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A Landslide in a Residual Soil's Cut Slope

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SUMMARY This paper presents several landslides that have occured in the same residual soil slope in Rio de Janeiro, Brazil. The solutions that were designed and their performances were also described.

1 INTRODUCTION

The modern Soil Mechanics really began with Terzaghi's first concepts. For its simplicity in predicting soil behavior, these theories are still being widely utilized nowadays.

Terzaghi and his contemporaneous engineers dealt chiefly with sedimentary soils, whose homogeneity helped the development of equations and methods that could previously quantify the performance of soil, either in its natural state or within engineering structures.

As modern civil construction was developed, the geotechnical engineers recognized the difference between sedimentary and residual soils. By that time, Terzaghi already pointed out the importance of tropical countries in creating a new Soil Mechanics--that of tropical soils.

Knowing that we are still in the earlier steps of this knowledge, the authors present in this paper a case history that they had faced in a residual soil cut slope, expecting to help in the task of understanding tropical soils.

2 FIRST DESIGN OF THE SLOPE

In 1985, an educational program supported by the Government of the State of Rio de Janeiro was constructing several full time schools to the lower social classes. These standardized structures have been constructed in precast concrete modules, and were composed of three buildings:

- The "main building" with dining-rooms, medical and odontological center and classrooms;
- A library;
- A gymnasium.

One of these schools was located in the Municipality of São João de Meriti, a region of biotite gneiss.

The boreholes made to investigate the foundations buildings indicated a typical weathering profile in metamorphic rock with a brown and red silty clay overlying a yellow silt. The "A" Deere & Patton (1971) horizon was not found.

As the topography had indicated only a seven meters high cut, no further studies concerning the slope stability were made, since stable slopes higher than that one were common in that area.

By the end of the earthworks, the engineers realized that this topography survey was not very precise, so this seven meters high cut slope turned out to be eighteen meters, 1:1 slope, with no benches.

