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Failure Slope in Clay Shales in Aratu Bay - Brazil

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SUMMARY Three slides at the same slope, 35m high, on clay shale are analysed.Geological and geotechnical features are shown. Back analysis from the last two slips and sensitivity analysis were done.Good predictions and miskates are shown.

INTRODUCTION

During the construction of the road system of Aratu harbour in state of Bahia, there were several slides in shales and residual soils along it. One of them, the most interesting and challenging, took place in a cut slope 35m high that failed three times and was re-designed twice.

The former was on may 78, when the slope was cut to enlarge the highway passage. At the top of the slope there is a railway terminal, the projection of which would cross the highway line design, fig.1. This slip has happened at the upper half portion of the slope with a volume lower than 20 000 cubic meters. The railway was left intact.

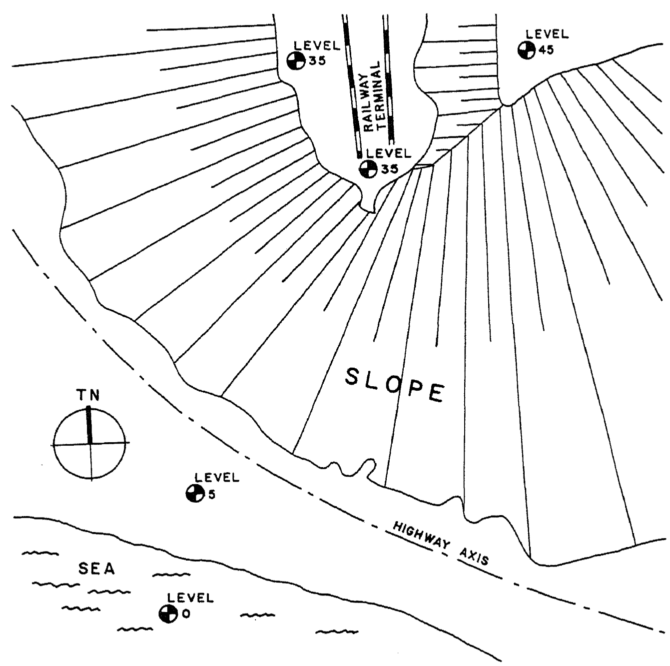
The first re-design had recommended a flattening, of the slope, an embankement compaction upon a drainage blanket at the top, fig.2,and some superficial drainage measure indications. At the beginning of the flattening a long crack appeared crossing the railway. The design was changed and the slope was more flattened. During the work two minor slides (2 000 cubic meters each) occurred. One of them near the crest and the other, with a shallow and circular shape, at midheight.

At the and of the work, a large dark spot, indicating presence of water, appeared at a midheight point of slope. Two weeks later, on september 28, 1978 the second slide occurred with a volume of 400 000 cubic meters.In spite of a bad whether, there were no heavy raining day during the last month.

The cracks were covered with clay and compacted by hand because it was inaccessible to machinery. Levelling and a detailed geological exploration were been done when, on january 79 —after two raining days (81 and 70 milimeters), the third and last failure occurred. The appearence of the slide was quite different from the others. One point near the top had been carried away more than 40 meter horizontally and down a few meters. It appeared as if the whole slope had been lifted up by an explosion and moved downhill.

The harbour authorities, few months later, decided to modify the highway line design, by loosing some area and by stabilizing only the broken soil-rock mass, that is still in place, until today (1983).

Fig. 1



Sketch of the slope situation

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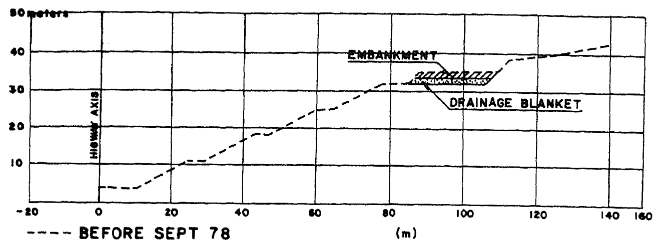
Fig. 2



