two techniques were used in association with FEM: the Technique of Stress with Defined Sliding Surface (Brown and King, 1966) and the Technique for the Reduction of Shear Strength (Griffiths and Lane, 1999).

Method of Spencer

The limit equilibrium method used here was proposed by Spencer (1967), and considers that the slopes of lateral forces are the same for any slice and the slope of the lateral force is calculated by a procedure that satisfies all the equilibrium conditions that are applicable to any form of sliding surface. Slope/W, a software program from the GeoStudio package (2004), was used in the analyses.

Four sections were used: three for the first instrumentation period, A1-A1, B1-B1 and C1-C1, from 1986 to 1999, and one for the second, from 2000 to 2004, A2-A2 (Figure 11). Based on the instrumentation results, groundwater levels and the sliding surface will be considered. The water levels used in the analysis were the minimum observed – the critical level – from which there is acceleration of movement and the maximum level.

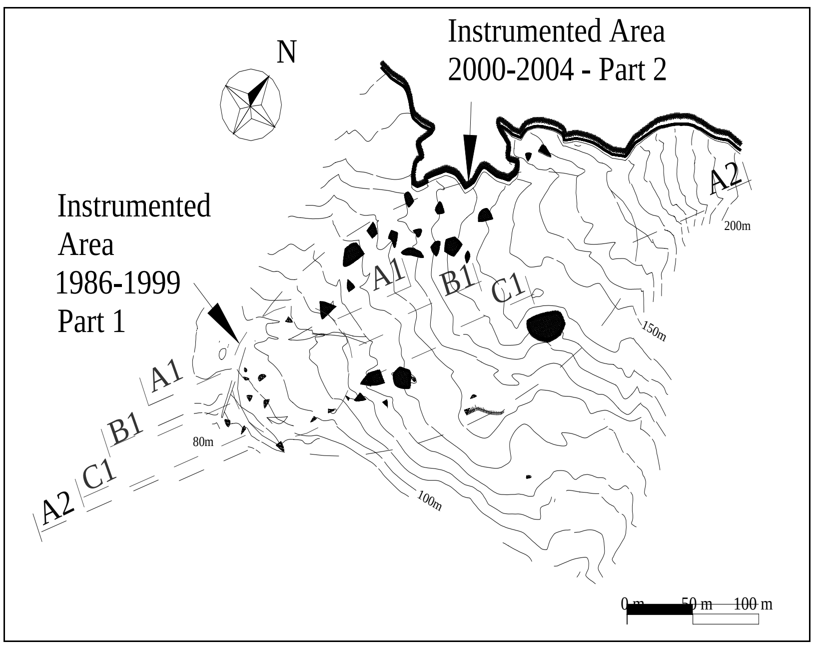

Figure 11: Study region indicating the instrumented areas and sections for the stability analysis.

The parameters obtained from characterization tests and shear strength – direct and shearing by torsion (Freitas, 2004) – are presented in Table 2.

The results for the three specific sections, A1-A1, B1-B1 and C1-C1, for the first period from 1986 to 1999 and section A2-A2 for the second period, from 2000 to 2004, are presented in Tables 3 and 4, respectively, for peak resistance parameters obtained from direct shearing tests and for residual strength parameters obtained from ring shearing torsion tests. According to the safety factors results, it is observed that their values, when residual strength parameters for the situation of maximum water level are used, are very close to unity, and that when peak strength parameters are used, regardless of the position of the groundwater level, the stability of the slope is confirmed. The fact of proving movements through instrumentation and observation in loco of cracks in several areas of Coroa Grande slope indicates that the residual strength parameters should reproduce results that are closer to reality. For other analyses, only the residual strength parameters will be considered.

Table 2: Results, expressed in average values, of characterization test and shearing strength of samples from the study area (adapted from Freitas, 2004) – Depths (m) 9.45 to 16.57.

