FAILURE OF A REINFORCED EMBANKMENT ON AN EXTREMELY SOFT PEAT CLAY LAYER

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Abstract: A case study of the failure of a reinforced embankment on soft clay is back analysed. The site consists of an 11 m thick very soft clay deposit. Measured (uncorrected) vane strengths vary in the range of 2 kPa on the ground surface to 20 kPa at 10m depth. The top 3m peat clay layer with water content as high as 900% showed some erratic high strength values due to the presence of fibres which misled the measured vane strength. The water content decreased rapidly with depth, displaying values of around 100% for the depth range from 3 to 6m, and around 50% for greater depths.

The reinforced embankment was about 4m high with a 12m long slope. The geogrid reinforcement was 29m long, with a nominal tensile strength of 210kN/m. The failure was transitional-planar in character at the interface between the peat and the clay layer, at approximately 2m depth. The failure occurred behind the geogrid and field evidence suggests that a significant factor was the fill buried in the clay, which instigated the failure by lateral thrust.

Keywords: clay, embankments, peat, reinforcement, stability analysis.

INTRODUCTION

Very soft clays are quite often found in the coastal regions in Southeast Brazil, particularly in the State of Rio de Janeiro (Silva, 1953; Almeida et al., 2008a). The districts of Barra da Tijuca and Recreio dos Bandeirantes, located east of the City of Rio de Janeiro, are presently the new frontiers for land use and the clay deposits found in these areas are typically extremely soft (Lacerda & Almeida, 1995). Therefore, the construction of embankments on these soft grounds has been a great challenge. The choice methods for embankment construction on these soft clays include vertical drains (Almeida et al., 2001), light material (Sandroni, 2006) and piled embankments (e.g., Sandroni & Deotti, 2008; Almeida et al., 2008b), most of them using geogrid reinforcements.

A residential building complex with 12 apartment blocks is being constructed on this extremely soft clay at Recreio dos Bandeirantes District. The site is 150m wide by 350m long, as shown in Figure 1. Curves of soft clay thickness are also shown in Figure 1. At the front of the site, which is the area of relevance for the present paper, the soft clay thickness varies from 4 to 11m. At the rear, the soft clay thickness varies from 2 to 4m. The computed embankment settlements, of approximately 4.3m in the thicker parts of the soil deposit, had to be stabilized in 30 months.

The site investigation carried out included *in situ* and laboratory tests. This campaign showed very compressible clay with very low undrained strength. Consequently, the embankment was designed to be constructed in three stages, with vertical drains to reduce the construction schedule and improve the strength between stages. It was also necessary to reinforce with geogrids at the borders of the front of the site. The embankment behaviour was monitored with settlement plates and inclinometers.

In the first construction stage, an embankment failure occurred in part of the left-side border. This paper presents and discusses the probable causes of this failure. Descriptions of the geotechnical characteristics of the area and of the construction of the embankment surrounding the buildings are presented initially.

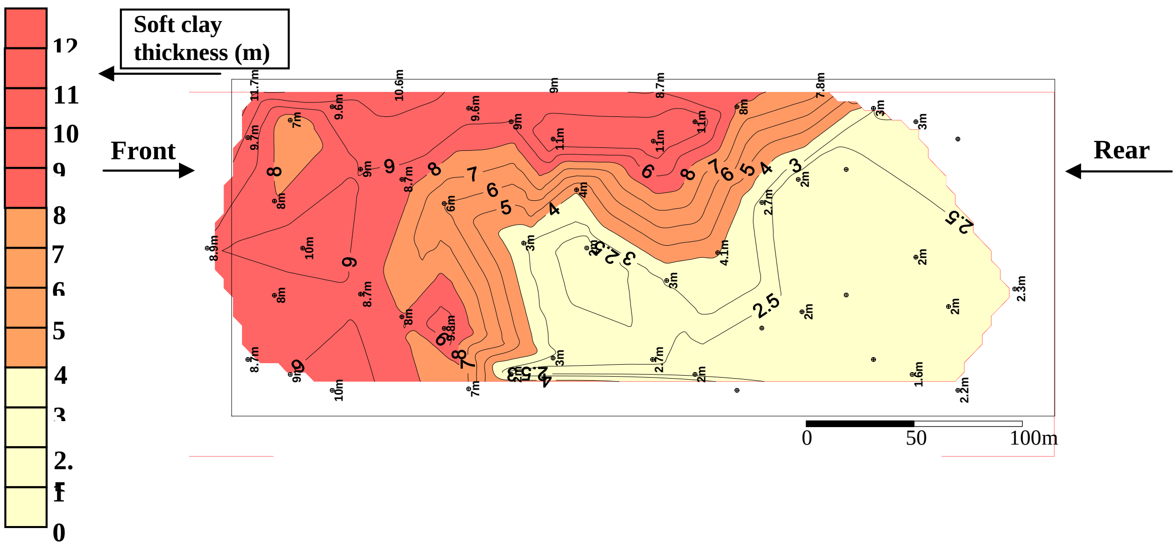

Figure 1. Curves of soft clay thickness.

SOIL CHARACTERIZATION AND CONSTRUCTION TECHNIQUE

The site investigation carried out consisted initially of SPT borings and subsequently of *in situ* and laboratory tests to obtain geotechnical parameters for settlement calculations and slope stability analyses. These tests were performed in vertical clusters and the typical characteristics of this clay deposit are shown in Figure 2 for cluster SP36. In order to obtain the variation of the peat thickness along the area, soil samples were collected with an SPT sampler for water content determination. These results are plotted in Figure 3, compared with laboratory test results carried out on undisturbed samples. The superficial peat layer is well defined in the area and its high natural water content reaches 950%. This layer's thickness is about 3m at the front of the site. At the rear of the site, the compressible layer is composed mainly of peat.

