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Analysis and control of the deformational process of a weak rock mass affected by a large landslide

Analyse et contrôle du processus de déformation d'un massif de roches tendres affecté par un grand glissement

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ABSTRACT: A large landslide more than 1.000.000 m³ affecting an excavation in weak rocks has been investigated during several years. The evolution and characteristics of the movement have been studied, as well as the behaviour of the weak rock mass during the failure process.

The characteristics of the deformational process affecting the rock mass and the geomechanical behaviour of the weak rocks involved have conditioned the corrective measures which have led into the stabilization of the landslide.

The corrective measures carried out were based on the analysis of the evolution of the movement, and their efficiency have been checked controlling the response of the rock mass througout the time.

A detailed investigation of the failure have been carried out, including the analysis of the rock mass stress-strain relationships by modelling its behaviour with a finite element stress-strain method.

1 INTRODUCTION

During the excavation of the toe of a hill slope in order to get an esplanade to install a mining equipment, a large landslide more than one million cubic meters began in the slope when the excavation works were finished, affecting the excavation and the natural slope above it. The unstable slope was 100 m high, 300 m wide and 400 m long.

The landslide has been investigated and controlled for almost four years. Boreholes, topographical measures and geophysical survey were carried out, and inclinometers were installed in the first stages. During the period 1990-1993 a continuous control of the movement in the slope has been carried out. The objective was the stabilization of the landslide to avoid the mobilized material reached the esplanade. The depth and shape of the failure surface was determined, as well as the failure mechanisms, water pressures affecting the rock mass and the geomechanical properties of the involved materials.

The corrective measures were based on the study and characterization of the failure process and on the results of the stability analysis, carried out by limit equilibrium and stress-strain methods. The numerical models allowed to know the influence of the different proposed stability measures on the evolution of the movement.

This paper describes the evolution of the deformational process of the slope and the stabilization measures carried out based on the characterization and analysis of the landslide by different methods. The characterization of the rock mass has been accomplished from field observations and data collection, boreholes, field instrumentation, gephysical survey and laboratory tests.

2 GEOLOGICAL AND GEOMECHANICAL CHARACTERISTICS

The slope is located in a complex structural area, with faulted and folded carboniferous materials. The rock mass, formed by an alternation of predominant shales, silts and sandstones with some thin coal layer, can be considered as a

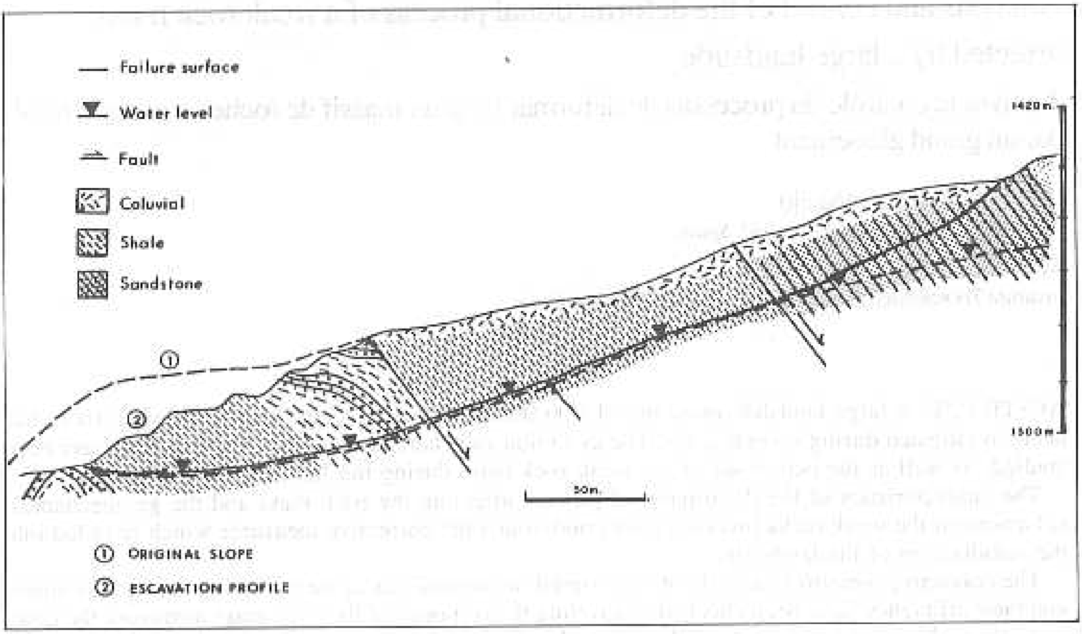


Figure 1. Geological section of the slope.

weak rock mass with respect to its geomechanical behaviour, which is not controlled by the joint planes; the geomechanical parameters of the whole rock mass depend on the intact rock strength.