Results of the characterization test							
Gravel (%)	Sand (%)	Silt (%)	Clay (%)	G_s	w_L (%)	w_P (%)	I_P (%)
2.12	66.86	24.89	6.24	2.72	-	-	-
Results of the direct shear test							
e_0	S_o	w_i	c' (kPa)	ϕ' (º)	γ (kN/m³)		
0.92	96.16	33.51	16.00	29.00	18.00		
Ring shear							
e_0	S_o	w_i	c' (kPa)	ϕ' (º)	γ (kN/m³)		
1.20	91.46	40.09	2.50	19.00	18.00		

Table 3: Safety factors from stability analysis assuming peak resistance parameters.

Water level situation	Safety factor values – Spencer method	
	1986-1999	2000-2004
	A1-A1, B1-B1 and C1-C1	A2-A2
Minimum WT	2.92 – 3.09	2.19
Critical WT	2.46 – 2.58	2.09
Maximum WT	2.24 – 2.44	2.04

Table 4: Safety factors from the analysis of stability parameters assuming residual strength

Water level situation	Safety factor values – Spencer method	
	1986-1999	2000-2004
	A1-A1, B1-B1 e C1-C1	A2-A2
Minimum WT	1.44 – 1.56	1.25
Critical WT	1.15 – 1.25	1.12
Maximum WT	1.02 – 1.16	1.09

Finite Elements Method

The Finite Element Method, which is frequently used in determining stresses and strains in structures, can also be used to determine the slope stability condition by calculating safety factors values.

From the 1960s, with the development of the Finite Element Method, studies with the goal of applying it to analyse slope stability started to be conducted. The FEM will be employed here in the generation of pore pressure in the massif sections and in the stability analysis using two different techniques: the Technique of Stress with Defined Sliding Surface (Brown and King, 1966) and the Shearing Strength Reduction Technique (Griffiths and Lane, 1999).

Technique of Stress with Defined Sliding Surface – TSDSS

Brown and King (1966) have already considered that, if the stress field in a landfill is properly configured, the sliding surface can be drawn and the stability condition given. This simple procedure is used in the Stress with Defined Sliding Surface method, which is considered as an indirect method or of improved limit equilibrium. Thus, from the stress state in the soil mass obtained by the Finite Elements Method and with the sliding surface data, the stability condition of a slope can be defined, with the surface defined by demand, or adopted from results of field instrumentation, as in the case of Coroa Grande.

The model used represents the situation in a quite simple way; however, it should meet the needs of the analysis. Therefore, the elastic parameters were considered based on the type of material and the average results of penetration rates of SPT-type surveys carried out on Coroa Grande hillside. The elasticity modulus was estimated according to the upper band limit (Lopes et al., 1994):

$$1{,}5N(SPT) < E'[MN/m^2] < 2N(SPT) \quad (1)$$

Considering characterization parameters, the soil can be considered as sandy. Thus, the thrust coefficient at rest (K0) can be obtained, according to Jaky (1944), for soils that are usually dense, depending on the soil internal friction angle (ϕ'):

$$K_{0,nc} = 1 - \sin\phi' \quad (2)$$

According to the elasticity theory, the effective Poisson's ratio (ν') can be estimated through the following relation:

$$\nu' = \frac{K_0}{1 + K_0} \quad [3]$$

After the generation of pore pressures and considering the same mesh with eight-node quadrilateral elements and groundwater level situations, the state of total and effective stresses for the slope was determined for different positions of the groundwater table in the monitoring period studied. The contour conditions and elastic parameters, permeability and shearing strength for section A2-A2, are shown in Figure 12 and the finite elements mesh used in Figure 13.

The results for section B1-B1 of the first period, from 1986 to 1999 and for section A2-A2 of the second period, from 2000 to 2004, for residual strength situations are shown in Table 5. The Seep/W, Sigma/W and Slope/W software programs of the GeoStudio package (2004) were used, respectively, to obtain the pore pressure, stress state on the massif and safety factor.

