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Landslide During Construction Of A Rockfill Breakwater In The Coast Of Sergipe, Brazil

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SUMMARY: On October 12th, 1989, a rockfill breakwater under construction in Sergipe state, Brazil, situated some 2,4 km from the beach, suffered a foundation slide failure. The breakwater had a total height of 16 meters, with 6 meters above sea. The soil profile, from sea bottom down, was 4 meters of sand, 7 meters of soft clay and below the soft clay, sandy layers to great depth. During the construction period that anteceded failure there was monitoring of displacements and pore pressures. A modified design has been conceived and construction was completed in October, 1992. Geotechnical instrumentation has been used to monitor the construction of the breakwater before and after failure. This work reviews the sequence of events, briefly presents the geotechnical characteristics of the foundation, including the fact that there was artesian pressure of unsalted water at the base of the soft clay, describes the instrumentation results before and after the failure, discusses its possible causes and, at the end, presents the criteria used to ascertain acceptable safety.

KEYWORDS: geotechnical instrumentation, soft clay, stability analysis, consolidation, embankment, breakwater.

1 INTRODUCTION

On October 12th, 1989, a rockfill breakwater under construction some 2,4 km from the beach, in Sergipe state, Brazil (see location and main components in figure 1), suffered a foundation slide failure. The breakwater had a total height of 16 meters of which 10 meters under water, and rested on 4 meters of sand (the "upper sand"), 7 meters of soft clay and below the soft clay, sandy layers. A modified design has been conceived and construction was completed in October, 1992. The breakwater has operated without geotechnical problems since then.

The present paper reviews the sequence of events, briefly presents the geotechnical characteristics of the foundation, describes the instrumentation results before and after the failure, discusses its possible causes and, at the end, presents the criteria used to ascertain acceptable safety.

Figure 1. Location and satellite image showing the main components of the works.

2 SEQUENCE OF EVENTS

The access to the berthing facilities and to the breakwater is made through a 2,4-km long bridge, as shown in figure 1. The design wave height was taken as 4 meters and tide (which peaks twice a day) amplitude (between maximum and minimum sea levels) considered as 2,64 meters.

In the breakwater location there is a soft clay layer that, from a geotechnical point of view, brings concern about settlement and stability issues. The characteristics of the clay are detailed on the next item.

The initial design of the breakwater established a cross section (see figure 2a) composed of a rockfill mattress placed with a side dump vessel (this stage of construction was called “maritime phase” which received the denomination of “geotechnical berm”) and a rockfill embankment placed with trucks (called “terrestrial phase” and named “hydraulic berm”). After failure, a final modified design was adopted, in which the “geotechnical berm” was widened and the “hydraulic berm” was made wider at the crest and was built with smoother slopes (see figure 2b).

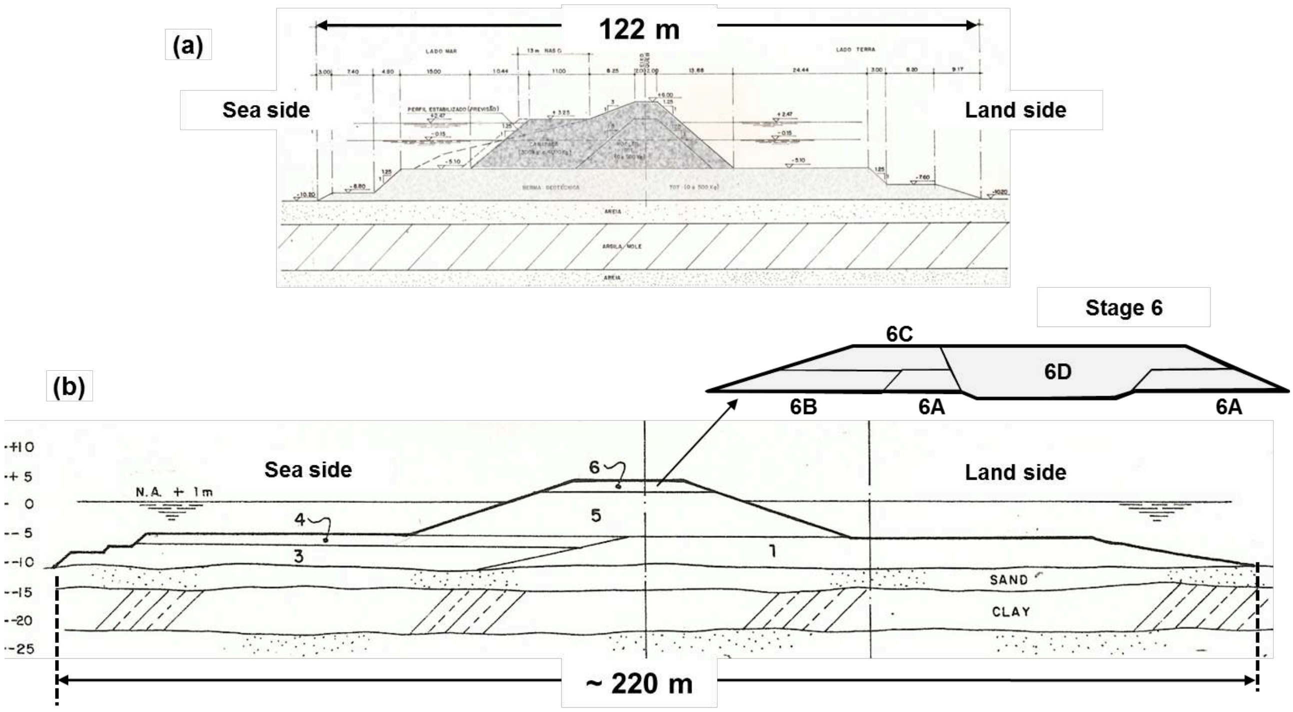

Figure 2. Designs of the breakwater: (a) initial; (b) final (showing the sub-stages of Stage 6). Stage 2 is the terrestrial phase before the failure and it is not represented here because the present section is not the failure one.

The main construction events were (see figure 3):

- The geotechnical berm of the initial design, which had a thickness of 5 meters (a minimum depth of 5 meters was needed for the side dump vessel), has been placed between October, 1988 and March, 1989. The rockfill was loaded in the side dump vessel at the provisional port (PEP, see figure 1). There was 7 months span from the placement of the geotechnical berm and the beginning of the hydraulic berm, which allowed some strength gain of the clay under the weight of the 5 meter thick submerged rockfill mattress.
- Placement of the terrestrial phase of the initial design began in August, 1989. Monitoring has been carried out at stations STA1 and STA2 (see figure 4);
- Failure took place on October 12th, 1989 when about 100 meters of the terrestrial phase had been built (see figures 3 and 4);
- Before failure, the (initial) design was developed by a Design Firm and checked by consultants hired by the Contractor;
- After failure a series of studies (geotechnical, hydraulic, logistics) have been carried out to develop a safe (final) design. The geotechnical investigation and the studies for final design, carried out by the Contractor, had the supervision of the first and second Authors and counted with the consultancy of Prof. Charles C. Ladd;
- The construction was resumed by widening the geotechnical berm (stage 3, see figure 2);
- Stages 4 and 5 (see figures 2 and 3) have been placed between December, 1990 and October, 1991 and have been monitored with stations STA4, STA5 and STA6 whose position is indicated in figure 5;
- After a waiting period, the last stage has been constructed in four sub stages (6A, 6B, 6C and 6D, see figures 2 and 3) between March and October, 1992;