The slope is one of the flanks of an eroded and faulted anticline, the strata with variable dips generally between 45° and 90° to the interior of the slope (Figure 1). Two faults affect the slope, with a lot of accompanying little folds. The upper part of the slope is formed by competent sandstone layers which constitute a hard rock outcrop. Some joint sets affect the materials, mainly to the shales, which present a more elevated fracturation degree.

Three boreholes were carried out on the landsliding mass. Table 1 shows a representative section from borehole C.

In the borings the tubes initially installed were cut by the slide at a depth between 22 and 25 m, except for borehole A, which was cut at 16 m and was reperforated to a depth of 60 m in order to find possible deeper failure planes; fresh shale was recorded between 16 and 60 m.

The mean values of the geomechanical parameters of the materials appear in Table 2. The strength of the shales was measured with conventional uniaxial tests between 15 and 70 MPa, while with a stiff servo-controlled testing machine the obtained values vary between 3 and 6 MPa, what agree with the geomechanical behaviour of these weak rock masses and with the data obtained in some other studies carried out for this type of weak materials (shales), been not valid the strength values from soft conventional test machines to carry out the analysis and modelling of the deformational processes that affect to the rock masses mainly formed by carboniferous shales (Ferrer and González de Vallejo, 1991; Ferrer, 1992).

Table 1.

Depth	Description	Weath. Degree	RQD
0-23 m	Alternation of predominant shales and sandstone layers	IV-V	> 30%
23-25 m	"	II-III	
> 25 m	Fresh shale		

Table 2. Intact rock geomechanical parameters from lab tests (mean values).

Material	Shale	Sanstone
Density	2.65 t/m^3	2.64 t/m^3
σ_c (MPa)	35	50
E (MPa)	3 x 10^4	4.5 x 10^4
ν	0.24	0.3 (0.25)*
C (MPa)	0.1 (0.05)*	(0.1)*
ϕ	25-30 (15)*	(30)*

(*) Weathered material

3 DESCRIPTION OF THE FAILURE PROCESS AND CORRECTIVE MEASURES

The first signs of instability appeared in April 1990, with the generation of large tension cracks in the upper part of the slope, about 300 m on the top of the excavation carried out in the toe of the slope (Figure 1), and coinciding with an outcrop of sandstones. During next weeks several new tension cracks were noticed, transversal and longitudinally to the natural slope, but no signs of instability affected to the excavated toe of the slope.

Some borings were drilled, and a field characterization and control of the tension cracks and scarps were carried out. Topographic benchmarks and inclinometers were also installed in order to controll the movements.

In July 1990, the movement accelerate, and the excavated benches in the lower part of the slope began to fail. A water spring is detected at the toe of the slope, and the material is wet in this zone; the failure surface appear at some meters above the toe of the slope, showing the displacement strias, and affecting to the lower excavated benches. The plane could be literally touched. The extent of the landslide and their limits were already clearly defined by large tension cracks.

From the log of the borings, the inclinometric curves and the data from the geophysical investigations, the failure plane was located between 16 and 35 m depth in different parts of the slope, with a sligth curved shape at the toe. The failure surface appeared as a clayey breccia with shale and sandstone fragments, its thickness been more than 1 m in some of the boreholes, and cut by several thin smooth slide planes. From the geophisical investigations it was deduced also the shape of the failure surface, with an inflection in their central zone.

The failure mechanism mainly corresponds to a traslational model, with a rotational component at the toe. No structural control due to preexisting discontinuity planes exists, what implicate the failure of the rock mass occurs through the intact rock.

The water level was puntually measured from the borings and their location in the rock mass was also investigated with geophysical techniques, which indicated their location in the failure surface, suffering frequent changes up to 10 m above the failure surface associated to the rainfall rate in the area .