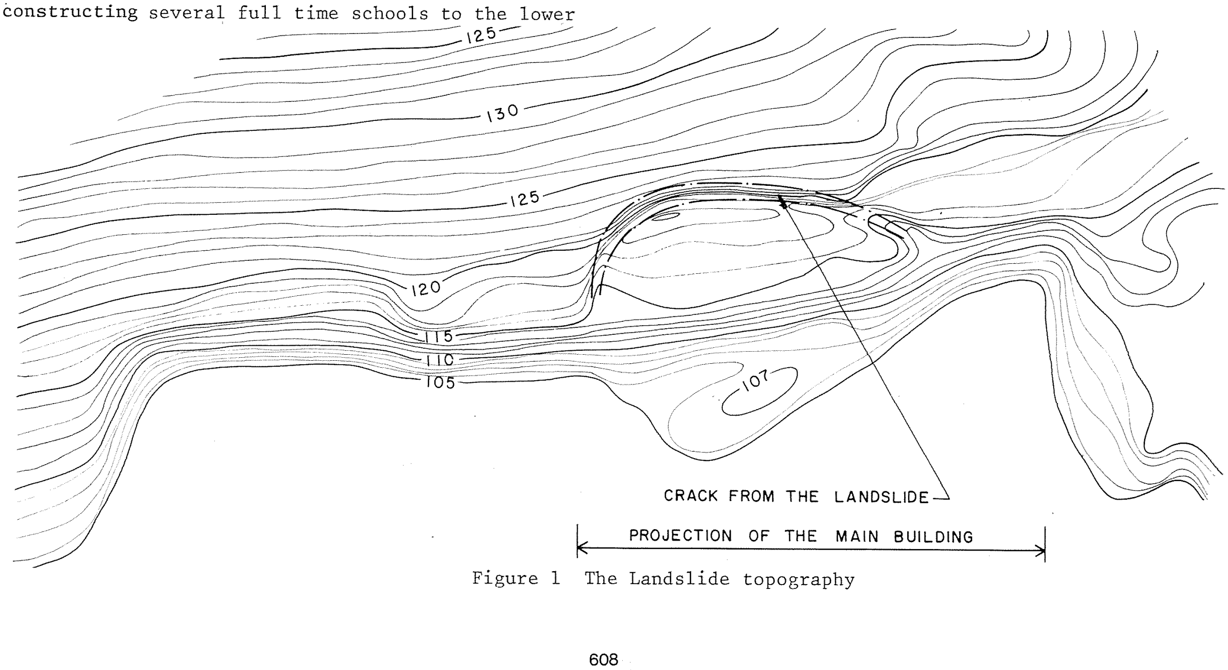

Figure 1 The Landslide topography

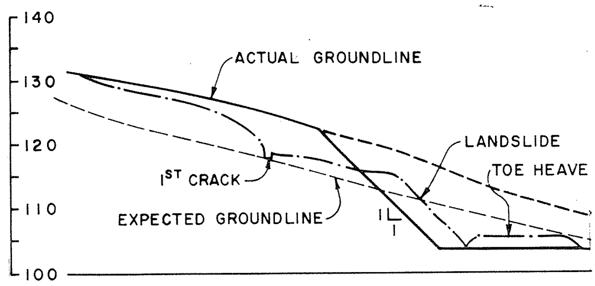

Figure 2 Cross-section

Considering that it was a slope behind a school, the engineers decided that a higher safety was necessary and indicated a flatter slope with the introduction of some benches. Nevertheless, before any further work was begun, a slide happened in part of the slope just in frontof the main building. The figures 1 and 2 show the landslide topography and the cross-section.

3 CONSIDERATIONS FOR THE LANDSLIDE BACK-ANALYSIS

The landslide happened just after the cut was completed. The water table was in its lowest level, so it was considered a small value for the pore pressure.

Also, the geotechnical profile, obtained from SPT-borings and auger holes, showed that the partially saturated residual soil had a drained behavior.

Unfortunately, as it was a governmental project, the need of an immediate solution and the present economic situation of the state didn't allow neither time nor money to make an adequate investigation. Hence, there was no confidence about the position or form of slip surface. Anyhow, quick back-analysis were made: one using the Kerisel charts (1967) which lead to a safety factor not far away from unity, and another one considering an approach to the tri-planar wedge surface, as suggested in Chowdhury (1978), and shown in Figure 3.

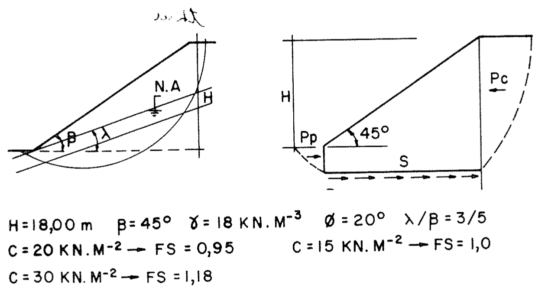

Figure 3 Slope stability of the first designed slope

Those considerations induced to two hypotheses to explain the rupture:

- the kaolinitic and very micaceous clayey silt located at the toe of the sliding slope, has lower shearing strength than the surrounding soil. Thus, as it failed when deconfined, its resistance dropped to the residual value. The stresses transferred to the surrounding soil brought this mass to its failure state.
- the low safety factor, not far from unity, let the mass deform to attain its maximum shear resistance. The "C" horizon, present at the toe of the slope, was a micaceous clayey silt with brittle characteristics which came to a progressive failure.

Finally, the presence of slickensides among the collapsed mass emphasized the opinion of the main role of the weak planes inherited from the gneiss bedding. In addition, the visual superficial inspection, performed just after the slide, has given the impression of a bottom failure or yet a wedge one as shown in Figure 4.

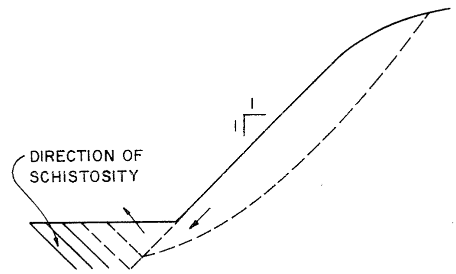

Figure 4 Admitted slip surface

4 SECOND DESIGN STABILITY ANALYSIS

To ensure the safety of the soil mass, a stability analysis was carried outsupported on friction angle and cohesion parameters governed by residual strength.