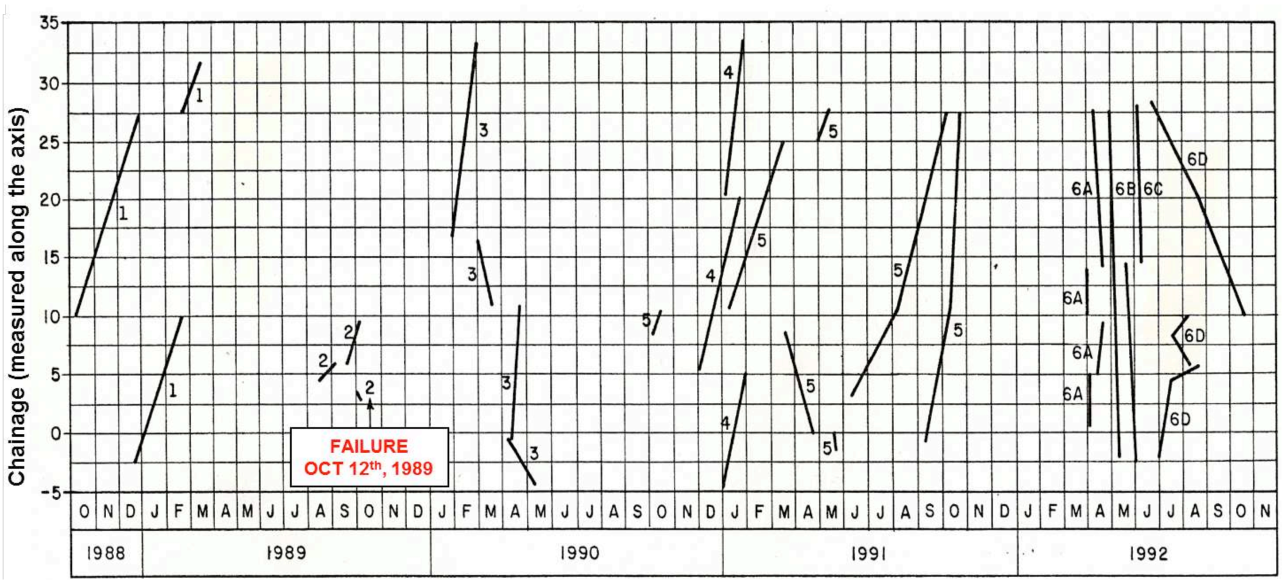

Figure 3. Time-path diagram of the construction stages: initial design (before failure, stages 1 and 2) and final design (after failure, stages 3 to 6). See Figure 5a for a plan view of the chainage.

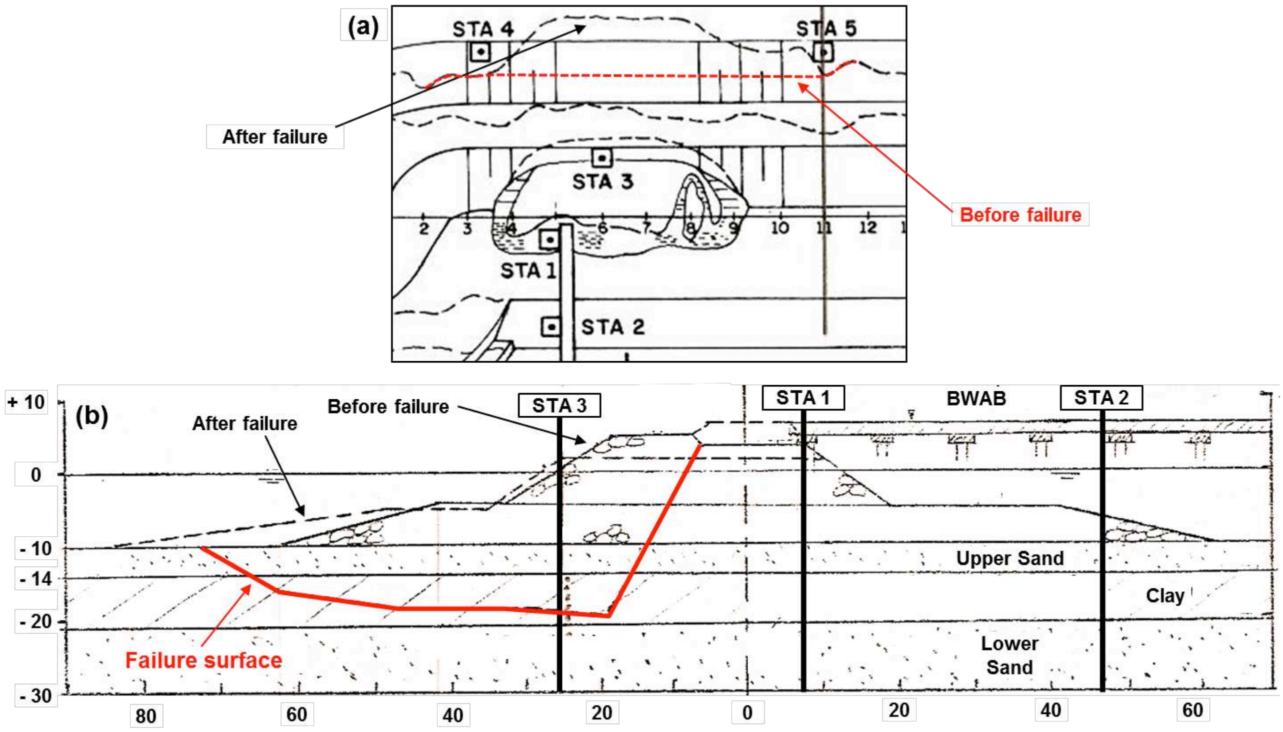

Figure 4. Initial design and failure surface: (a) plan view; (b) section.

- Inclinometer readings continued until December 19th, 1992, and crest settlements continued to be measured until April 5th, 1994;
- No geotechnical stability problems happened during construction and no excessive settlement has been reported.

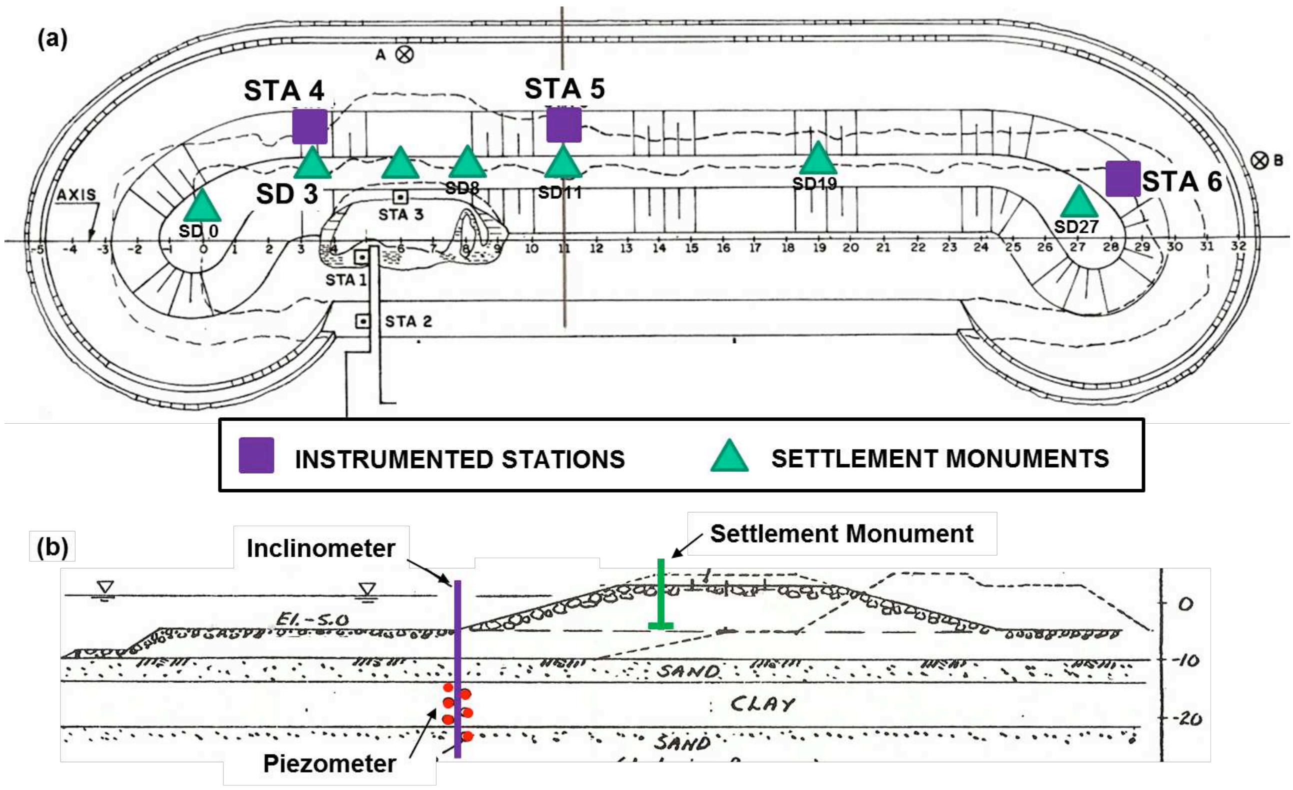

Figure 5. Layout of instrumentation: (a) plan view; (b) typical section.