Typical cross section of the former re-design

GEOLOGICAL SETTING

Aratu harbour is located in a region called Recôncavo, composed of magmatic and sedimentary rocks as conglomerate, siltstone sandstone, shales. The most important geological attribute is the fault presence with a general strike N10°E and N15°E. The west side of this fault is formed by sedimentary rocks from jurassic and cretaceous; the east side: pre cambrian embasement of gneiss. Covering these two groups of rocks are the clayey sand and gravely sand of Barreiras formation from tertiary. Its horizontal and subhorizontal layers indicate no tectonic effects since its deposition. All of these sedimentary formations has been formed in a typical non marine enviromental.

The slope foccus of current analysis is sedimentary from Ilhas formation. Upon this were in the past São Sebastião formation 1 000 to 2 000m thick or more from cretaceous too. As time goes by, erosion took place removing S. Sebastião formation upon Ilhas, exposing it and, later in tertiary, deposition of Barreiras formation took place.

EXPLORATORY DATA

The first slide was a top failure on Barreiras formation which had not disturbed the other layers beneath it, from Ilhas. Hence it was possible to make measures of strike and dip in these untouched layers, that later on has been moved by the second slide.

The slope can be divided into three sequences: the upper, the intermediate and the lower sequence, fig.3b. The upper sequence is formed by sandstone and siltstone with short intercalated layers of shales, with an overburden of residual soil. The other two sequences are composed of shales with interbeded layers of siltstone and sandstone. The clay mineral predominances are:

- upper sequence - rich in kaolinite with some montmorillonite.
- Intermediate - rich in montmorillonite and some kaolinite and micaceous minerals.
- Lower sequence - predominance of kaolinite some montmorillonite and carboniferous materials at some levels.

These information suggest a visualization of a more intense flow of water at the upper and intermediate sequence. The presence of montmorillonite increases the possibility of shorter spacing of fractures in these zones.

The systems of fractures on sandstone in the upper sequence (fig.3) with the bedding, during the excavation could form joints that filled with rainning make it easy toppling failure and rock fall.

The second stage of the geological survey was done after the second slide and its results are presented in fig.4. Photo from this slide are presented in fig.5. The large discrepancy betweem the bed dip and strike measures, taken from the first survey stage (fig.3) and those taken from the second stage (fig.4), rise the need to emphasize the high degree of discontinuities in so restrict area.

The last stage of the geotechnical/geological exploration was the openning of pits on may 79, whose positions are shown in fig.4. Seven were open an the slope, three had springs from slided material, another one from untouched material and the other three were completely dry. This fact reflects the difficulties to get efficient drainage in such material. Sowers (1976) wrote some remarks on similar problems. These pits were thought to be used for dewatering.

The three pits on the railway terminal showed a "very fractured and very weathered" zone 7.0 to 9.5m deep, CEPED (1979). Under this zone there is a 7m thick layer of clay shale with oxidated fracture planes. This is a strong indication of water flow. Beneath these layers is a shale lightly weathered. These observations confirm the relation between expansive clay minerals and fractures mentioned by other authors, e.g. Bjerrum (1966). They also confirm the prediction on water flow zone from the first exploratory stage.

From the second slide, the following observations, very usefull for back-analysis (next section), were recorded:

a) During drilling the water level from borehole 8 had a sudden drawndown 1.5 meter suggesting the presence of a perched water table.
b) The same borehole as transformed in water level indicator, dropped 8.5 meters on its water level at the time of the second slip.
c) The open cracks, larger than 0,5m, were subvertical reaching five meters deep.
d) Translational and some rotation were the nature of slopes motion. No top failure was found at any high. The toe has advanced into highway lane.
e) In some cracks, seams were found. These seams were 3cm thick, of high plasticity clay (Liquid limit= 60; plasticity index = 30).
f) For a long time even before the first slip, water pools have been along the toe slope.
g) Sometimes from a day to another a top part of the slope had been moving down hill. The visual inspection really gave the sensation that some trees were kept away. No measures were made in order to evaluate it.