On trying to minimize the earthworks, it was decided to take the risk of letting the failure mass partially remain partially in place.

The calculations were made again using the Kerisel charts, taking in account ground water flow and geotechnical parameters assumed with base on local experience. They are related in Table I.

TABLE I

GEOTECHNICAL PARAMETERS

γ (kN/m^3)	C' (kPa)	Ø (degrees)	λ/β	SF
18.0	15.0	20°	3/5	1,12
18.0	15.0	25°	3/5	1,40
18.0	20.0	20°	3/5	1,24

The assumed section according to the above safety factors turned out to be two steps of a 2:1 (H:V) slope into the damaged micaceous material with an intermediate three meters bench, and 1:1 slope in the mature residual soil as shown in Figure 5, on next page.

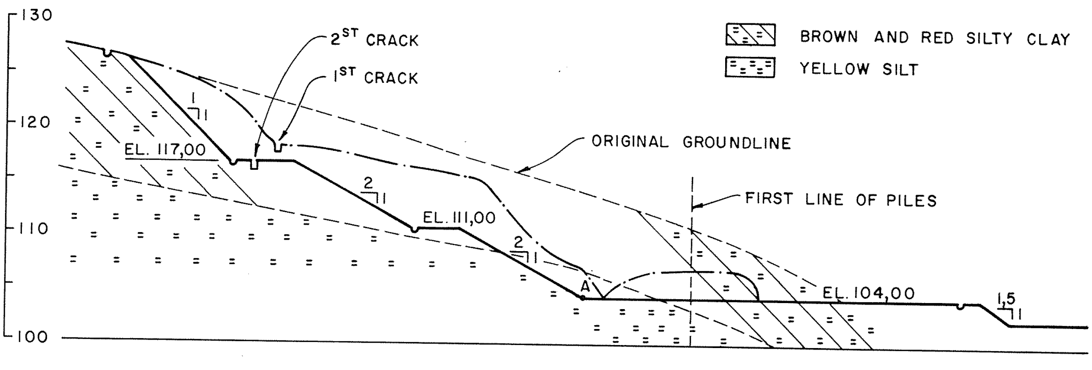

Figure 5 Second designed cross-section

Again, when the earthworks were almost finished, the slope began to present a mass displacement, initially inducing a "crack" on the second bench (from bottom to top) near the outcropping slip surface.

The explanation to this fact seems to be clear. The non thixotropic soil in its residual strength needed a larger deformation to mobilize its shear resistance than the not remolded one. Hence,the different degree of strain-mobilization, shown in figure 6, resulted in an open joint.

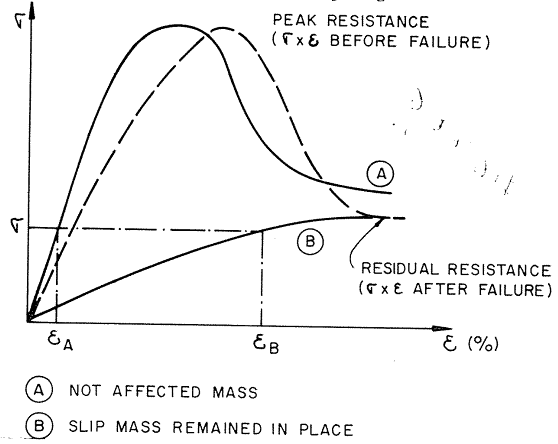

Figure 6 Degree of strain mobilization

During this period the pile driving of the main building was initiated using the available equipment. In about 20 days, it developed a toe heave of 25 cm and a throw of 30 cm at the head of the slide that forced the driven piles. Three of these piles sheared with this movement, showing a horizontal displacement of about 15 centimeters.

In order to relieve the pressure forcing the pile caps, a trench was dug in front of the caps and the soil was substituted for a loose material.

With no additional experimental data available, a new solution had to be designed.

5 THIRD DESIGN - REMOVAL OF THE MATERIAL

Finally, the intention of removing all the affected material has led to a solution of cutting 10 meters from A to A' into the toe of the slope as shown in Figure 7.