Some results from the consolidation tests are shown in Figure 2. It is observed that the high values of CR = $C_c/(1+e_0)$ of the peat layer were mostly responsible for the high settlements predicted. A temporary surcharge was designed to compensate for these secondary settlements, estimated to be as much as 0.70 m for the life span of the building complex.

Primary plus secondary settlements were predicted to be of the order of 4.3m, for the area with a thicker clay layer, which means vertical specific deformations of about 40%. Due to this high value of settlement, it was necessary to use a very thick fill so that the embankment could reach its final stabilized elevation of about +2.6m. Considering the stress variation due to fill construction, the vertical and horizontal consolidation coefficients at those levels of stresses were $c_v = 1.67 \times 10^{-8}$ m^2/s and $c_h = 2.5 \times 10^{-8}$ m^2/s, from the consolidation tests and piezocone dissipation tests, respectively.

The construction technique to stabilize the settlements at the front of the site, for the clay thickness greater than 4m, was vertical drains with surcharge, in a triangular grid distribution with 1.4m spacing of the drains and three-staged embankment construction. After settlement stabilization, a surcharge of 1.5 m of fill was scheduled to be removed.

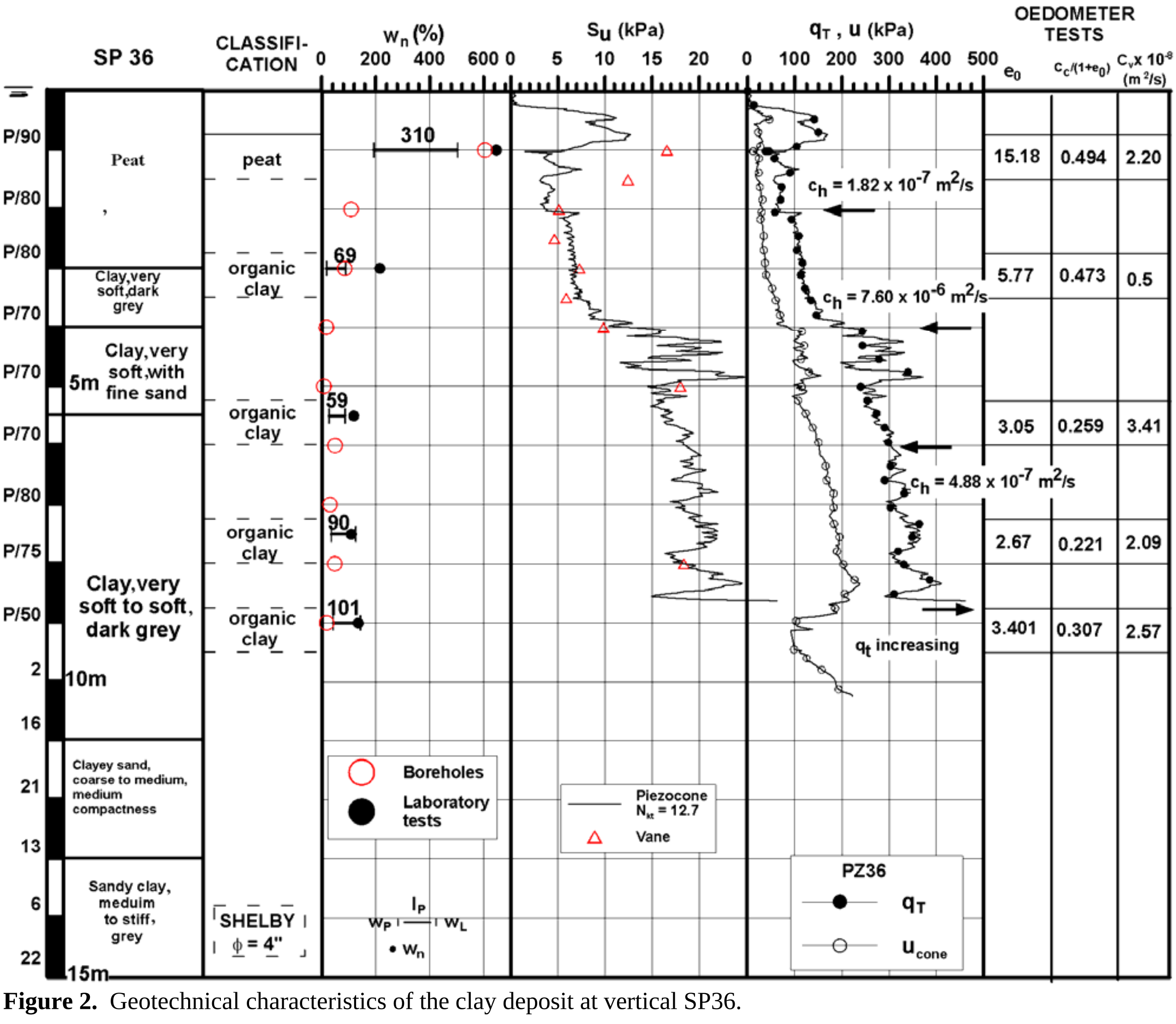

Figure 2. Geotechnical characteristics of the clay deposit at vertical SP36.

