Stability and movements of an anchored wall in phyllite

La stabilité et les mouvements d'une paroi ancrée dans le phyllite

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ABSTRACT
The paper discusses the stability and movement of a 24m deep anchored retaining wall. The retained material is a schistose phyllite which discontinuities layers are inclined towards the excavation. Stability calculations of the anchored retaining wall were carried out by limit equilibrium method adopting planar and circular failure surfaces as well as by the finite element method. These analyses were based on strength parameters obtained from the direct shear tests in which the residual strength parameters were estimated with the failure plane coincident with the schistose phyllite discontinuities. The analyses showed a safety factor value of the retaining system close to one for the current situation at the time of the analyses (24m excavation). This study shows that the designer adopted peak parameters without considering the schistose discontinuities. The numerical analyses also confirmed that an entire mass of soil underwent a considerable horizontal displacement. This result was in accordance to what was registered using the inclinometer installed on the site.

RÉSUMÉ
Cet article traite de la stabilité et le mouvement d'un mur de soutènement ancré à une profondeur de 24 m. Le matériel retenu est un phyllite schisteux dont les couches discontinues sont inclinées vers l'excavation. Les calculs de stabilité du mur de soutènement ont été réalisés par l'adoption de la méthode d'équilibre limite et les surfaces de rupture circulaires et planes, ainsi que par la méthode des éléments finis. Ces analyses sont basées sur les paramètres de résistance obtenus à partir des essais de cisaillement direct dans lequel les paramètres de résistance résiduelle ont été estimés à partir des essais de cisaillement direct avec le plan de rupture qui coïncide avec les discontinuités du phyllite schisteux. Les analyses ont montré un coefficient de sécurité de la fermeture du système d`un soutènement à l'autre pour la situation actuelle au moment de l'analyse (24 m d`excavation). Cette étude montre que le concepteur a adopté des paramètres de pointe sans tenir compte des discontinuités schisteuses. Les analyses numériques ont aussi montré que la masse entière du sol a subi un déplacement horizontal considérable. Ce résultat est conforme à ce qui a été enregistré à l'aide de l'inclinomètre installé sur le site.

Keywords: anchored wall, limit equilibrium, numerical method, peak parameters, post peak parameters, phyllite

1 INTRODUCTION

This paper presents a study to evaluate the behaviour of an anchored retaining wall located in the city of Belo Horizonte, state of Minas Gerais, Brazil. The profile of the retained soil behind the wall consists of a layered residual soil overlaying a moderately to highly fractured phyllite rock. A commercial building is located behind and alongside the retaining wall as shown in the photo presented in Figure 1. A new building was being constructed within the excavated site. The photograph shows the foundations of the building being constructed. The wall construction and excavation works were carried out in the period between 1991 and 2004 including a number of interruptions. As the excavation in front of the wall progressed, the building suffered cracks and structural problems. In 2004 when the excavation was about 24 m deep (the final planned depth was 30 m), the construction was halted.

Figure 1. Anchored wall and building behind.

Subsequently, the Geotechnical Group of COPPE/UFRJ was consulted to carry out a study to evaluate the overall conditions of the existing building and the retaining structure in the context of litigation between parts. The main aim of these studies was to determine the factor of safety of the retaining system. The strength parameters of the retained soil were assessed by laboratory tests on undisturbed samples collected in the site.

2 GEOLOGY AND STRENGTH PARAMETERS

The nature of the site geology, the mechanical parameters of the retained materials and the field test data including the inclinometer measurements were considered important to carry out consistent stability limit equilibrium and deformation analyses.

2.1 Site geology

The local geology of the site indicates the presence of massive weathered phyllite schist which is a typical metamorphic formation existing in Minas Gerais. The weathered site profile consists of a thin layer of old residual soil, a thick layer of predominantly-young residual soil with visible relict structure, and a layer of highly fractured rock overlaying a moderately fractured rock.