By the end of July, the excavation of the material of the upper part of the slope is decided, trying to stabilize the landslide with the change of the weight conditions, as well as the installation of horizontal drains in the toe of the slope, in order to reduce the water pressures and the flow of water along the failure surface at the toe (Figure 2). The removed material was 390.000 m^3.

As a consequence of this stabilization measures, the movement rate decreased considerably as can be seen in Figure 3, which shows the evolution of the surface displacements in time for almost four years, where the results of these actuations can be noticed from August 1990.

At the end of October, new tension cracks are detected in the lower benches of the new slope excavated, showing that the lower part of the slope were still unstable. The new tension cracks connected with the initial failure plane; the upper part of the initial landslide (were the material had been removed) did not present any sign of instability. In Figure 3 the curves from October 1990 represent this "second" step of the movement.

During December 1990, new excavations were carried out to remove part of the material at the toe and on the previous excavation, with a new decrease of the movement rate as show Figure 3. However, during next weeks the displacements increased when intense rainfall occurred in the area, while for normal rainfall there were no appreciable movement. From the rainfall curve in Figure 4, the relation between the ammount of rainfall and the surface displacements in time can

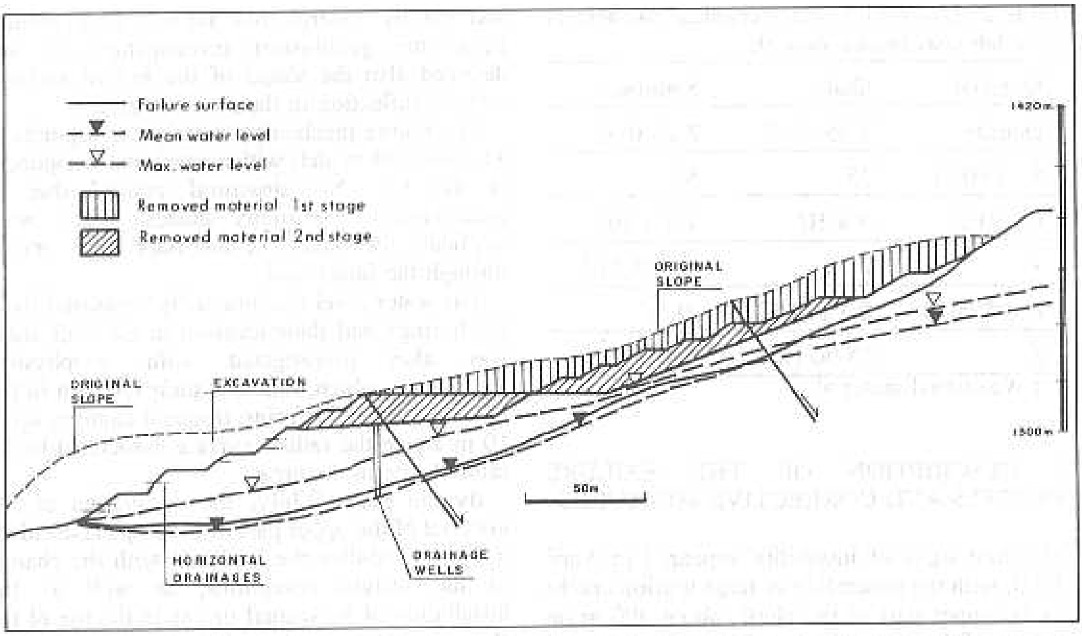


Figure 2. Section of the slope with the corrective measures.

be deduced: the increase of the displacementrate occurred in March-April 1991, correspond to a rain peak with more than 200 l/m² in March.

In April-May 1991, four drainage wells 1.5 m diameter and 16 m depth were excavated in the medium-low zone of the landslide, and new horizontal drains are installed at the toe. From this date, the movement decrease drastically.

Since then, a continuous control of the movements have been carried out by topographic benchmarks, and the displacement rates are also represented in Figure 3 untill October 1993. Some new horizontal drains have been installed at the toe during 1992-1993, and the slope has been revegetated.

During 1993, two rain peaks have been recorded in the area with more than 220 l/m² in the months of May and October 1993, but there have not been displacement increase associated to the rain ammount.

Figure 5 shows the curves representing the volume of water fallen in the landslide and the volume of water drained by the drainage system installed in the slope. A good correlation can be deduced, and the imbalance between the two curves can be associated to the surface flow and to the infiltration of water into the rock mass.