Initially, a 1 m thick working platform of sand material was executed in order to access the flooded area, also providing a drainage blanket for the vertical drains, as shown in Figure 4. Due to the high thickness of peat and low undrained strength of this deposit, it was necessary to use light geotextile reinforcement at the base of the working platform (not shown in Figure 4). A geogrid reinforcement layer and a gentle slope 1(V):3(H) was used at the borders of the front of the site (see Figure 4), with the purpose of ensuring a factor of safety greater than 1.30 during construction using the translational block type of analyses. Two alternatives with longer berms and smaller geogrid strengths were also considered. However, the owner had boundary limitations and decided to go for the shortest berm and highest geogrid strength. The geogrid was installed above the 1 m thick working platform. Ideally, reinforcement has better efficiency when installed at the lowest possible elevation but, in the present case, the geogrid would be damaged by the mandril used for the wick drains installation, thus it was placed above the working platform after the drains were driven.

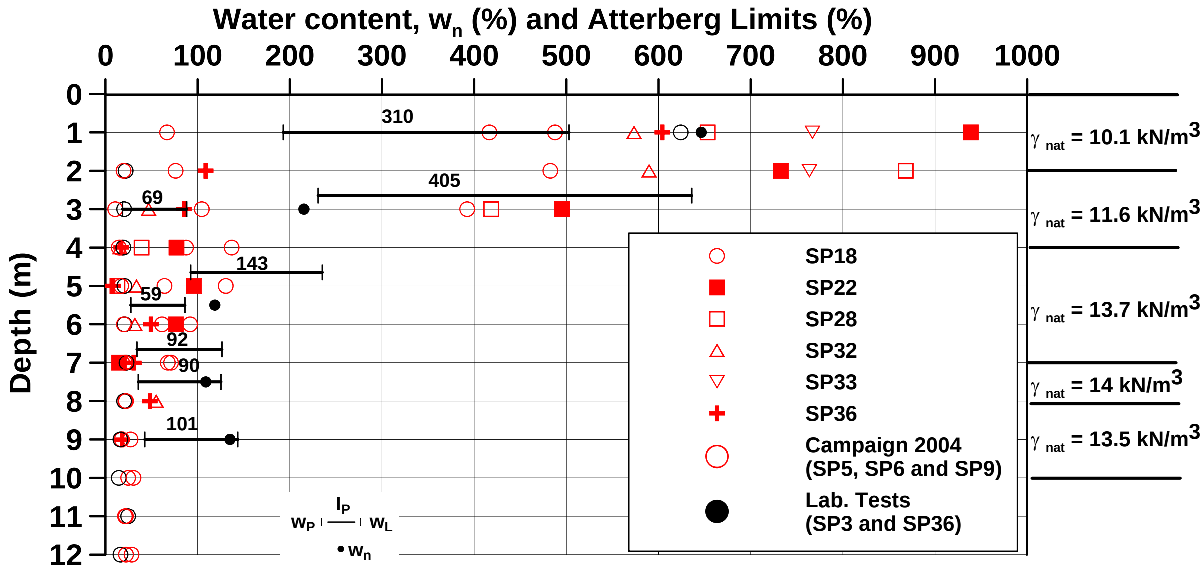

Figure 3. Natural water content profile (w_n), liquid limit (w_L), plasticity limit (w_P) and plastic index (I_P).

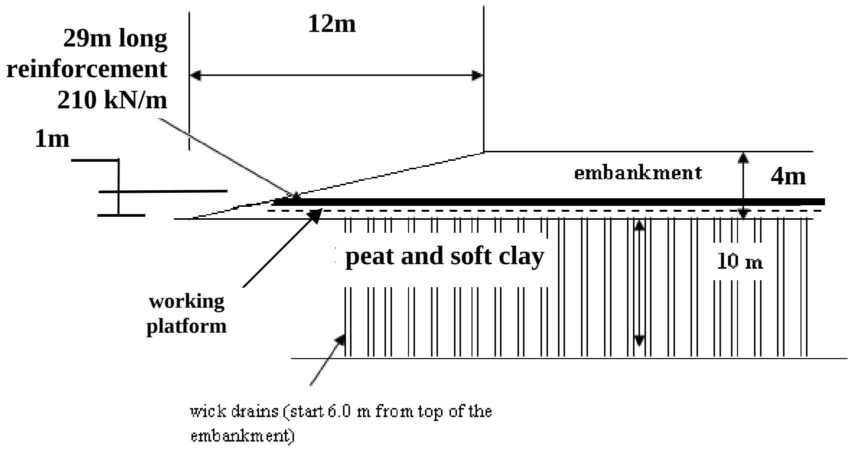

Figure 4. Border of the embankment: reinforcement and slope.

The embankment behaviour was monitored from 2006 to 2008, with settlement plates, piezometers (in the clay layer and in the drainage blanket) and inclinometers. The piezometer readings inside the drainage layer allowed the analysis of the efficiency of the drainage blanket. The inclinometers installed after rupture were used to monitor the stability of the extended lateral berms, executed after the embankment failure.

UNDRAINED STRENGTH

Figure 5 shows the corrected undrained strength (S_u) profile, obtained from vane and piezocone tests, which are compared with vane tests data from a nearby site (Navarro, 2004) about 800 m away. The Bjerrum correction factor used for the vane tests was $\mu = 0.65$, due to the high plasticity index of this clay. Piezocone undrained strength profiles used the cone factor $N_{kt} = (q_t - \sigma_{vo})/S_u$, = 12.7, obtained by vane–piezocone correlations at the site, where q_t = corrected tip resistance from piezocone test and S_u is the uncorrected strength from the vane tests. Therefore, the piezocone S_u profile shown in Figure 5 is also corrected with $\mu = 0.65$, a commonly used correction factor for Brazilian coastal clays (Sandroni, 2006).