The discontinuities in the site are represented by both weathered and unweathered intrinsic schistosity planes. These planes have an inclination at an angle of about 70^o (from the horizontal plane) towards the excavation side as shown in Figure 2. The profile also features some tectonic joints and fractures with discretely defined directions.

The side of the excavation perpendicular to the building did not show any problem whatsoever as the direction of the schistosity planes was not unfavourable. Therefore analyses will concentrate on the anchored wall side alongside to the building.

Figure 2. Bottom of the anchored wall where 70^o discontinuities can be seen.

The weak nature of the material within the discontinuity planes is responsible for intense lamination (laminae) of the highly and moderately weathered rock. The resulting rock mass is considered as moderately weak and high permeable rock (Bieniawski, 1989; Hoek, 2000).

2.2 Strength parameters

In order to determine the geotechnical parameters of the retained material, undisturbed samples, shown in Figure 3, were taken from the soil surrounding the bottom of wall and drained direct shear tests were carried out. The measured unit weight of the phyllite

Figure 3. Phyllite schist samples collected for direct shear tests

was 20.2 kN/m^3. The tests were carried out on specimens where the orientation of the schistosity material within the phyllite specimens was aligned in the same direction of the shearing plane of the shear box. The tests were performed on samples of 25 cm^2 in area and an initial height of 2.5 cm. The specimens were tested in saturated conditions and subjected to normal stresses in the range of 100 - 600 kPa.

Figure 4 presents shear stresses versus horizontal displacement measured during direct shear tests and it is seen that the peak strength corresponds generally to a horizontal displacement in the range of 1-4 mm. Following the peak value, the soil strength dropped to fairly constant values in most of the tests, as shown in Figure 4. For the present conditions in which schistosity planes were made coincident with the shear box

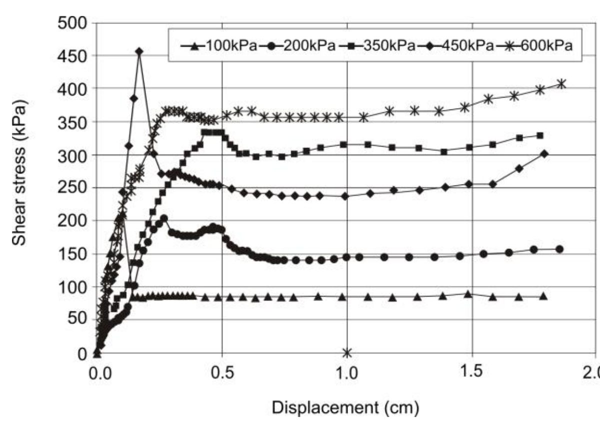

Figure 4. Direct shear tests: shear stress versus displacement

horizontal plane, the strength parameters can be obtained considering the residual strength.

In order to obtain the strength parameters to be used in subsequent stability analysis, the test results shown in Figure 4 were carefully analysed. It was then decided to discard both the peak point and the data points of the residual part of the curve for both normal stress values of 350 kPa and 600 kPa. A normal stress of 600 kPa was higher than the field stress in the field that is about 480 kPa.

Figure 5 presents the peak and residual data points. The data points indicated by full squares are the ones used to obtain the residual strength parameters of the soil: effective cohesion c' = 43 kPa and the friction angle $\phi' = 21^{o}$. The peak strength envelope is shown by the data points drawn as full circles in Figure 5.

These values are shown in Table 1 together with peak shear strength data used in design obtained from triaxial tests performed on specimens collected at shallow depths in the weathered phyllite schist. A large difference may be observed between these two sets of values, i.e. the strength parameters used in design were overestimated.

The deformations observed in the field are highly influenced by the presence of the planes of schistosity. Therefore, the residual strength parameters were considered adequate to be used in the limit equilibrium and numerical analyses.