The measured displacements for last year (1993) affected mainly to the upper part of the landslide, with a maximum value of 5-10 cm/year. The representation of these values in the surface displacement curve show a clear tendency to the stabilization, mainly in the toe of the slope, and implicate an acceptable degree of stabilization for the slope in spite of the heavy rainfall occurred in some periods in the area. No signs of instability can be observed in the slope, except some local surficial instabilities.

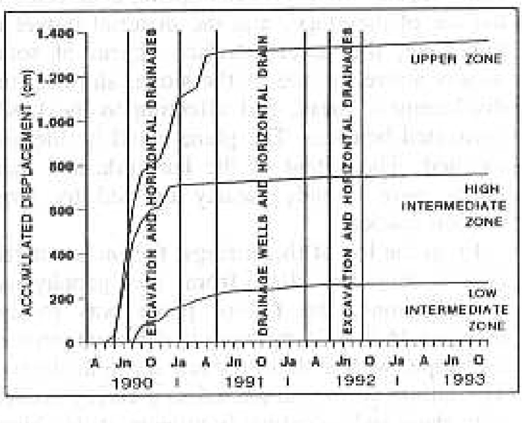


Figure 3. Evolution of the surface displacements of the slope in time.

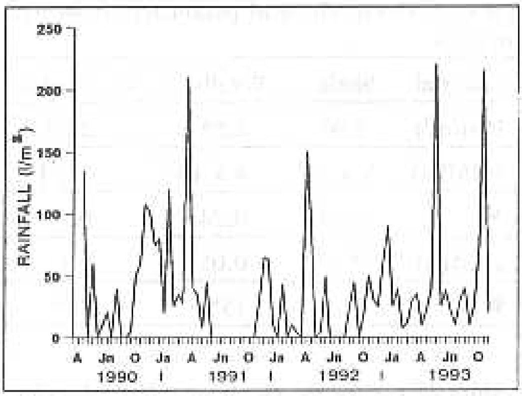


Figure 4. Rainfall data for the investigated period.

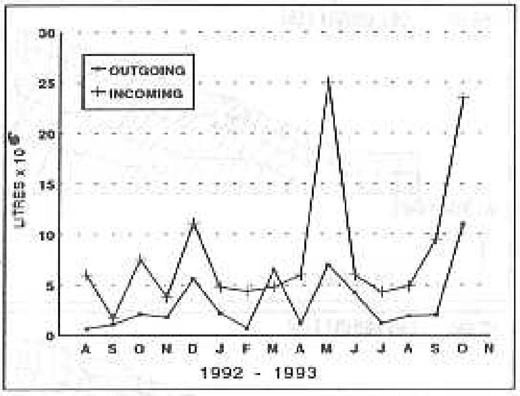


Figure 5. Curves showing the amount of water incoming (rain) and outgoing the landslide (drainage system)

4 STABILITY ANALYSIS

In the initial steps of the instability process, after the geometry and location of the failure surface, the geomechanical properties of the materials and the water conditions were identified, stability analysis of the slope were carried out in order to modelize the deformational process, to optimize the control and corrective measures and to check their effectiveness.

Initially, limit equilibrium analysis were carried out with the data obtained from laboratory tests (Table 2) and with STABL code. Two different models were considered, the natural slope (before the toe excavation) and the slope with the excavation carried out to build the esplanade. For the second case, the circular failure surfaces analyzed did not adapt to the shape of the actual plane, being always deeper. The minimun value for the factor of safety obtained for the modelized surfaces were in all cases greater than 2.3 (Beltrán de Heredia, 1992).

In order to modelling the actual failure plane, a non circular surface was prepared, but again the factor of safety obtained was greater than 2.0. So, the geomechanical parameters of the materials were decreased to get a factor of safety = 0.99 (back-analysis) for this representative (non circular) modelized failure surface. The value of the geomechanical parameters (cohesion and internal friction) resulted very much lower than those from the laboratory tests.

With these reduced values, circular failure surfaces could also be modelized for factors of safety minor than 1.0.

The main conclussion from the limit equilibrium analysis was the no correspondence between the laboratory values and the back-analysis values for the involved materials, mainly for the weathered shales.