3 GEOTECHNICAL CHARACTERISTICS

Thin tube piston sampling and field tests have been carried out at points A, B, C and D (see figure 6a). The results of piezocone tests (driving velocity 20 mm/sec, area of point 10 cm²) are summarily reproduced in this figure. It is worth noting that the results are remarkably uniform (values of qT, fs and Bq in very narrow intervals, in spite of the distance of hundreds of meters between the points).

The material of main interest is the soft clay layer between elevations -14 m and -21 m that controlled the failure. A typical geotechnical profile under the breakwater is shown in figure 7.

Above and below the soft clay layer there are sandy layers. The 4-m thick upper sand above the soft clay has SPT 6 to 15 and grain size with 50% to 70% sand, 20% to 40% silt and less than 10% clay size (less than 2 µm). The lower sandy layer, in the region just below the soft clay, has SPT larger than 8 and grain size with 60% sand, 30 % silt and 10% clay size.

The soft clay layer has SPT typically equal to 1/45 cm and grain size composed of 42% clay size, 40 % silt and 18% sand. Figure 7 also presents characterization tests of the soft clay layer (water content, liquid limit, plastic limit, liquidity index and salt content). Liquid limit has been determined without pre-drying the samples. The plasticity index (PI = LL-PL) of the clay has a mean value of 35 % and varies between 19 % and 46 %. The liquidity index values are above unity in several points. Pore water salt content varies from low values (2 to 4 grams/liter) in the bottom half of the layer to higher values (10 to 15 g/l) in the top half, but always quite smaller than the average sea water salt content which is around 36 g/l.

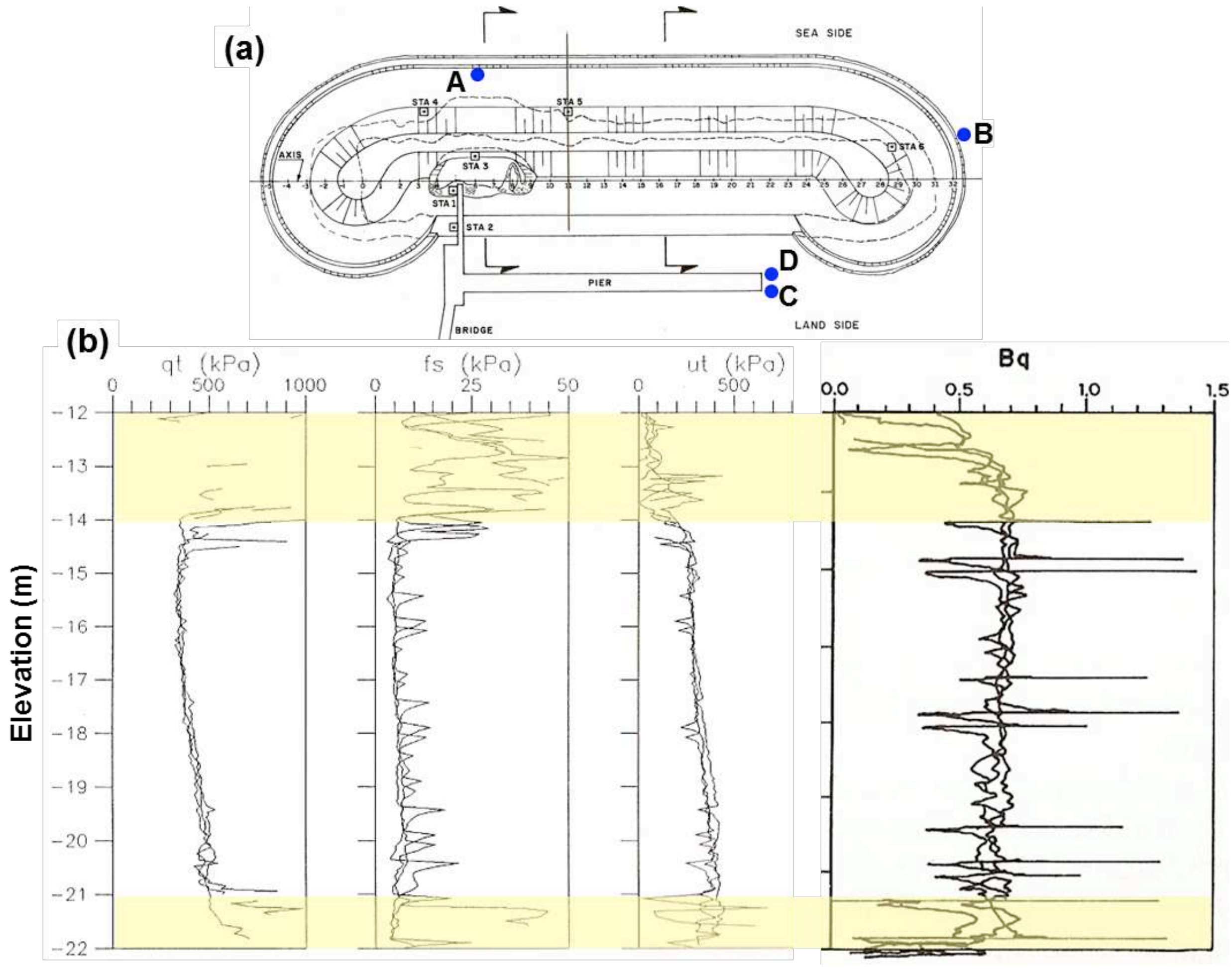

Figure 6. (a) Position of sampling and field tests (Dotted line in plan is the foot of the initial geotechnical berm); (b) Piezocone results at points A, B and C/D plotted together.