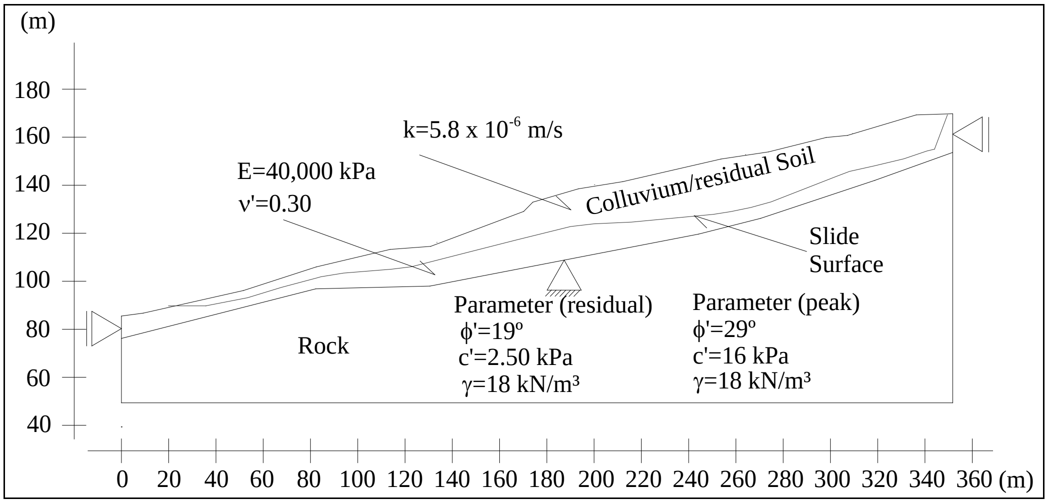

Figure 12: Section A2-A2 with contour conditions and soil parameters.

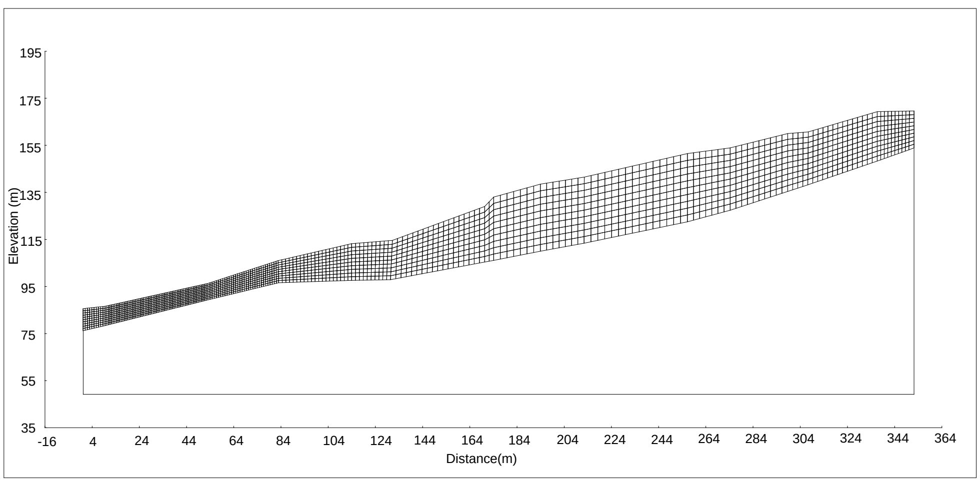

Figure 13: Finite element mesh of Section A2-A2.

Table 5: Safety factor values from stability analysis assuming residual strength parameters.

Water level status	FEM – TSDSS Safety factor values	
	1986-1999	2000-2004
	B1-B1	A2-A2
Minimum WT	1.45	1.37
Critical WT	1.18	1.18
Maximum WT	1.08	1.09

Shearing Strength Reduction Technique – SSRT

In this technique, the finite elements model is directly used to locate the critical sliding surface in the soil mass and to determine the safety factor. This is accomplished through collapse simulation with the progressive reduction of the soil shearing strength parameters. The view of the slope rupture is verified through zones in which the shearing strength is insufficient to withstand the shearing stresses.

The Griffiths and Lane (1999) technique considers the plane strain analysis of soils with elastic–plastic behaviour and the Mohr-Coulomb rupture criterion. The soil is initially assumed as elastic and the model generates normal and shearing stresses in all Gauss points within the mesh. These strains are then compared with the Mohr-Coulomb rupture criterion. If the stresses at a particular Gauss point are located within the rupture field, then this region remains elastic. If stresses are located on or outside the Mohr-Coulomb rupture field, then this region is yielding. The stresses are redistributed across the network using the viscous–plastic algorithm (Zienkiewicz, Humpheson and Lewis, 1975). The shearing rupture occurs when a sufficient number of Gauss points have drained to allow the development of the mechanism. At the rupture, shearing deformations develop from the base to the top of the slope.