However the engineers faced a problem: just before reaching the final level of the earthworks, the crack opened again to about 5 cm.

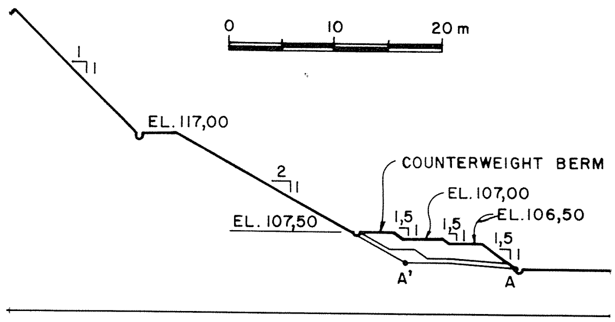

Figure 7 Third design and counterweight berm

6 THE COUNTERWEIGHT BERM

It was seen that when the earthworks were 0,5 meters above the final level, the joint opened, thus characterizing a limit situation (SF = 1.0). To increase the safety factor at the toe of the slope, an earth counterweight (3.0 meters high) was then designed extending over the limits of the damaged mass as shown in Figure 7.

Its construction had begun immediately and quite soon the movement decelerated finally attaining a rest condition.

7 THE BIGGEST MOVEMENT

Though the movement seemed to be under control, an unexpected crack appeared surrounding the whole cut as shown in figure 8.

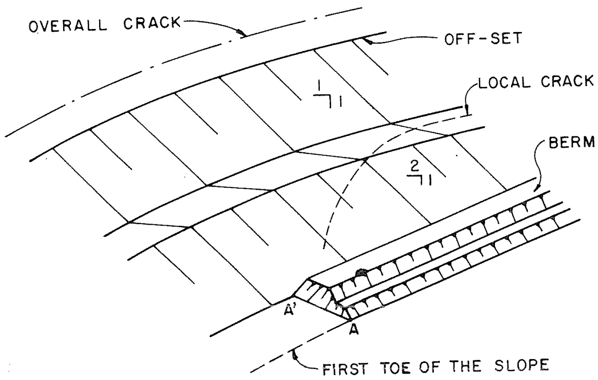

Figure 8 Overall crack and final situation

Then, what had been observed as a small joint became a wide opening (aprox. 30 cm) and step (aprox. 80 cm).

Three things became apparent. First, that the initial potencial slip surface, when the berm was built, had its factor of safety increased, leading the slope to "search" for another potential slip surface.

Second, the berm designed on the damaged toe area should take in account the whole slope stability, and probably should be extended all over the toe of the slope. Third and more important, this creep movement confirmed the trend of the slope to slip over its weak joints.

The final solution was a much larger earthwork whose construction was not completed, because a change in the administration has caused a temporary delay at the educational program thus suspending the civil works. Anyway the displacement has stopped.

Now it has elapsed almost one year of work interruption and in a recent visit it was observed that the creep movement has decreased almost to zero. Despite that the drainage system was not concluded and no conservation was performed during that time, the cut is having a good behavior. At the present, the government is resuming the services to finish the school, and at that time will probably include a complete investigation of the materials.

8 CONCLUSIONS

This slope has emphasized the importance of understanding the geological structure of residual soils to well predict the performance of slopes.

It also pointed out that we still know little about its mineralogy and its rhelogic characteristics. Physical and strength parameters are almost non-existent on that kind of soil, specially the sensitive brittle type.

The first slide and the creep movement showed that the traditional cylindrical surfaces and typical values of cohesion and friction angle don't always help to solve difficult problems in residual soils. The many variables that take place in its behavior are still not known and not well measured.

Also, it is said that the engineers would hardly have predicted that this slope would be so problematic, if supported only on the traditional manner of investigation. Many important features are not detected in boreholes with intercalated samples and wash boring above the water table. For instance, the material brittleness is hardly detected by the standard penetration test.

Finally, among others suggestions, it raises the importance of a geotechnical engineer to supervise the earthwork in order to confirm the predictions made in the office, and to adapt the project to the exposed conditions.

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