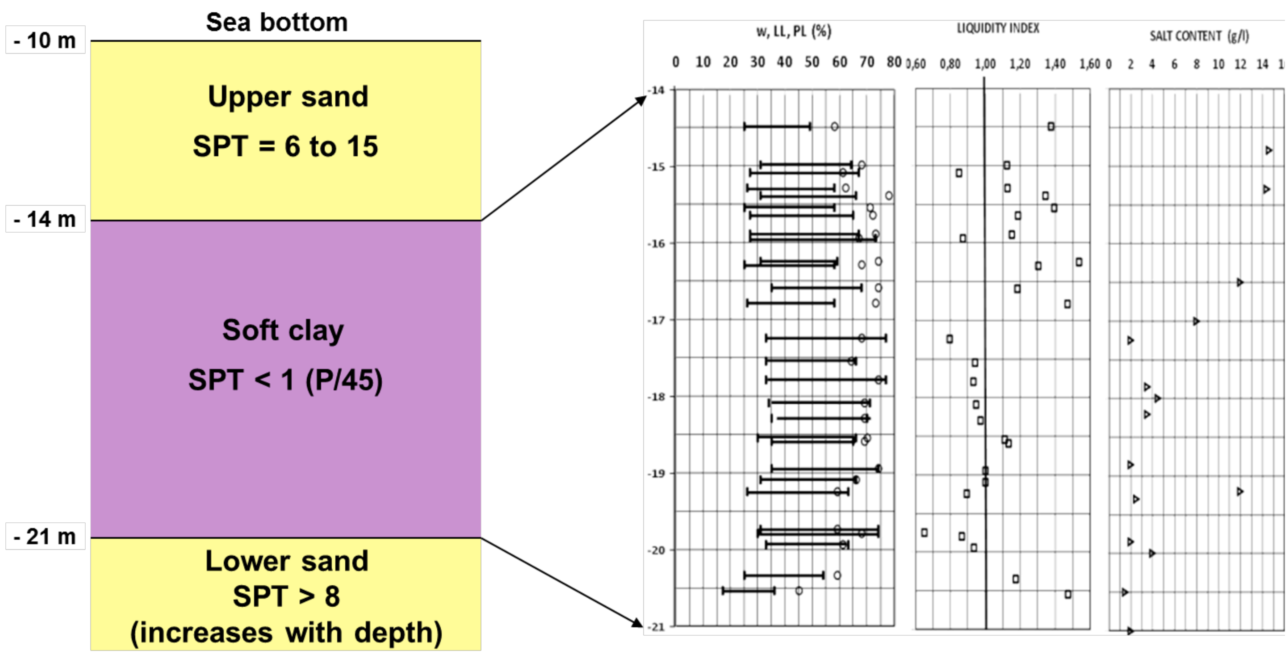

Figure 7. Simplified geotechnical profile and geotechnical characteristics of the clay under the breakwater.

More about the laboratory and field tests and about the geotechnical characteristic the clay layer can be found at Sandroni et al. (1997).

4 MONITORING BEFORE FAILURE AND LOAD TEST

4.1 Monitoring Before Failure

Monitoring of the construction of the terrestrial phase has been carried out at stations STA1 and STA2 (see figure 4) using inclinometers and vibrating wire piezometers. Both these stations were installed at the breakwater access bridge (BWAB, see figure 1) in the land side of the breakwater.

Excess pore pressures measured with open tube piezometers indicated an artesian excess pressure between 1,50~2,50 meters at the base of the layer (elevation -21 m) and zero at its top (elevation -14 m), as shown in figure 8.

It is possible that part of this excess pore pressure is remnant of the load of the geotechnical berm, although it is considered that this part would be very small.

The horizontal displacement and the pore pressure measured in some piezometers at STA1 in the months prior to the failure are presented in figure 9a.

It is worth mentioning that the tube of the inclinometer at STA1 was spiraled due to manufacturing defect, which was providing incorrect readings. In view of this, the values presented in figure 9 result from a correction and show large horizontal displacements between August 18th and October 2nd.

The excess pore pressures presented in figure 9b are coherent with the stages of heightening of the breakwater.

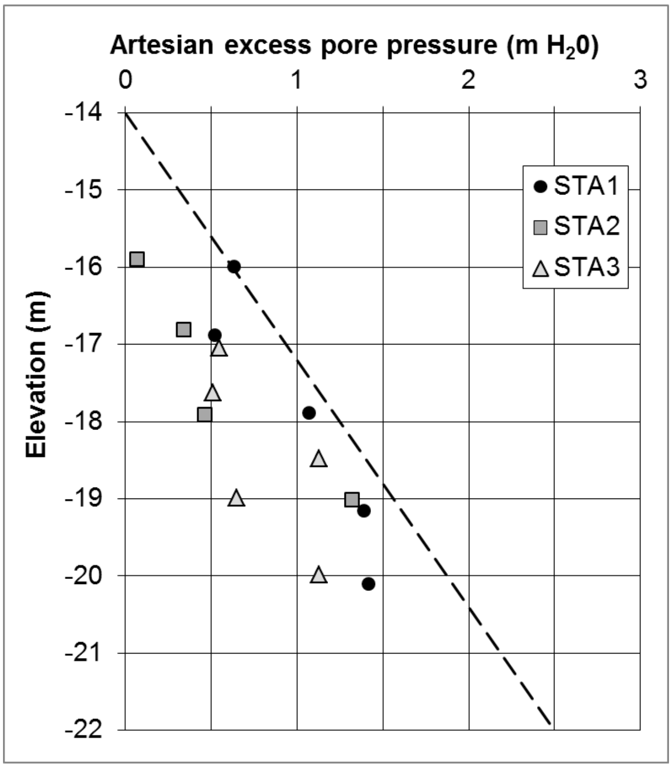

Figure 8. Measured excess pore pressures across the soft clay layer.

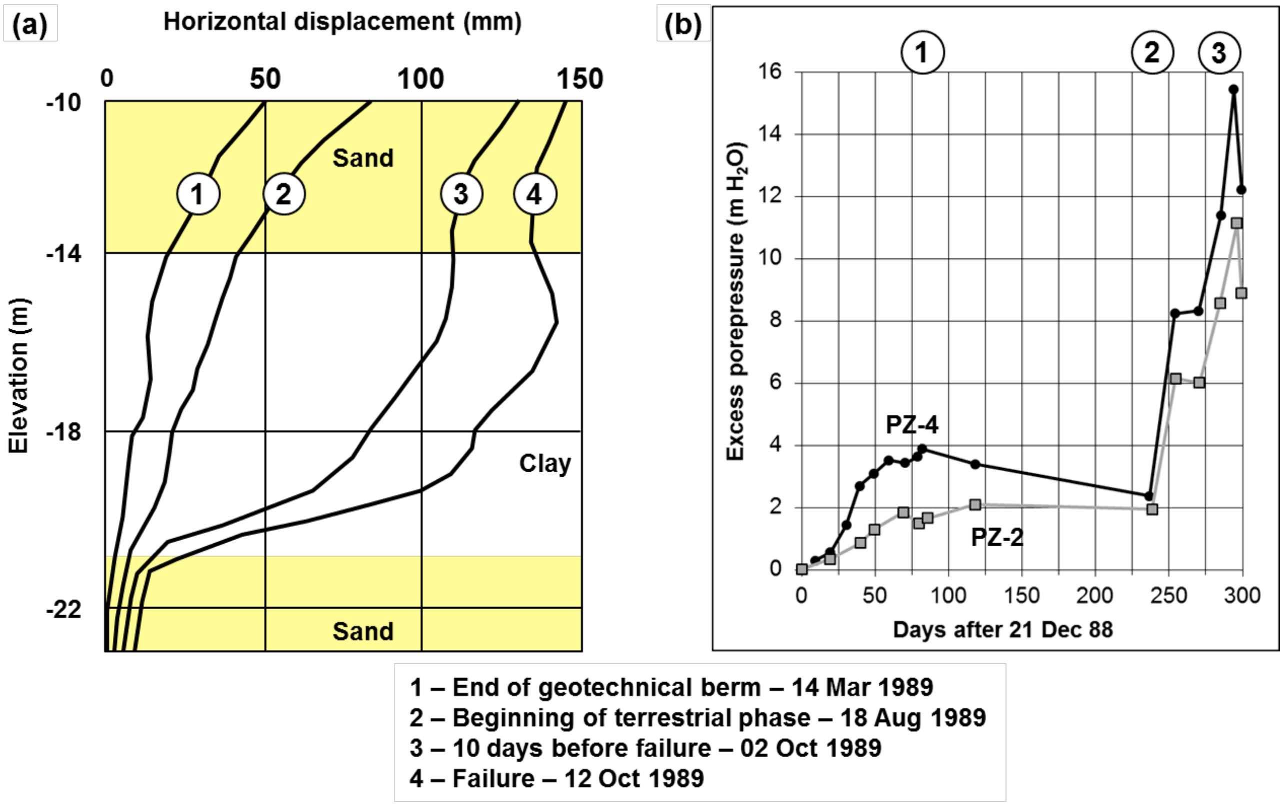

Figure 9. Monitoring before failure at STA1: (a) Horizontal displacements; (b) excess pore pressures.