Due to the characteristics of the progessive failure process of the studied slope and the geomechanical characteristics of the weak materials involved, and considering that the deformational process in rock masses can not be modelized by limit equilibrium methods and no data related to the stress-strain state of the rock mass can be obtained, it was decided to aply a FE stress-strain analysis method (ZSOIL code) (Ferrer, 1992).

The rock mass was then modelized to carry out the numerical analysis. The method allows the consideration of the stress-strain relationships of the material as well as the use of plastic failure criteria, aspects which are not considered by other conventional methods.

The back-analysis allowed to reproduce the deformational process and the modelization of the different steps of the behaviour of the slope.

The different materials were modelized with an elasto-plastic behaviour; the Drucker-Prager criterium was used for the ductil materials, being one of the more extended plastic criteria used in rock mechanics.

The objective of the analysis was to reproduce the actual behaviour of the slope in order to get the representative geomechanical parameters of the materials, as well as to modelize the deformational process and the stress-strain

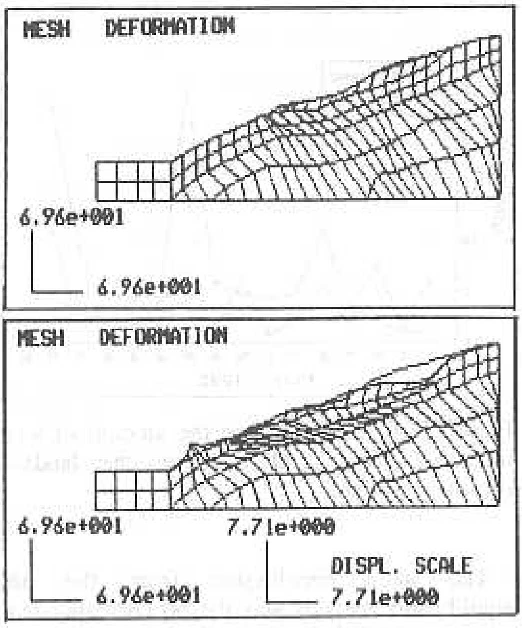


Figure 6. Initial (above) and deformed FE meshes for the back-analysis of the slope.

conditions in the different zones of the rock mass. It was considered that instability occur when non recoverable deformations are reached. The influence of the water pressure and the faults were also investigated, as well as the elasto-plastic behaviour of the shales.

In Figure 6a the FE mesh corresponding to the model of the slope with the initial toe excavation (which caused the beginning of the landslide) is presented. The back-analysis carried out in order to modelize the actual deformational process, provided the values of the geomechanical parameters for the involved materials when the failure is reached. The correct modelling of the failure process was obtained, with the same deformational steps observed in the slope, mainly those features affecting to the upper part and to the toe of the slope. Figure 6b shows the deformed mesh when the instability of the model is reached in the analysis, with the main failure features affecting the slope and the failure plane coming out some meters above the toe.

Table 3 shows the values of the geomechanical parameters of the materials from the back-analysis. For these same values, the analysis carried out considering the initial model of the natural slope (without the excavation) provided

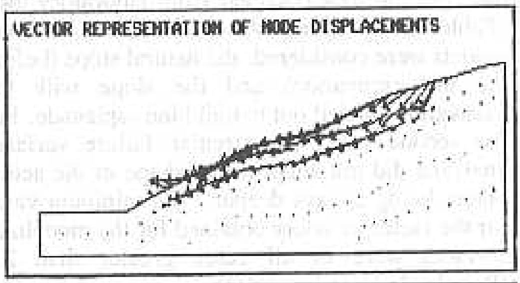
Table 3. Geomechanical parameters from back-analysis.

Material	Shale	Weath. shale	Sandst.
D (t/m^3)	2.65	2.65	2.64
E (MPa)	5×10^3	4×10^3	9×10^3
ν	0.24	0.24	0.31
c (MPa)	0.1	0.05	0.1
ϕ	25°	15°	32°

values of 1.3 for the factor of safety, showing the stability of the slope before the excavation of the toe.

In the first steps of the back-analysis (when the stress-strain relationships are elastic and the slope is still stable), maximum displacements occur at the upper part of the slope, while compressive stresses increase in the inner and at the toe. When the no convergence is reached (that is generalized plastic deformations take place), maximum displacements appear also at the toe's slope zone, implicating plastic displacements through the intact rock.