Monitoring the horizontal displacement in the field using the inclinometer showed that the observed

Table 1. Strength parameters of the phyllite schist

Material	c' (kPa)	ϕ' (°)
Tests at COPPE/UFRJ (residual strength data)	43	21
Original design values (peak strength data)	40 – 100	28 - 32

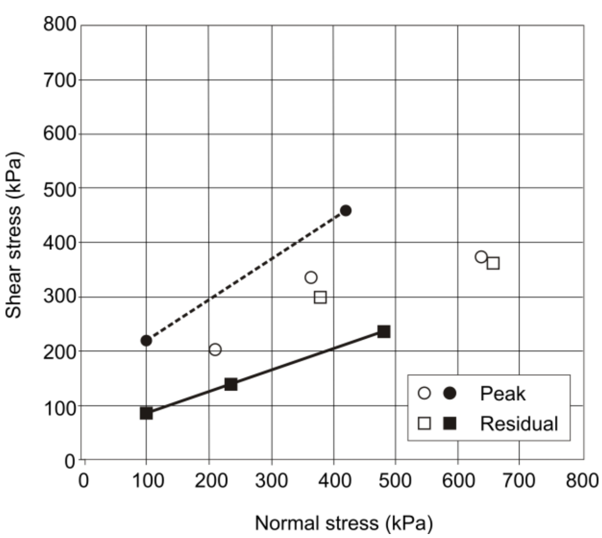

Figure 5. Strength parameters from direct shear tests.

displacements are higher than the displacement values that correspond to the peak strength values obtained from the direct shear tests.

3 WALL STABILITY ANALYSES

3.1 Loading tests

Loading tests were carried out in anchors installed in a vertical line along the 24 m high wall. The results of loading tests are presented in Table 2. Measured anchor loads were in the range of 230 - 400 kN. An average value of 318 kN was used in the stability analyses. The design service load, as indicated by the designer, in all anchors was 350 kN for a wall height of 30 m as shown in Table 2. Therefore at 24 m height, the wall mobilized about 90% of the service loads of the anchors.

Table 2. Results of anchors loading tests

Anchor	Anchor Type	Measured Load (kN) (H=24m)	Design Load (kN) (H = 30m)
TI 10 D	4 strands of ½"	247	350
TI 11 D	4 strands of ½"	283	350
TI 11 C	4 strands of ½"	305	350
TI 11 B	4 strands of ½"	373	350
TI 11 A	4 strands of ½"	231	350
10 Z	Dywidag	344	350
TI 8 Y	Dywidag	402	350
TR 7 B	4 strands of ½"	287	350
TR 16 B	4 strands of ½"	390	350
Average loads		318	350

3.2 Stability analyses

The Brazilian standard NBR5629/1996 – Installation of ground anchors - recommends a minimum safety factor (FS) of 1.5 for analysis adopting planar slip surfaces.

Initially, in order to evaluate the original design of the anchored wall system, the forces acting on the wall were calculated considering the soil parameters adopted in the original design and considering a FS equal to 1.5. The calculation showed that the global force acting on the wall was 84% higher than the forces estimated in design.

Table 3 presents the values of the factor of safety obtained from both limit equilibrium and finite element methods. The analyses were carried out using the residual strength parameters summarised in Table 1. These parameters (c' = 43 kPa and ϕ' = 21°) were determined corresponding to the post-peak part of the strength curve. These analyses were carried out for both wall heights: 24 m (the current one) and 30 m (the design height).

As can be seen in Table 3, the FS values for a wall height of 24 m were close to a unity. At this stage, the anchored wall height was 12 m and the height of excavation (without a wall) was 12 m. Ground anchors were installed over the height of the excavation; however, installation of the anchor head was not completed at this stage. The values of safety factors obtained from the finite element method were marginally higher than the values obtained from the limit equilibrium considering either a planar surface or a circular slip surface and this may be attributed to the fact that FE analyses considered the presence of the anchored wall, not considered in limit equilibrium analyses.