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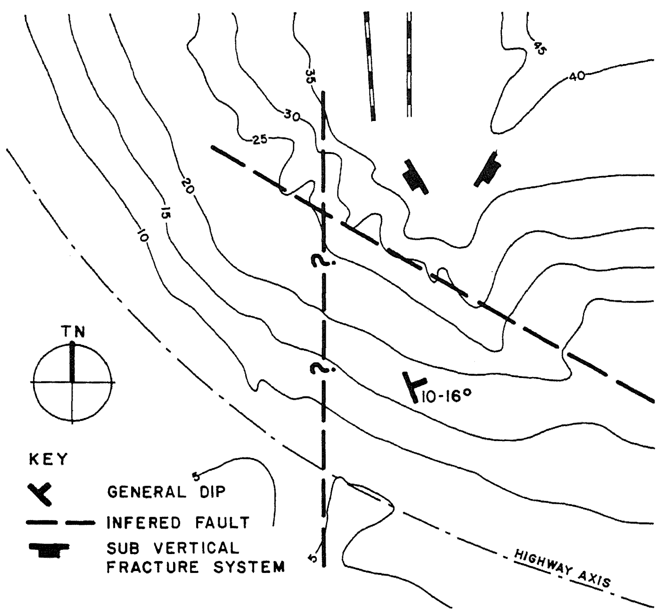
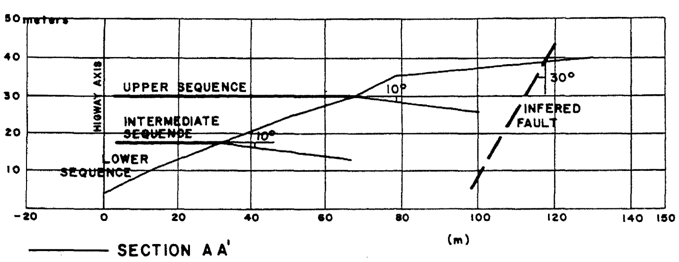
Fig. 3a



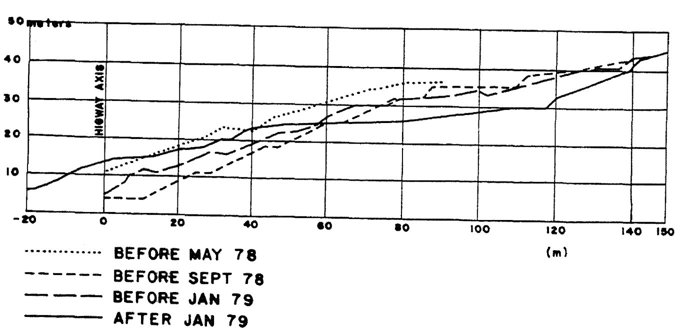
Location of the inferred fault

Fig. 3b



Typical slope profile assumed from surface inspection

Fig. 3c



Typical sections from each slides

ANALYSIS OF THE SLIDES

No shear test was carried out on slope material because there are no field and laboratory data at disposal on these shales from Recôncavo. Hence, there is none correction factor to be applied on laboratory results in order to get predictions near field behaviour.

First Re-Design

The second slide occurrence suggests an under-estimate of the inferred fault presence on first re-design studies. Slope stability estimation from block analisys, considering the dip bedding only measured on slope surface, leads to high factors of safety, although lower strenght parameters had been assumed, fig.6. The election of plane failure method is due to the following field observations:
a) All other failures on this material had its failure surface represented accurately by two planes.
b) These planes were passed through discontinuities being subvertical and subhorizontal.
c) Usually these discontinuities were filled with high plastic clay, Guimarães (1980).

The block analysis presented by Esu (1966),who considers discontinuities filled with water and moment equilibrium, furnished no satisfactory results. These analysis and the second slide leads to the conclusion that the existence of the fault reflects possible changes on dip bedding inside the shale mass.