Due to the high water content of the peat layer, a low S_u value was expected. However, the S_u values obtained from the vane tests carried out on this layer were very high due to the presence of organic matter (wood sticks, roots, fibres etc.). The S_u values from the piezocone tests were not high in the peat layer, maybe due to the form of tip and the rupture surface. At subjacent layers, S_u values are similar to those obtained by Navarro (2004). The typical value of sensitivity defined as the ratio of intact and remoulded vane strengths was S_t = 7.0 (i.e., very sensitive clays), with no clear difference between values for the peat and for the clay layer.

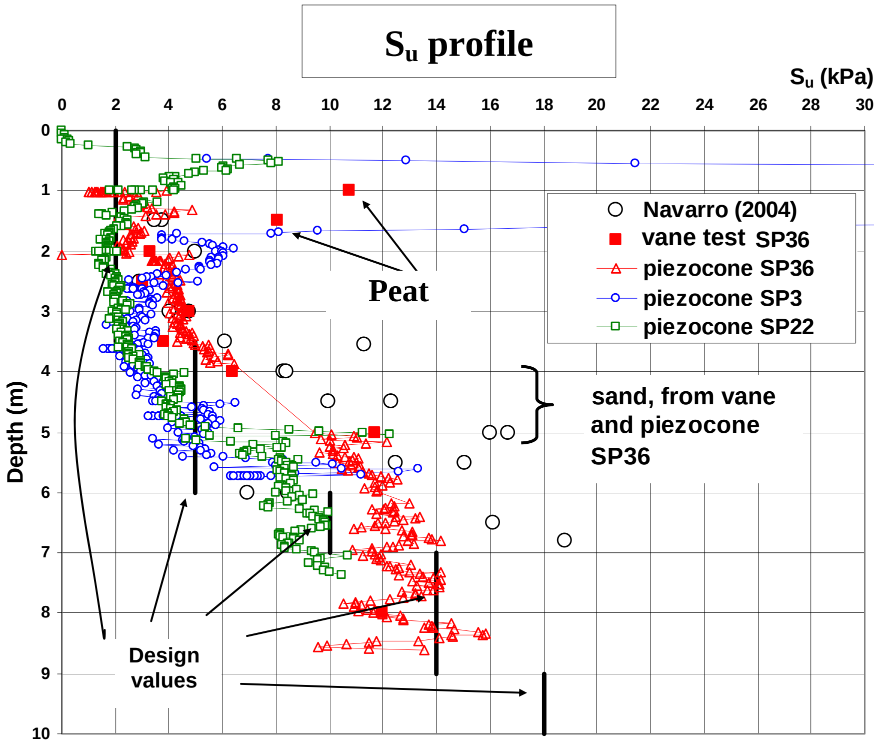

Figure 5. S_u corrected profile from vane and piezocone tests.

ANALYSIS OF THE FAILURE

Failure and remedial measures

In July 2006, about three months after construction started, a failure occurred. The location of the failure was parallel to the left border, as shown by the fissures indicated in Figure 6. The fissure shown in Figure 6a was located at the end of the geogrid. Elevation of peat in the neighbouring area was noted and the border fence was displaced by approximately 2m.

Figure 6 also shows the haul road used by the trucks to transport fill material in the site, quite close to the fill fissure. Investigation borings performed in the embankment fill later on indicated that the thickness of the fill along the haul road could be as much as 8.0m, supporting the hypothesis that this may have triggered the failure in this area. The estimated fill thickness below the haul road at the time of the failure was 6.0m. In March 2006, at the beginning of the construction, as soon as the working platform was being placed, a localized failure occurred and the peat rose laterally. This occurred at the entrance close to the haul road (see Figure 6), thus contributing to the disturbance of the clay in this region.

In order to remediate the embankment failure, a number of measures were implemented. These included filling the fissure with fill and the installation of two layers of 20m wide 25kN/m strength geotextile patches through the failed area. The slope was also repaired and a berm was constructed to increase the factor of safety. To monitor the local behaviour, additional instruments were installed: in particular, three inclinometers at the top of the slopes, two at the left border, where the failure took place, and one at the right border.

Figure 7 shows the results of IN1 and IN3, both installed one month after the failure at the left border. The analysis of these inclinometers showed that the movements were still occurring along the mobilized failed surface, which is located about 2m below the fill, at the peat–soft clay interface.

Geometry of the embankment and failure

Figure 8 shows the embankment geometry at the time of the failure. At its highest point, the embankment was 4.8m thick and was 2.7m below the water table. At that time, the settlement plates showed settlements of about 1.9m. In the failed region, the soft soil was originally 3.0 m peat layer followed by 5.0 m of very soft organic clay layer, i.e., typically 8 m overall thickness. Considering the CR values presented in Figure 2, the estimated layer thicknesses at the time of the failure were approximately 2m for the peat layer and approximately 4.1m for the soft clay layer.

Field evidence showed that the fissure observed in the embankment was quite close to the end of the geogrid, as shown in Figure 5. Therefore, the back analysis carried out herein assumes that the failure surface developed outside the geogrid. Field evidence showed a translational movement of the embankment, and inclinometer readings (Figure 7) indicated the lowest failure surface 2.0 m below the base of the embankment, i.e., involving essentially just the peat layer. Therefore, a planar-block type of stability analysis was assumed, similarly to the design phase.