However, the values of factor of safety obtained from the above three methods were relatively low. These values of factor of safety are considered unsatisfactory as they are lower than the minimum value specified by the Brazilian standard.

The results of FS shown in Table 3 indicate that the wall is at failure at the height of 24 m.

Table 3. Wall factors of safety - heights of 24 m and 30 m.

Method of analysis	F.S [(a)] (H=24m)	F.S [(b)] (H=30m)
Lim. equilibrium planar	0.90	0.90
Lim. equilibrium circular Simplified Bishop method	0.87	0.70
Finite element	1.10	1.00

(a) using anchor load of 320 kN as obtained from loading test

(b) considering anchor load of 350 kN (design service load)

4 DISPLACEMENTS AND NUMERICAL ANALYSES

The building located behind the wall has five storeys two of which are below the street level. The building is founded on 110 piers of about 8 m long. Settlements measured in some of the building columns are presented in Figure 6. Most of the structural elements of the building are precast elements thus forming an isostatic system that is sensitive to movements. Damages which occurred in the building are exemplified by cracks and movements shown in Figure 7.

Finite element analyses were carried out to evaluate the deformation behaviour of the anchored wall and the adjacent building. The analyses were carried out using the COPPE/UFRJ ProGeo finite element program in which the phyllite was modelled as a non-linear elastic hyperbolic material with adequate considerations to the question of the phyllite fracturing. Two material zones of phyllite were considered as shown in Figure 8. For the active part in which the direction of the shear plane coincides with the cleavage direction of the material, the strength properties used were the residual strength parameters. Higher strength parameters were assigned to the material present at the bottom passive zone of the excavation. In this zone, the direction of the shear plane is different from the direction of the cleavage.

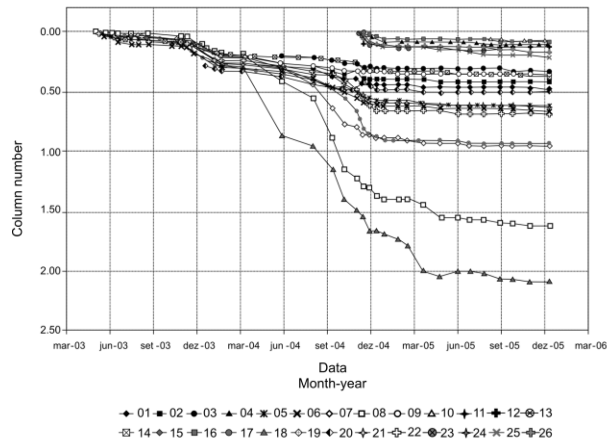

Figure 6 Measured settlements in the period 05/2002-01/2006

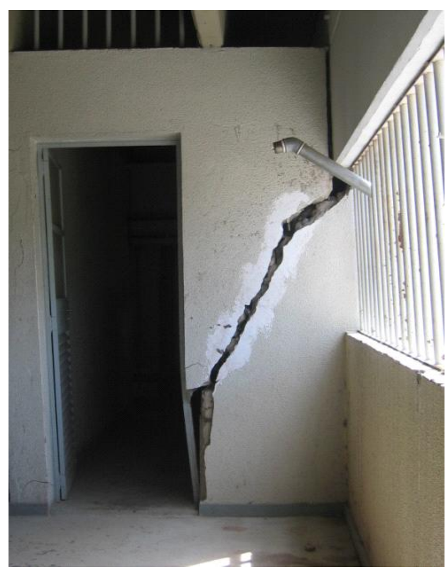

Figure 7 Cracks and movements in the building

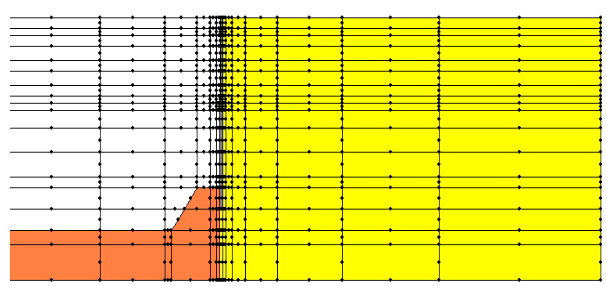

Figure 8. Finite element mesh showing excavation of 24m.