Back-Analysis

Back analysis were performed based on the following assumptions:
a) The slip surface is composed of two intercepting planes.The former is subhorizontal intercepting the toe, the last is vertical passing through the last top crack. The subhorizontal plane intercepts the vertical plane at the same water level from borehole 8 measured after the second slide, fig.7.
b) Valid Mohr-Coulomb shear strength envelope.
c) The state of stress is uniform for all points of the failure surface.
d) Hydrostatic pressure distribution as in fig. 7.
e) End restrains effects neglected.
f) Possible effects of the state of tension on the slope neglected (refers to those analyzed by Chowdhury, 1976).

The back-analysis results are shown in fig.8 in which a few different assumptions were considered. The most probable c' and Ø values are:
effective cohesion $c' = 0 - 15 kN/m^2$
effective angle of internal friction $Ø = 13\text{-}17°$.

In fig.7 shows the positions of field slip zone and the failure plane assumed for back analysis.

Variations on this plane, making it pass through failure zone and through the water level after failure, have little effects on shear strength parameters (c' and Ø').

Back-analysis carried out on slided section before the last failure, show a residual friction angle upper 7° and lower 13°. This range is

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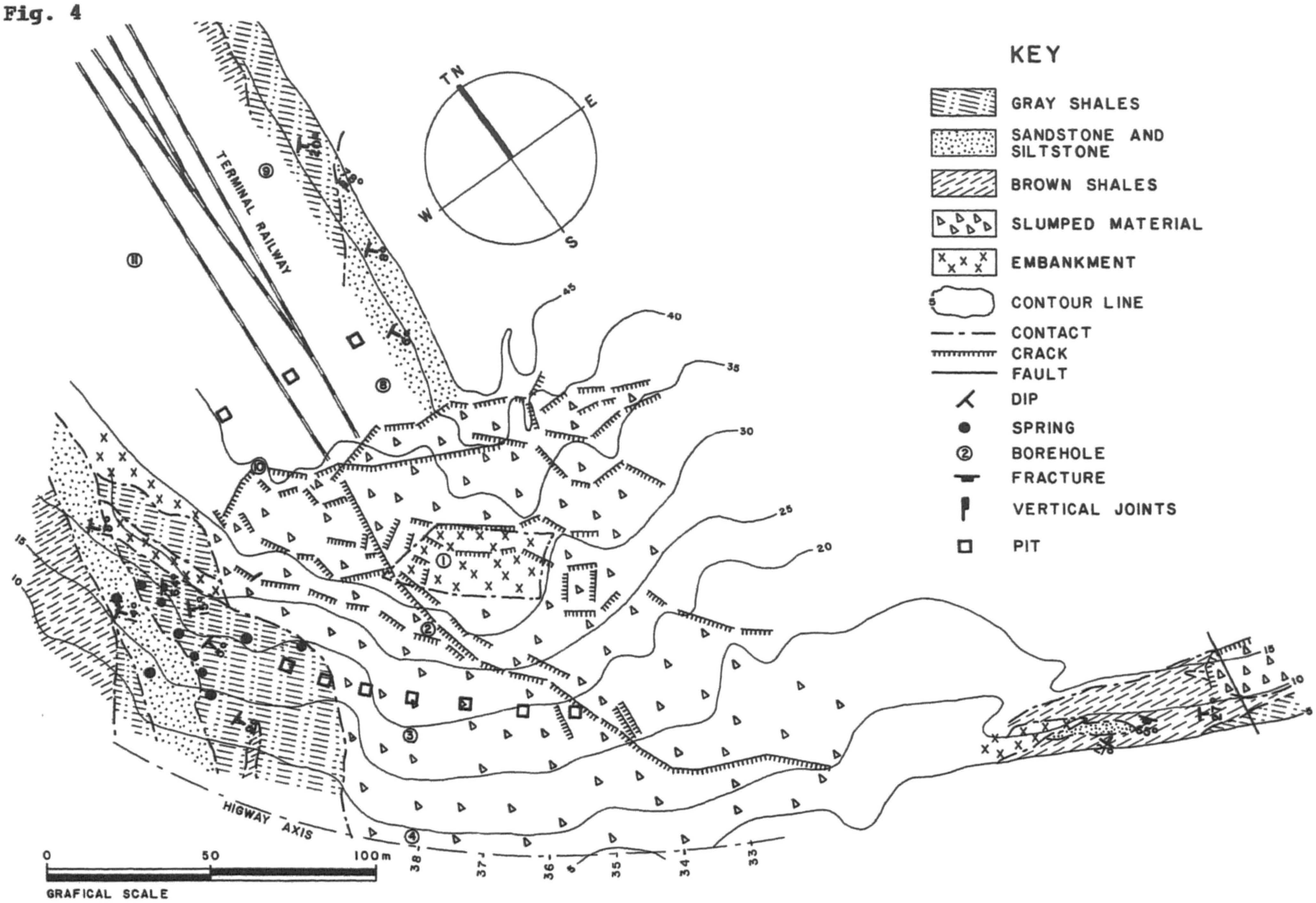
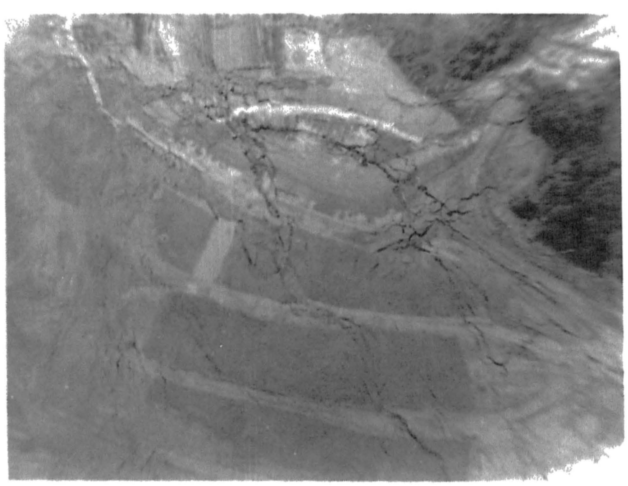
Fig. 4