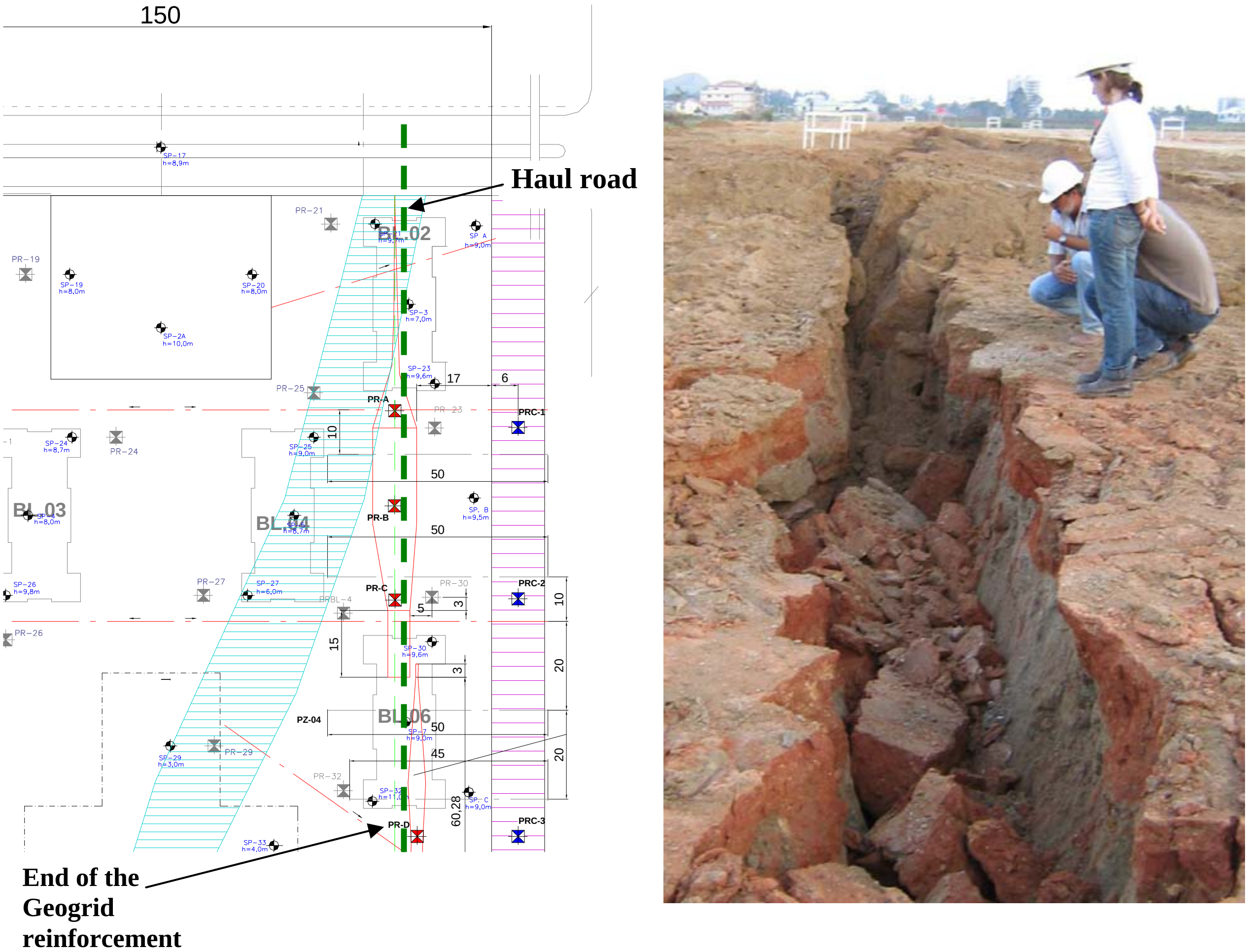

Figure 6. Embankment failure: (a) location showing the haul road (b) fissure developed at the embankment.

Soil parameters and possible failure scenarios

Based on laboratory tests and local experience, the parameters adopted for the embankment fill were bulk weight 19 kN/m³ and strength parameters $\phi' = 28^o$ and $c' = 0$ kPa. The bulk weights for the peat layer and clay layer were measured as 11 kN/m³ and 14 kN/m³, respectively.

The S_{uo} = 2.0 kPa value adopted at the design stage was used for the intact peat layer in front of the embankment. The purpose of the present analysis was to obtain mobilized S_u values during failure.

The gain in the clay strength ΔS_u that was expected to occur during the three months of consolidation was estimated to be ΔS_u = 3.0 kPa. Therefore, back-analysed values should be $S_u = S_{uo} + \Delta S_u$ = 5.0 kPa.

Two scenarios were considered for the back analyses carried out herein: (1) a lower limit S_u value given by the geometry shown in Figure 8; (2) an upper limit S_u value resulting from the active thrust behind the line of the geogrid reinforcement, due to the trucks' traffic on the buried fill.