The critical sliding surface can be viewed through the distribution of shearing deformations on the massif, in addition to which several laboratory tests show the zone of maximum shearing strains at the ruptures, which coincides with the shearing or sliding surface. Therefore, it is considered that the slope rupture mechanism is directly related with the development of shearing deformations in the Shear Strength Reduction Technique (Matsui and San, 1992).

To meet analysis conditions, according to the method used, it was necessary to complement the geometry, reducing differences in results due to the influence of contour conditions imposed in the supports. Unlike previous methods, where the sliding surface and the soil parameters in this area were crucial for the analysis, more detailing of the soil mass is necessary in order to use the Shearing Strength Reduction Technique. For this, two soil layers and the rock were considered, shaping the field situation with greater realism. The side supports restrict the movement in both directions and are distant from the section studied, so that they do not interfere significantly with the results (Figure 14).

Table 6 shows the effective shear strength parameters of materials used in the analysis. The values were obtained from specific literature, based on laboratory tests of the studied region, except for the rock parameters, which followed the recommendations of Goodman (1989). The six-node triangular mesh used in section A2-A2 is shown in Figure 15. The distribution of pore pressure in section A2-A2 by the maximum water level is in Figure 16.

Table 6: Parameters considered in the analysis with FEM-SSRT

Parameters	Material		
	Colluvium	Residual soil	Rock
k (m/s)	5.80E-06	5.80E-06	1.00E-08
E’ (MPa)	20	60	400
ν'	0.3	0.3	0.3
γ (kN/m³)	18.0	18.0	21.0
Cohesion – peak (kPa)	16.0	33.0	140.0
Cohesion – residual (kPa)	2.5	2.5	140.0
Friction angle – peak (º)	29.0	38.0	22.4
Friction angle – residual (º)	19.0	19.0	-

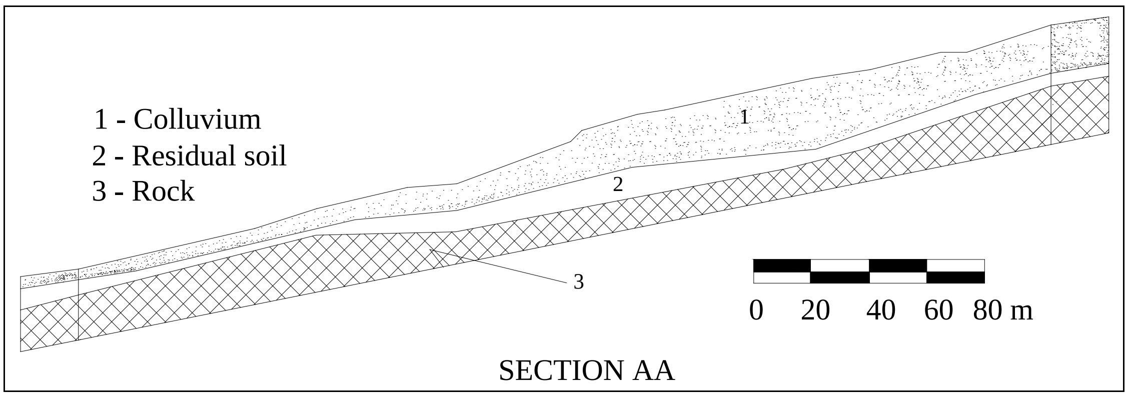

Figure 14: Section A2-A2 considered in the analysis with FEM-SSRT.