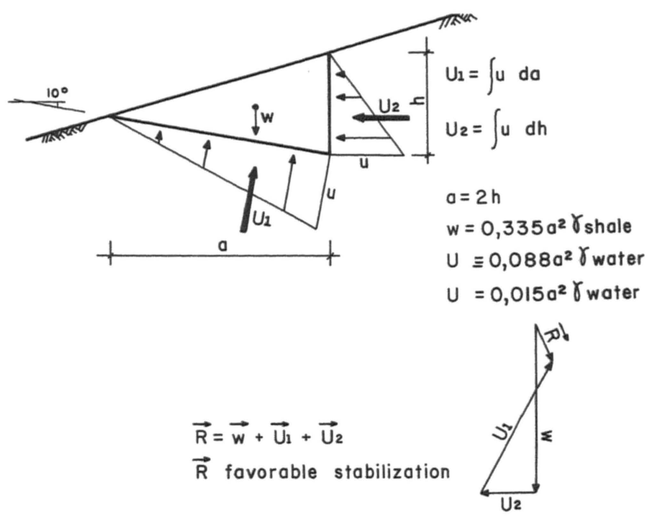
Geological map

Fig. 5



Aerial photo sliped area

Fig. 6



Stability analysis

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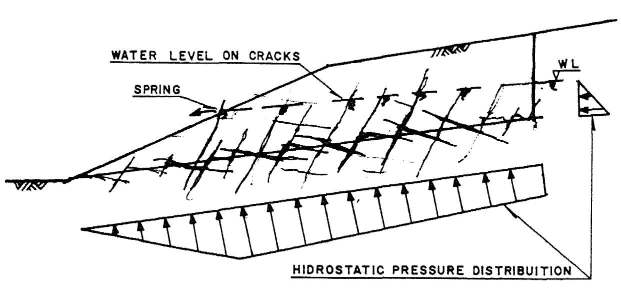
due to the great uncertainly of water level into the cracks. The lower value is due to the hypothesis of pratically zero hydrostatic pressure into cracks. This is the lowest boundary value, probably irrealistic.