4.2 Monitoring During Field Load Test After Failure

Soon after failure, in March, 1990, station STA3 with one inclinometer and six VW piezometers (see figures 4 and 10) has been installed to verify if the failed mass had stopped moving and also to observe the position of the failure surface.

To induce additional movement, a 2,5 meters thick, 25 meters long and 50 meters wide fill was placed over the part of the failed mass that remained above water, as sketched in figure 10.

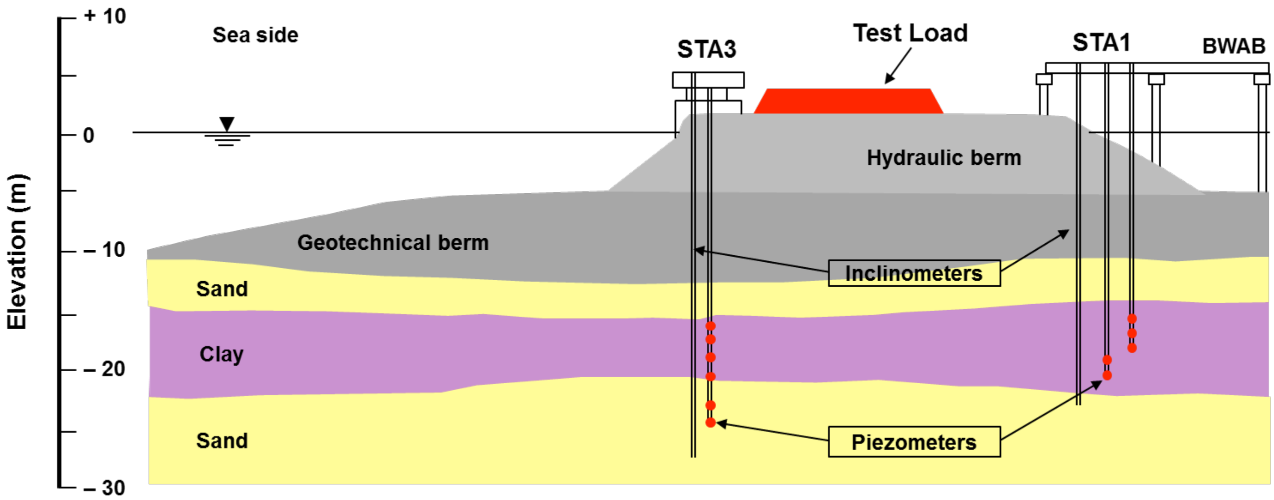

Figure 10.Load test after failure and details about STA3.

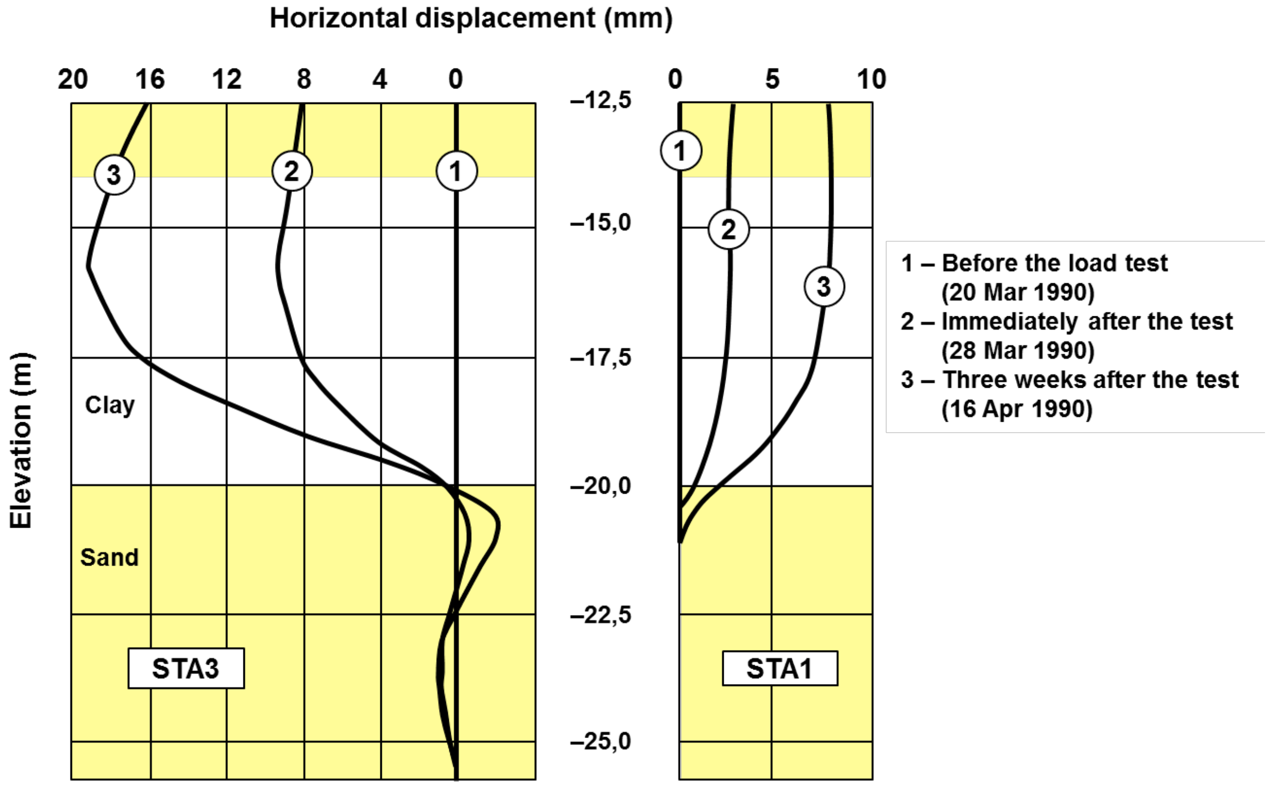

Figure 11. Horizontal displacement before and after the load test.

The horizontal displacements measured at STA3 are presented in the left hand side of figure 11, and (together with topographic and bathymetric surveys) helped to establish with confidence the position of the failure surface shown in figure 4b (and also the extent of the failed area shown in figure 4a.

The observations also showed that displacements would be tolerable for the access bridge (BWAB). In the right hand side of figure 11 the horizontal displacements measured by the inclinometer at STA1 are shown.

Piezometric data of STA 3 have been used to confirm the artesianism (figure 8). About two months after the load test (in May, 1990) STA3 was destroyed by a storm.

5 ANALYSIS OF THE FAILURE AND REDESIGN

5.1 Back Analyses of the Failure

Limit equilibrium analyses, using Spencer's and Morgenstern & Price's methods codified in STABL and GEOSLOPE softwares, in terms of effective and total stresses, indicated that:

- Effective stress analyses. Considering the effective stress parameters obtained from triaxial tests ($c' = 0$; $\phi' = 25°$), the excess pore pressures observed in the piezometers of STA1 and STA2 and the failure surface obtained as described before, a safety factor equal to 2,04 was obtained.
- Total stress analyses. With the field vane tests, considering the strength increase due to six months of geotechnical berm (based on measured excess pore pressures), dividing the clay layer in seven sub layers and considering the same failure surface, a safety factor around 1,35 was obtained.

In view of the fact that the best estimates of the safety factors for failure conditions were well above unity, the redesign team carried out a number of studies to understand the reasons for this discrepancy and to establish geotechnical models for the redesign. These studies are briefly described in the next item.

5.2 Evaluation

In the search for the possible causes of the discrepancy between the high safety factors and the fact that failure did happen, the following aspects have been considered: high shear induced excess pore pressures, progressive failure and mobilized strength in the rockfill. These three aspects are discussed in what follows.