Figure 7a shows the displacement vectors for the step of the analysis when failure is reached



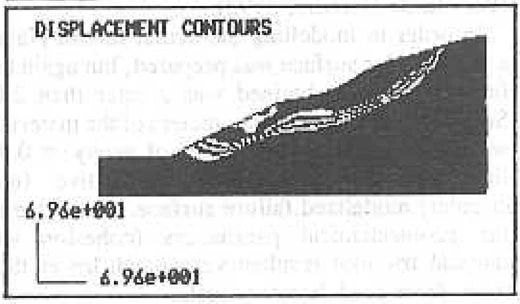


Figure 7. Displacement vectors (above) and displacement contours for the failure.

(F.S. < 1.0). These vectors indicate the type of movement affecting to the rock mass and the location of the failure surface; the portion of the rock mass which was not affected by the slide can also be seen, corresponding with what actually happened. Figure 7b shows the displacement contours, indicating again the location and shape of the failure, with the inflection of the failure surface which was also deduced from the geophysical investigations.

From these first analysis modelling the beginning of the instability process, the corrective measures designed aimed at the stabilization of the landslide were modelized and analyzed, what allowed to get the deformational and the stress distribution patterns corresponding to the different actions.

5 LANDSLIDE CONTROL AND STABILIZATION

The factor of safety for slopes in weak rock masses as the case here exposed (mainly formed by shales), where large unstable masses and deformations take place, can not be considered in the same way that for slopes in other type of materials, where factors of safety of 1.2 or more are usual.

Due to the characteristics of the progressive failure process and the geomechanical behaviour of the rock mass, associated to a plastic deformation rather than to a peak failure process, the economical costs associated to the consecution of a factor of safety greater than 1.0 are so elevated that would be better to eliminate all the mobilized mass.

On the other hand, the corrective measures usually applied to other type of landslides and materials (anchors, walls, etc...) are not effective due to the special characteristics of these soft materials. For example, the construction of retaining structures at the toe of the slope is not an effective measure because the landsliding body is not a rigid block which could be held at its base. Furthermore in the studied case no weight masses could be placed at the toe of the slope (directed to the increment of the strength in this zone) due to the lack of space (the initial excavation was carried out to get an esplanade).

After the failure has been reached, the stability state of the slope can not be never the same as before the movement, the material has already reach its yield point, that is, non recoverable deformations affect the shales in the failure plane.

Due to the above considerations, the measures for the estabilization of the landslide were carried out in different steps and based on:

- the knowledge of the deformational process
- permanent control of the evolution of the slope
- results of the analysis carried out to modelize the behaviour of the slope for different conditions
- design of the corrective measures based on the response of the slope to the previous corrective actions.

The curves of the evolution of the movement (Figure 3) show the effectiveness of the corrective measures carried out in the 1st stage (1990). Later, the analysis to modelize the behaviour of the slope indicated the necessity to carry out aditional drainage (deep drainage) measures and the removal of material in determinated zones of the slope (2nd stage during 1991 and 1992).

In the 2nd stage the removal of material was carried out in the upper and medium part of the slope, in order to avoid the influence of the weight of the material on the toe, which constitute the critical point of the landslide.

Since 1991, the slope has not been affected by remarkable movement even during heavy rainy periods (Autum 1993 was specially rainy in the region). The superficial displacements represented in Figure 3 for 1991 to 1993 indicate that the slope is affected by a "residual" deformation, which do not represent any actual hazard for the slope.

This stability state has been the result of the different corrective measures carried out related to the geometrical changes in critical zones (which result in a different distribution of the weight in the slope), and to the decrease of the water content into the rock mass (which result into an increase of the material strength and the decrease of the weigth of the materials).

From the analysis carried out, it has been deduced that the decrease of the water level below the failure plane, implicate almost a 10% increase in the factor of safety. However, this actuation involve some problems associated to the operative difficult to evacuate the necessary flow to decrease the water level, the efficient capture of the water due to the low permeability of the materials (the water flows along fractures and discontinuities, and it is difficult to capture it) and the problems associated to the upkeep of the drainage systems and to the response when heavy rainfall occur.

With respect to the removal of material in different parts of the slope, the difficulties are associated to the accessibility for the machinery and to the volume that must be removed to increase the factor of safety: a 10% increase implicate, after the excavation carried out in the first stage, the removal of 250.000-270.000 m^3.

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