Sensitivity Analysis

This kind of analysis applied to slope stability consists on keeping unchanged all factors except one of them and on evaluating the consequences of its variations on the factor of safety. This work must be done for all relevant parameters. Doing so, it will be possible to decide which factors must be changed in order to increase the factor of safety. This is possible, for instance, by flattening a slope, making a draw down or loading the toe. The term "sensitivity analysis" was taken from economy which has the same meaning. Sometimes "change-of-conditions" is also used, more over "sensitivity" alone is employed by Hoeck and Bray (1977). The importance of sensitivity analysis has been restricted for this slope because there are few parameters that could be changed on field. Height is changeless, flattening also because it is necessary to keep the railway unthouched. Hence it was left over as stabilizing factors, dewatering and retaining walls or other methods for sustaining part of the slided material. The toe loading is a doubtful stabilizing process because of the uncertainties of the residual friction angle value. If it is lower than slip plane angle, surely, loading any part of the slope will lessen the factor of safety. Thus, dewatering and retaining wall construction were the solutions recommended. To avert raining water infiltration through the railway terminal, that has been a formidable catchment area, a sealing betuminous coating was made between the residual soil and the rail way gravel blanket base. For similar reasons a covering of compacted clay upon the failure mass was recommended.

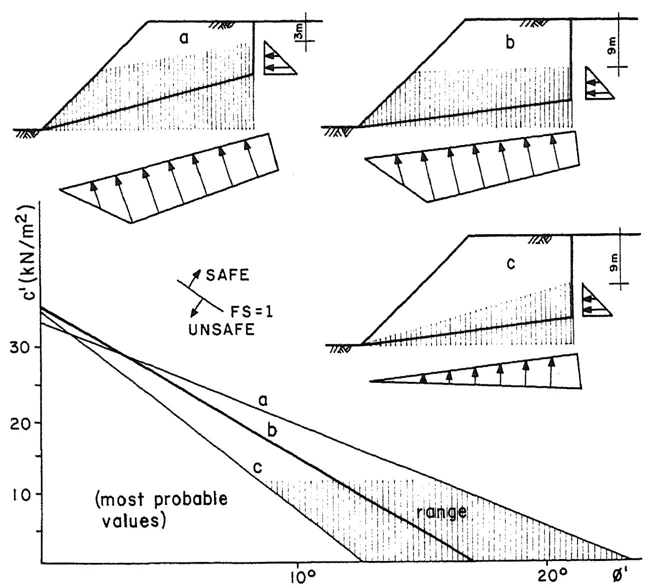
Later on, after the modification of the highway design, it was decided not to make any retaining wall, stabilizing only the slided material.

Fig. 7



Back-analysis assumptions

Fig. 8



Back-analysis results

CONCLUSIONS

From 7 opened wells on slided material three were dry and one had spring only in unthouched material. This fact reflects the difficulties to make dewatering and to determine water flow on the materials, two very important things in any stability analysis.

The first slide was a toppling failure. The second was a delayed base failure due to water pressure equalization.

A fault presence was underestimated on the first re-design.

The oppening of test pits or trenches are the best ways to sound the subsoil on this kind of material.

The author states that he has been very puzzled by the slope shape after the third slide and he does not exclude the possibility of a release of the recoverable strain energy from shales mass. This strain released energy could be possible due to the occurrence of second slide. Although there is other possible justification.

Another explanation could be the hypothesis that the residual friction angle is lower than the slip plane.

The failure plane position prediction using Hoek and Bray (1977), charts furnished an excellent agreement with the availiable field data. Such prediction was put in a technical memorandum, Guimarães and Machado (1979) before any collection from evidences of plane failure position were made.

Because of the occurrence of two slides assumed on the same failure plane, it was possible to estimate the shear strength parameters.

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A good correlation in predominance of expansive clay minerals a number of fractures and evidences of water flow inside the shales masses were observed.

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