Artesianism with unsalted water seems to have induced leaching of the clay, as indicated by the low salt content of its pore water (see figure 7). Leached marine deposited clays tend to develop a metastable (sensitive) condition (see, for example, Rosenquist, 1953; Quigley, 1980; Torrance, 1983). Liquidity indexes above unity presented in figure XX seem to confirm this fact. Clays in such conditions are prone to develop high shear induced pore pressures. The high A_f values obtained in the triaxial tests (Sandroni et al., 1997) are indications that this might have been the case. Therefore, the generation of considerable shear induced excess pore pressures (which could not be registered by the piezometers) may have played an important role on the failure.

Another possible cause of the discrepancy is progressive failure, that is to say, the reduction of the effective friction angle with displacement along the failure zone (Skempton, 1964). Residual effective shear strength angle of the Sergipe clay may be as low as 8 or 10 degrees (Lupini et al., 1981). The relatively moderate brittleness of the soils observed in the triaxial tests (Sandroni et al., 1997) invites to consider that this agent may not have been important. However, it was the view of the design team that the occurrence of progressive failure should not be ruled out.

The third aspect considered to explain why safety factors well above unity correspond to failure can be called "shared failure" as shown schematically in figure 12. The instrumentation (at the landside) showed that up to a certain moment displacements occurred in both directions: towards the sea side and towards the land side. During this period, the rockfill above point A may have acted as a "liquid" in the sense that it did not develop shear strength. With the continuation of the instability process the movements increased in the sea side and failure was completed. Therefore, the strength mobilized in the rockfill up to a certain (unknown) point of the failure process is virtually impossible to ascertain. One can speculate that the strength effectively mobilized in the rockfill is much smaller than its effective friction angle.

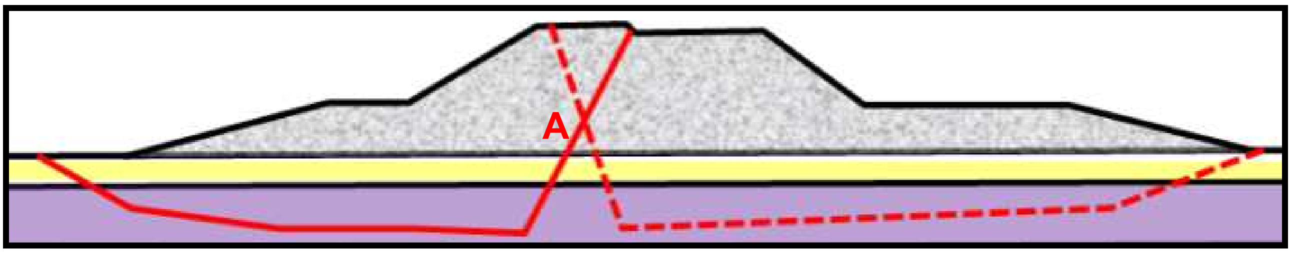

Figure 12. Schematic indication of shared failure.

5.3 Final design

The final design of the breakwater involved a multidisciplinary team composed of hydraulic, geotechnical and construction logistic specialists. The hydraulic team, with support of the geotechnical specialists, carried out models (in Brazil and in Denmark) aiming at a wider and lower crest for the hydraulic berm. By the end of this process the width of the crest of the hydraulic berm was chosen as 19,5 meters (in the initial design it was equal to 6,0 meters), the crest level (after settlements) was at elevation +4,50 m above mean sea level (it was +6,0 meters in the initial design) and the inclination of the slopes of the hydraulic berm was smoothened down to V=1:H=3 (as opposed to the 1:1 inclination in the initial design). The crest level was +5,25 m to compensate for consolidation and secondary settlements after completion of construction. The axis of the breakwater was moved 39 meters seawards and an additional maritime phase was planned such that a 45 meters wide geotechnical berm was granted (in both sides of the breakwater, counting from the foot of the hydraulic berm). This width resulted from the stability analyses described in what follows. The thickness of the geotechnical berm was kept at 5 meters because the same side dump vessel had to be used to build it. Concerns with the possibility of erosion required that a 15-m wide rockfill be placed at the foot of the geotechnical berm. Due to this, the width of the 5-m thick geotechnical berm was reduced from 45 meters to 42 meters. The initial and final redesign sections were shown in figure 2 together with the sub-stages of Stage 6.

In view of the uncertainties, presented in the previous item, it was decided that, instead of using fixed best estimate parameters, the geotechnical stability analyses would be conducted using cycles of limit equilibrium back analyses and analyses with different sets of parameters, covering a wide range of reasonable values.

A safety factor around 1,25 to 1,30 was considered as acceptable provided that: (a) geotechnical monitoring would be carried out during the whole construction period, (b) a six months waiting period would be guaranteed between placement of the geotechnical berm and construction of the hydraulic berm, (c) construction would be carried out in stages and, (d) contractual provisions would be implemented in such a way that the eventual necessity of improvements (say, halting construction to allow for longer consolidation time between construction stages and widening the geotechnical berm) would be implement.

In January 1992, after Stage 5 had been completed and before Stage 6 was started, Prof. C. C. Ladd has been retained as true expert to evaluate the design procedures and the safety conditions. He produced a number of unpublished reports (Ladd, Lee & Whelan, 1993; Ladd & Lee, 1993a and 1993b, among others) and mentioned the job in his Casagrande Conference (Ladd & DeGroot, 2003). Prof. Ladd carried out an independent stability analysis of the final design section using his best estimate geotechnical parameters and got a safety factor of 1,29 for the final elevation (before settlements:+5,25 m).

6 MONITORING DURING FINAL CONSTRUCTION

6.1 Results of Geotechnical Monitoring

Three monitoring stations, STA4, STA5 and STA6, with piezometers and inclinometers, have been installed as shown in figure 5. Seven settlement monuments with base at the top of the geotechnical berm have been placed in the positions shown in the same figure. All instruments were continuously measured by a full time technical team.

Figure 13a presents the monitoring of pore pressures in piezometer PZ-3 at STA4, installed at elevation -18,6 m. Figure 13b shows that the excess pore pressure varied in the expected way through Stage 5, decreasing as the consolidation progressed considering the existence of the aforementioned artesianism. Figure 13b also shows the result provided by the piezometer PZ-5, installed at elevation -20,0 m.

The readings of the inclinometer installed at STA4, shown in figure 14, indicate that the horizontal displacements kept inside acceptable intervals, even though large values were observed. However, during placement of Stage 6 an increase of waiting period has been recommended due to what was then considered as excessive horizontal displacement.

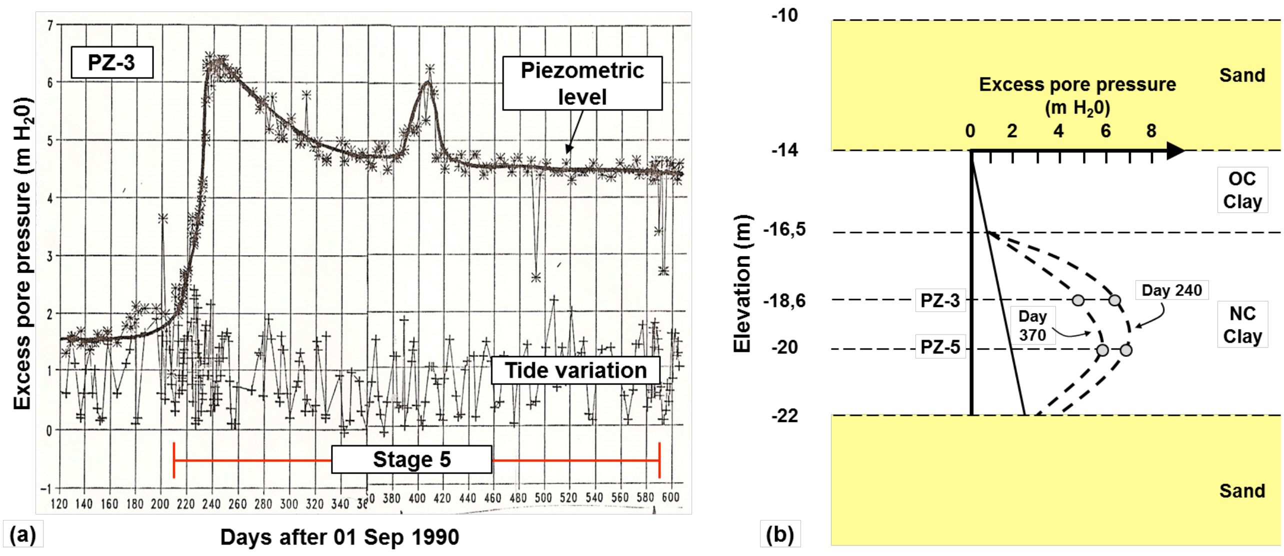

Figure 13. Monitoring of pore pressures at STA4 during Stage 5: (a) at PZ-3; (b) section showing PZ-3 and PZ-5 at different moments (estimated isochrones in dotted lines).