Back analysis and discussion

The back-analysed values for the scenarios (1) and (2) described above are 3.5 and 8.0 kPa, respectively. These values are plotted in Figure 9 using normalized depth in order to take into account the decrease of the clay thickness due to settlements. Figure 9 also shows the values used for the design (first-stage construction) and measured post-failure values (2nd campaign), with Bjerrum correction applied. The values measured in the left border are in the range of the back-analysed values and support the estimated S_u = 5.0 kPa with gain in strength. Values of S_u at the right border are much greater than S_u values at the left border and showed a greater gain in strength, which is in accordance with the case history (Almeida et al., 2001) of Barra da Tijuca soft ground deposit, the top peat layer of which showed much greater gain in strength than that given by ΔS_u = 3 kPa.

Therefore, the overall data shown in Figure 9 support the hypothesis that the failure was caused by the active thrust due to the embankment buried inside the peat layer.

Figure 10 shows vane correction factors including the present work of 2-D failure (using the design value μ = 0.65), and a number of Brazilian case histories. The relevant plasticity index to be used here is I_p = 106%, the average value at the depth of the slip surface. It is observed that μ = 0.65, following the Brazilian practice (Sandroni, 2006), appears to be quite acceptable.

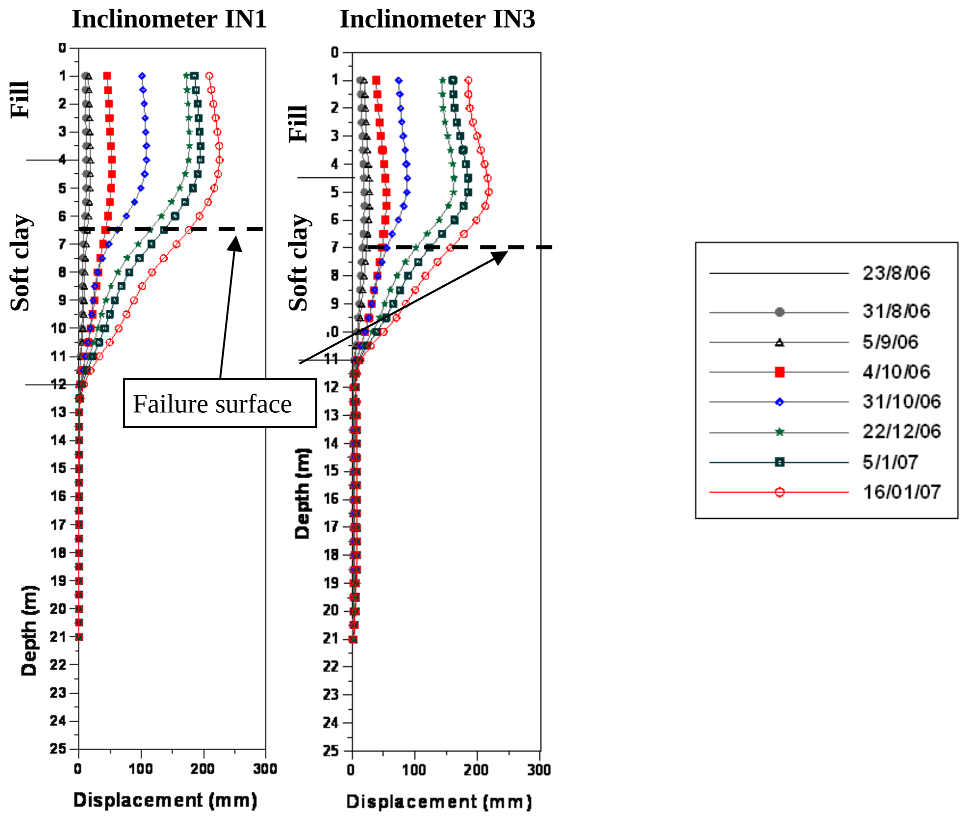

Figure 7. Inclinometer readings shortly after failure.

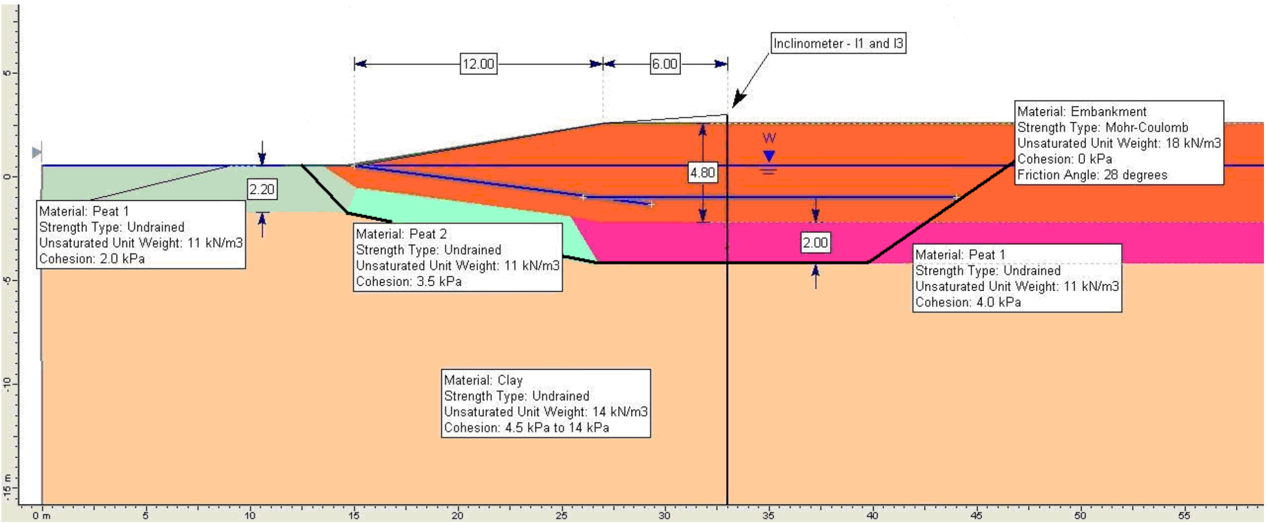

Figure 8. Geometry of the slope.