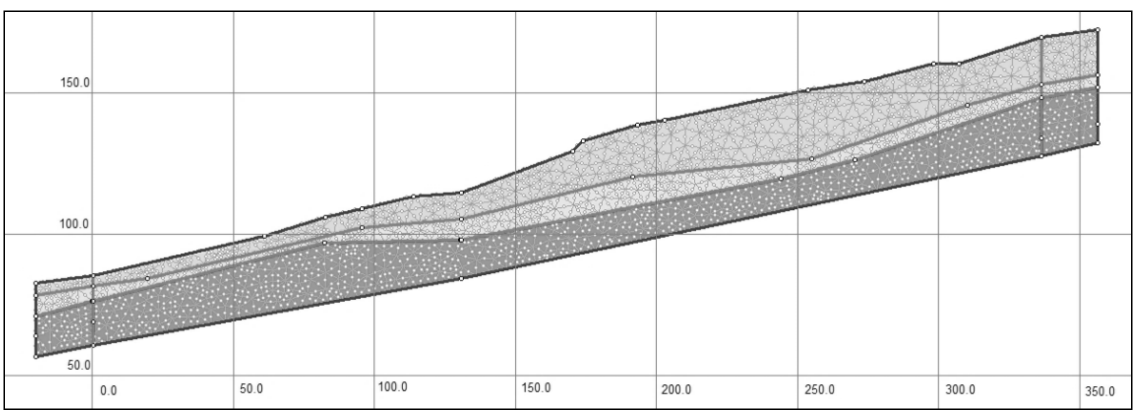

Figure 15: Mesh of section A2-A2 with six-node triangular elements used in the analysis.

The results for section B1-B1 of the first period, from 1986 to 1999, and for section A2-A2 of the second period, from 2000 to 2004, are presented in Table 7. In the same analysis, using this technique, peak and residual shearing strength parameters are considered. Phase ² software (2005) was used for the stability analysis with the Shearing Strength Reduction Technique.

Table 7: Safety factor values from stability analysis assuming residual strength parameters.

Water level	FEM – SSRT Safety factor values	
	1986-1999	2000-2004
	B1-B1	A2-A2
Minimum WT	1.54	1.22
Critical WT	1.22	1.07
Maximum WT	1.11	1.03

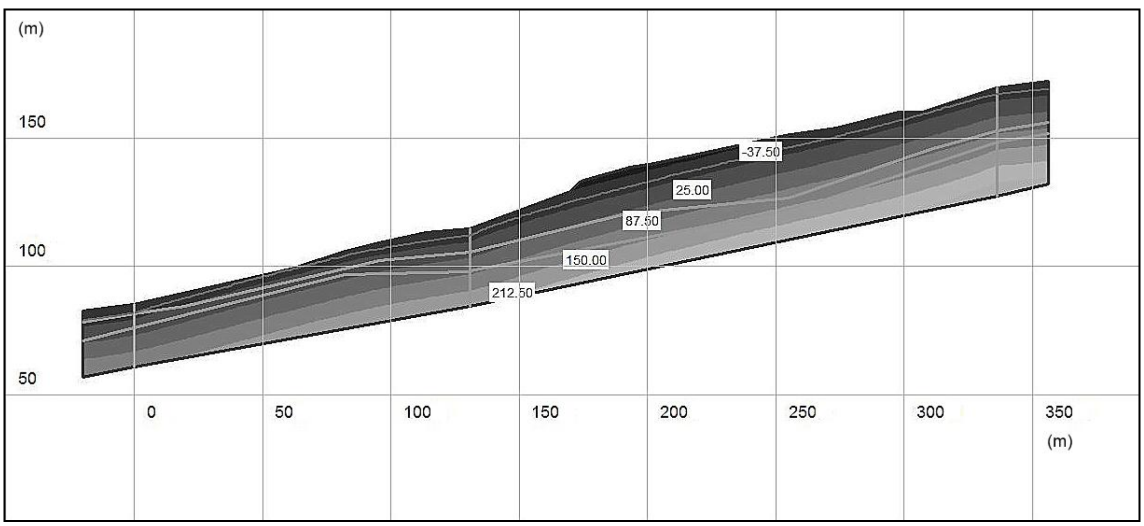

Figure 16: Distribution of pore pressure (kPa) in section A2-A2 – maximum groundwater level.