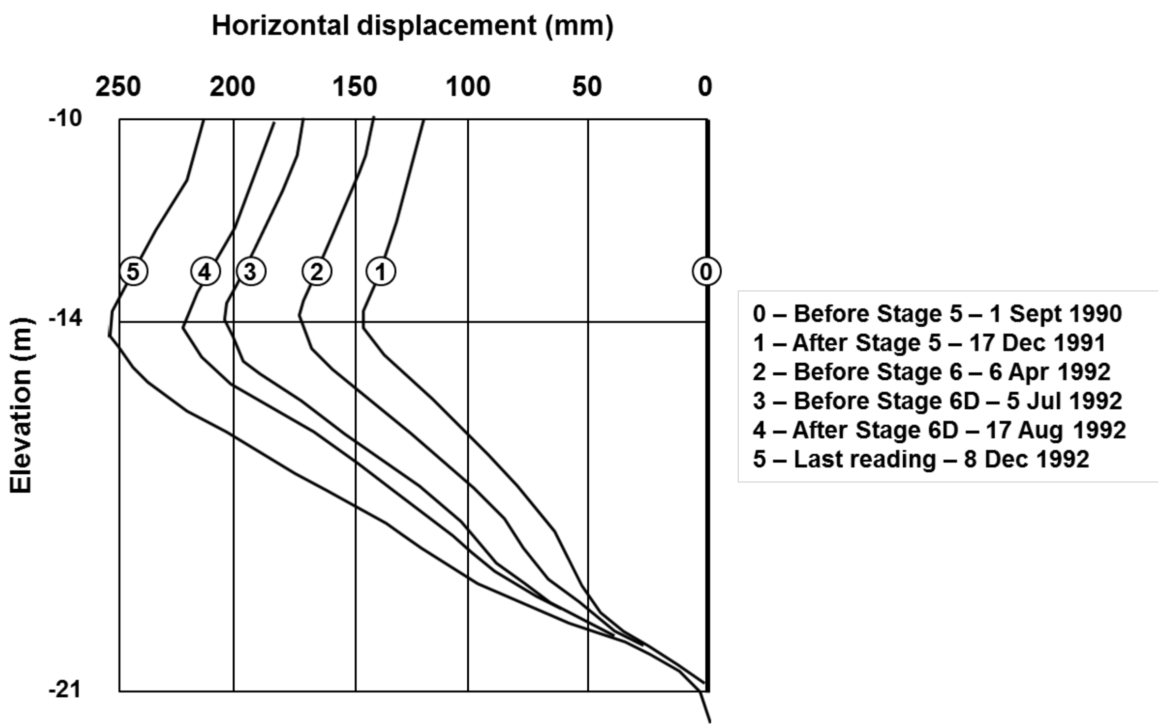

Figure 14. Horizontal displacements measured at STA4 since before Stage 5 until after conclusion of Stage 6.

Settlements in excess of 1,0 meter have been measured in all the SD towers, from the beginning of Stage 5 to the end of construction. Settlements after the end of construction were lower than 75 cm, therefore guaranteeing that the crest elevation would remain above elevation +4,25, as desired. The monitoring of the monument SD3, measured from the beginning of Stage 5 until 19 months after the completion of Stage 6D is presented in figure 15.

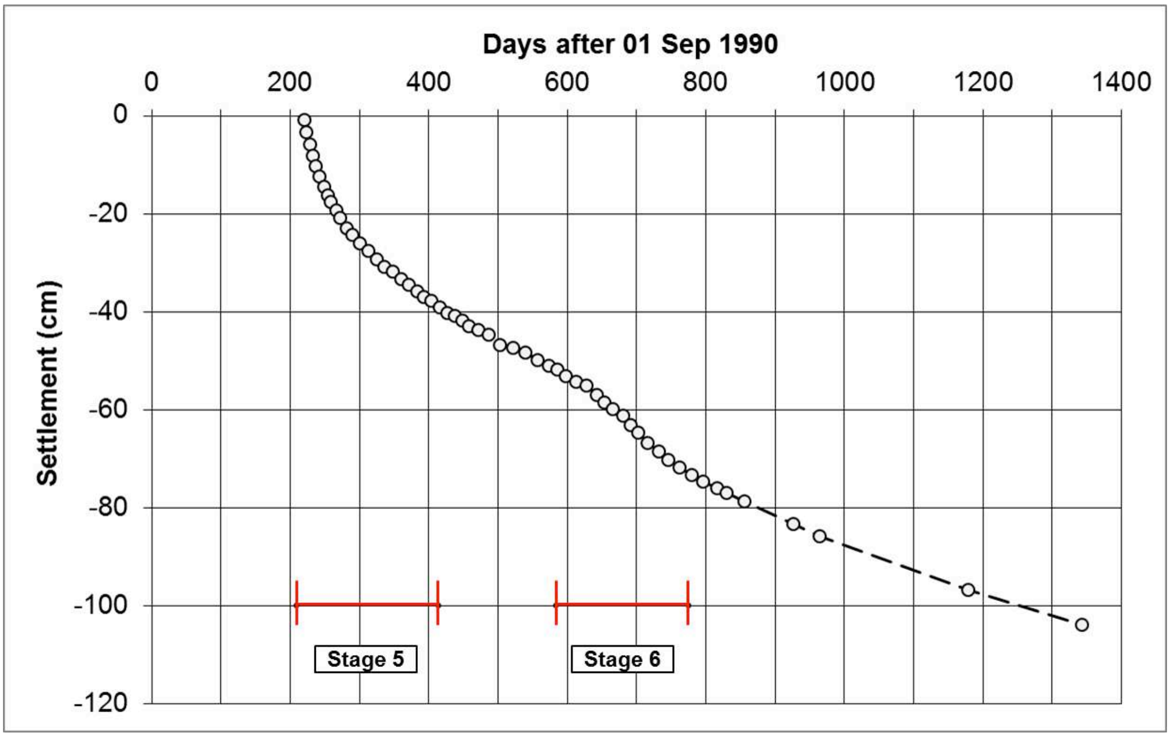

Figure 15. Settlement measured at SD3TA4 since before Stage 5 until after conclusion of Stage 6.

A representative set of monitoring results, obtained in station STA4, and comparisons with numeric estimates can be found in Brugger et al. (1998).