PERFORMANCE OF THE EMBANKMENT FOLLOWING FAILURE

After the failure was noted in part of the left border, there was some concern about the performance of the vertical drains of the embankment at the left side, since there were two buildings to be constructed in this area. In order to improve the drainage at the front of the site, pumping wells were installed and the pore pressure was monitored. The measurements taken showed that the settlements were well within the expected range, as shown in Figure 11. Settlement plate PR24 in Figure 11 was installed at the centre of the area, where the fill thickness was 3m at the time of rupture.

Regarding the lateral movements at the left border, Figure 12 shows the maximum horizontal displacements for the remainder of the three-staged construction. It is clearly seen that the horizontal displacements showed a stabilized trend, even with the two additional stages of fill construction.

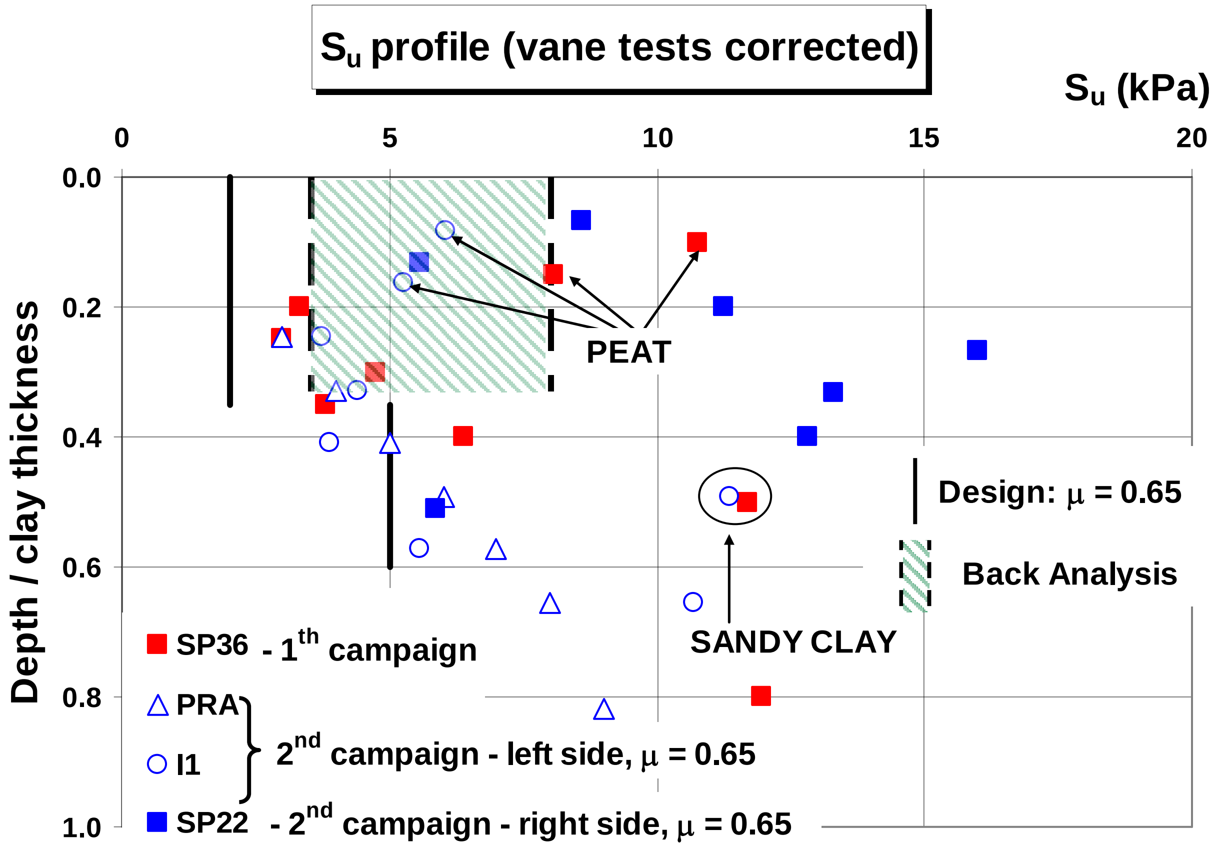

Figure 9. S_u profiles for normalized depth: design values; back-analysed values; measured values just after failure.