Maximum shearing strains obtained for the situations of imminent rupture and groundwater level conditions studied indicate the early development of the sliding surface, as can be seen for sections B1-B1 and A2-A2 for maximum groundwater level, in Figures 17 and 18. For the strength reduction factor equal to the safety factor or situation of imminent rupture, the results indicate major shearing deformation at points of the slope shown to be influenced by its geometry. According to the arrows in Figures 17 and 18, the points with the highest maximum shearing strain are verified in the massif areas after intense inclinations exactly where the colluvium soil layer meets the residual soil. Therefore, it appears that, in natural slopes with irregular topography, there is a tendency for several sliding surfaces to form, according to the change in slope of the natural ground surface.

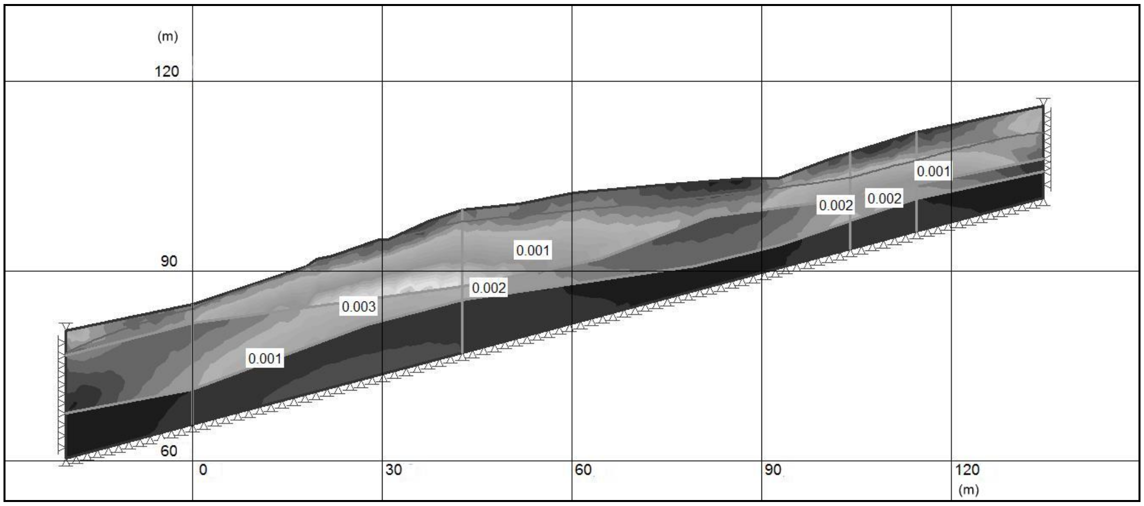

Figure 17: Graphic representation of maximum shearing strains, which is a result of the stability analysis, by finite elements, of section B1-B1, with maximum groundwater level, SSRF = FS = 1.11.

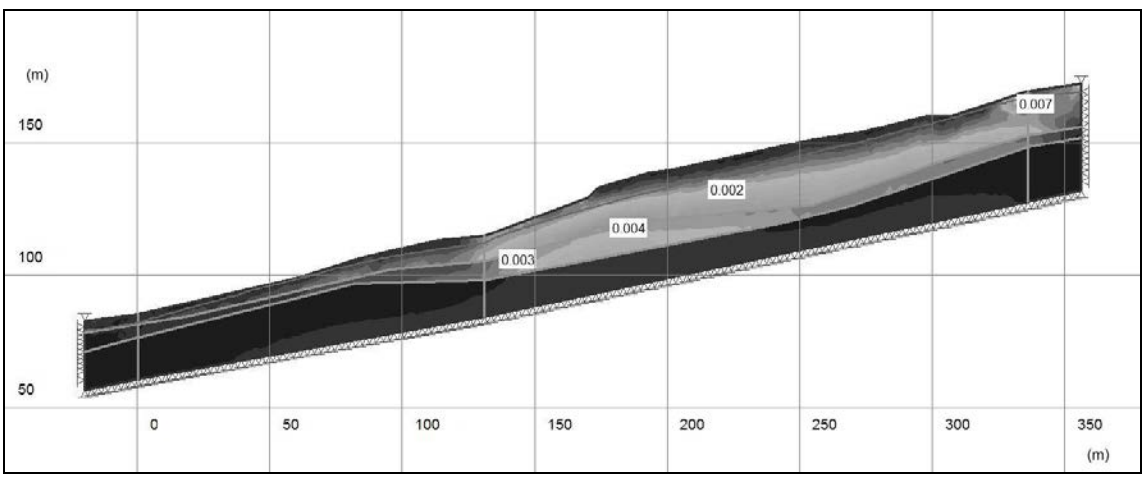

Figure 18: Graphic representation of maximum shear strains, which is a result of the stability analysis, by finite element, of section A2-A2, with maximum water level, SSRF = FS = 1.03.