6.2 Criteria for Geotechnical Safety

An issue of fundamental importance was the selection of the criteria that should be applied to indicate eventual tendency towards a new failure. A number of criteria have been considered such as:

a) Values of safety factors from effective stress limit equilibrium analyses using pore pressures obtained from piezometers. This criterion has been discarded because of the "sensitive" nature of the soil remembering the safety factor around 2 obtained in this type of analyses when applied to the failure;

b) Values of safety factors from total stress limit equilibrium analyses considering the undrained strength increase based on settlements and pore pressures. In spite of probably being acceptably accurate, this criterion was discarded because of the relative complexity of the calculations involved (and the consequent delay in decision taking).

c) Limiting distortion values from inclinometer readings. This criterion was discarded because of the difficulty in choosing the limiting values in presence of drained and undrained creep that the Sergipe clay presents.

d) Limiting values of the relation between vertical and horizontal displacement volumes (Sandroni et al., 2004) $\Delta r/\Delta h$ (Δr = change in average value of crest settlement; Δh = change in average value of horizontal displacement):

- $\Delta r/\Delta h$ larger than 7 – STABLE
- $\Delta r/\Delta h$ from 5 to 7 – STABLE BUT REQUIRES ATTENTION
- $\Delta r/\Delta h$ from 3 to 5 – TENDING TO BE UNSTABLE
- $\Delta r/\Delta h$ less than 3 – UNSTABLE

e) Variation with time of the relation between vertical and horizontal average displacement values (Sandroni et al., 2004).Tendency towards stability indicated by increasing values of the relation Δr/Δh with time and tendency to instability by decreasing values of the relation.

When applied at the job, criteria d) and e) proved to be the adequate. Figure 16 shows the evolution with time of the relation Δr/Δh. As can be seen the value fell from values above 10 at the beginning of Stage 5 to values around 5,5 at the end of Stage 5. During Stage 6 the relation fell a bit further to values around 5,25 and kept stable after the end of Stage 6.

The sharp decrease in the relation Δr/Δh during placement of Stage 5 caused some concern during construction and resulted in the introduction of a waiting period before Stage 6 and also propitiated Prof. Ladd's consultancy.

Another crucial aspect of the behavior of the breakwater was the settlement in the long range. Crest level at the end of Stage 6 was placed at elevation +5,25 m and it was necessary that, in the long range, it remained above elevation +4,50 m. Expected final settlement after the end of construction were estimated as smaller than 0,75 m, therefore indicating that the crest would remain above the desired elevation. Crest settlements readings ended in 1994. No problems associated with crest elevation have been reported.

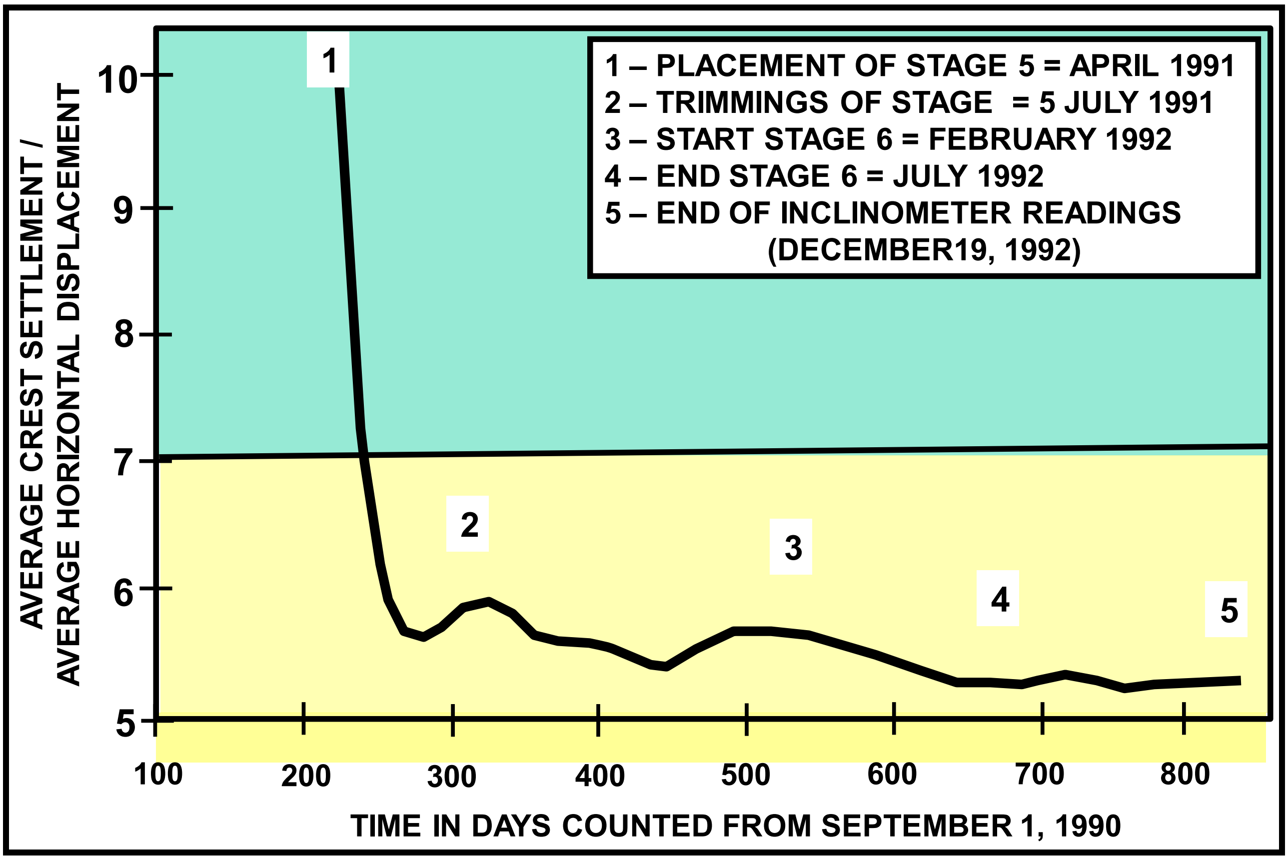

Figure 16. Method of volumes applied to station STA 4.

5 CONCLUSION

The tests and studies carried out after failure and briefly presented here have led to a reasonable understanding of the causes of failure. A new design was conceived and built, and thanks to the monitoring during all stages the work was completed successfully.

A view of the completed breakwater is shown in figure 17.

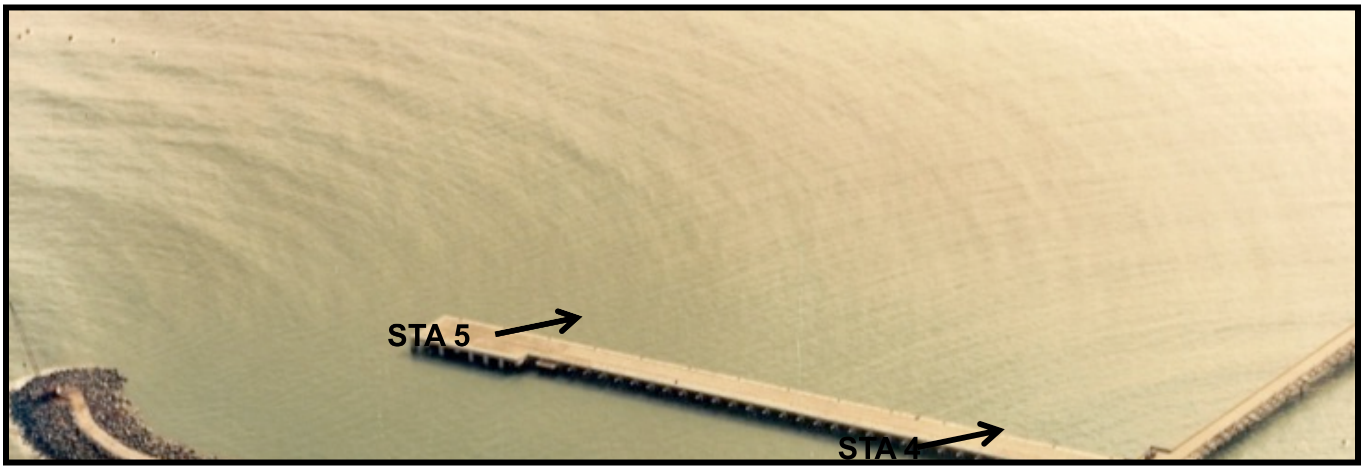

Figure 17. Inclined aerial photograph of the completed breakwater.

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