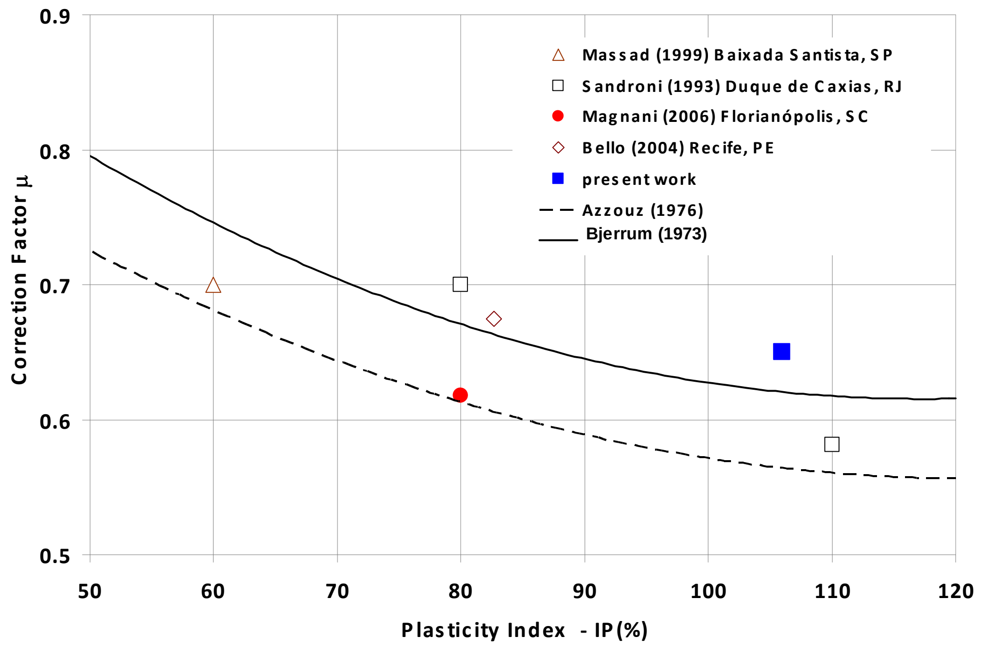

Figure 10. Vane correction versus plasticity index: present work and Brazilian case histories.

CONCLUSIONS

The failure described here took place three months following the start of construction in an extremely soft clay deposit with the undrained strength as low as 2 kPa in the peat layer close to the surface. Field evidence suggests that a translational type of failure occurred with the failure surface approximately 2m below the base of the embankment at the limit between the peat and the clay layers. The failure was located behind the geogrid reinforcement as the haul road located at this point forced the fill to penetrate the peat layer. The back-analysed undrained strength values are in the range of the corrected post-failure measured vane strengths. This suggests that the hypothesis that active thrust caused by the buried embankment did contribute to the failure is correct.

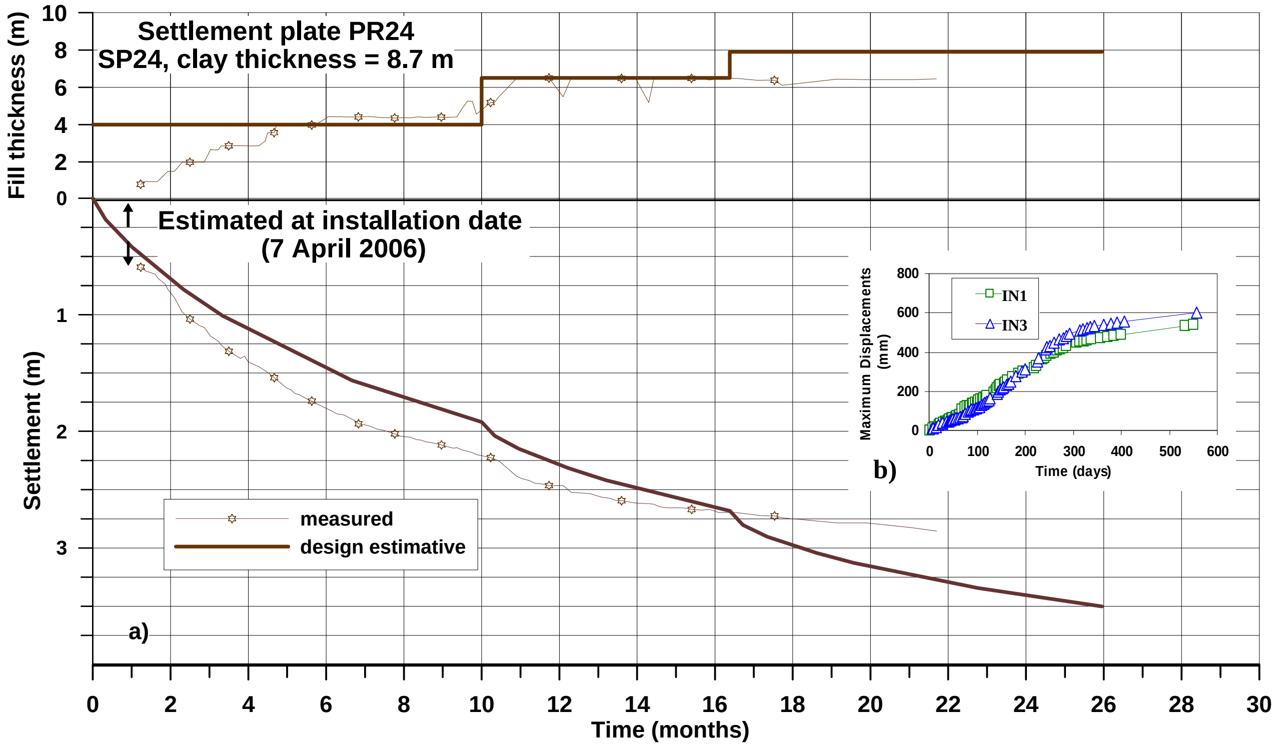

Figure 11. Long-term displacements: (a) typical settlement plate data; (b) long-term maximum horizontal displacements for inclinometers at the left border.

ACKNOWLEDGEMENTS

The authors would like to thank Sergio Guedes, the Director of Craft Engenharia Ltda, in charge of the earth fill construction, for his continuous interest and collaboration throughout the whole duration of the project. Thanks are also due to Dr Maurício Ehrlich, COPPE-UFRJ, for his comments on the paper.

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