FINAL CONSIDERATIONS

From the results of the tests performed in Coroa Grande and considering the ability of the techniques associated with the Finite Element Method to calculate the safety factor of natural slopes, relevant information about the behaviour of the massif was obtained.

According to the inclinometer results, throughout the monitoring period, one could observe that the sliding surface was predominantly verified on the contact surface of the colluvium with the residual soil. Considering the results from the water level meter in the study period, it was found that the sliding surface is located in saturated soil, even in the dry season.

The study of the displacement speed proved that the movement that occurred in the studied region was caused by creeping. The increase in the amount of rain, according to what was observed in inclinometers, piezometers and water level meters, in the period from June 1988 to June 1991, seemed to cause increases in the piezometric hydraulic loads and horizontal displacement speeds, as well as a rise in the groundwater level. Despite changes in horizontal displacement speeds, in the situations studied, the trends in the behaviour of the "horizontal displacement speed vs. time" curves did not indicate rupture.

Results of stability analysis by equilibrium-limit methods and by techniques associated with the Finite Element Method were presented. These results include strains with defined sliding surface and shearing strength reduction. The analysis of results allows us to conclude that the material in the sliding zone is in residual strength, since the high safety factors values obtained with the use of peak strength parameters disagree with the slope behaviour observed in the monitoring period. Considering the FEM for the maximum water level, the safety factors obtained ranged from 1.03 to 1.11, from 1.07 to 1.22 for the critical water level, from which accelerations in the slope movement were observed, and for minimum N.A., the range was from 1.22 to 1.54. The changes in safety factors values obtained with equilibrium-limit methods for maximum, minimum and critical groundwater levels were from 1.02 to 1.16, from 1.25 to 1.56 and 1.12 to 1.25, respectively. This explains, according to Lacerda (1997) for the maximum N.A. the verification of surface cracks in Coroa Grande hillside, for critical N.A. the movement acceleration, and for minimum N.A. the stability of the slope or decreased sliding velocity in the dry season.

Using FEM with the Shearing Strength Reduction Technique, it is possible to observe, with the view of maximum shearing strains, the configuration of the sliding surface. The results indicate the emergence of larger deformations at certain points on the slope, showing the apparent influence of its geometry, in massif areas, after increased inclination, since these points are exactly where the colluvium soil layer meets the residual soil. Therefore, it could be inferred that there is an initial trend, in natural slopes with irregular topography, of the formation of several sliding surfaces, according to the slope variation.

Considering the complexity of the case studied, the variation between the results obtained with the methods used was very satisfactory, proving that the use of the Finite Element Method for stability analysis in natural slopes may be an effective alternative, since the layers that compose the slope are properly identified and the soil parameters are determined in the laboratory. For the most critical situation of the maximum water level and soil strength parameters in the residual condition, it was observed that the results obtained with the Infinite Slope and Spencer methods did not differ, respectively, by more than 7% and 9% from the results obtained with FEM. The Finite Element techniques had a maximum difference between results of 6%. These results were, however, slightly higher than those observed by Duncan (1996).

In this specific study case, considering that the sliding surface was identified by inclinometers and in view of other factors such as the strong heterogeneity of the massif and possible anisotropy, it seems that the results obtained with methods that define the sliding surface, for parameters from strength parameters to residual shearing, point to questions about the massif stability, since the instrumentation results show that the slope is moving. However, one can also verify the importance of applying FEM to analyse the stability of slopes, resulting in consistent safety factors and the shape of possible sliding surfaces for cases where instrumentation does not exist.

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