The FE analysis incorporated the most important construction stages aiming to explain the mechanisms that caused the structural defects in the adjacent building. Figure 9 shows the displacement vectors at the end of construction stages. It is clear that a soil mass moved quite significantly toward the passive part of the wall.

Table 4 summarizes the material properties used in the FE analysis. Some geotechnical parameters were estimated. Therefore, the finite element analysis aims mainly to represent the overall pattern of displacements rather than trying to match the observed displacements.

The measurements gathered using inclinometers in mid 2004 indicate a pattern of displacements along the wall height that has a similar shape to the displacement pattern obtained with FE analyses. The displacement profile showed larger displacements concentrated near the foot of the wall. However, results of numerical displacement are mostly qualitative due to limitations regarding deformation (Young's modulus E) and stress history parameters (coefficient Ko) as a number of simplifications regarding the construction sequence have been made.

Table 4. Material properties used in the FE analysis.

Material	c' (kPa)	ϕ'(º)	E_i (kPa)	ν	R_f
phyllite [1]	43	21	$2{\cdot}10^4$ - $2{\cdot}10^5$	0.4	0.8
phyllite [2]	80	35	$4{\cdot}10^4$ - $4{\cdot}10^5$	0.4	0.8
Concrete wall			$2{\cdot}10^6$	0.2	

[1] shear plane in direction of schistosity.
[2] shear plane in a different direction to the schistosity.
ν - Poisson`s ratio
R_f - Failure ratio

5 RECOMMENDED REMEDIATION SOLUTION

It was recommended to install a greater number of anchors to improve the stability of the system and increase the safety factor from about 1.0 to 1.5 as specified by the standards. The additional anchors should provide a force of about 1050 kN/m. The anchorage length should be determined in order to ensure that the anchor end is positioned beyond the failure slip surface; i.e. the anchor should intercept the failure slip surface.

6 CONCLUSIONS

The results of the stability analysis carried out in this study show that the retaining wall in question does not meet the Brazilian standard that requires a factor of safety not less than 1.5.

The structural problems including development of cracks in the adjacent building were due to the inadequate retaining wall that did not provide a reliable retaining system and hence caused significant displacement during the excavation works.

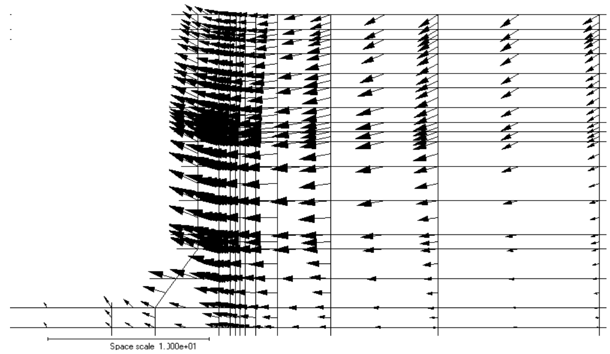

Figure 9 Displacement vectors in FE analysis for H=24m

The analysis also showed that significant displacements occurred within the zone in which the orientation of the weak material (the cleavage) coincides with the direction of the shear plane. This finding emphasizes on the importance of carrying out direct shear test on samples that reflect the nature of the material encountered in the field in term of direction of weak inclusions. The results also showed that at the bottom of the excavation, relatively small displacements were obtained. This was due to the fact that at this zone the direction of shear surface does not coincide with the direction of the cleavage within the material.

The results of the analysis indicate the necessity to improve the stability of the system and increase the safety factor from about 1.0 to 1.5 as requested by the standards. This can be done by installing a greater number of anchors.

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