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ABSTRACT

This paper presents the study of the rupture of an embankment on soft clay, localized in Recife, Brazil. The data was obtained from partnership of the Department of Civil Engineering of the Federal University of Pernambuco, Brazil and the company Gusmão Engineers Associates, which it was possible to perform a Master Thesis.

The embankment was constructed without geotechnical investigation project, accompaniment and technological control. After the rupture, to understand the process, a geotechnical investigation was performed to permit a total stress stability analyze / back-analyze, considering circular and non-circular surfaces, occurrence of cracking in embankment, and the three-dimensional effect. The Data Base of Recife soft clays (Coutinho et al. 1998c) were used to complement the technical information necessary to the study. In the evaluation of the undrained shear strength of the foundation (Su) was used the results of field vane test, with consideration of the corrections proposals for Bjerrum (1973).

All the results of this work showed that rupture was previsible if adequate geotechnical investigation and stability analysis had been done.

INTRODUCTION

The development of sites underlain by thick deposits of soft soil has turned to be more and more common in the cities placed at the Brazilian Lowlands. That occupation happens in many different applications such as foundations of buildings, embankments of highways, airports, dams, urbanization areas, etc).

The construction of embankment on soft clay is one important geotechnical problem and it has been studied for various authors, accumulating experiences for a better understanding of soft soils under solicitation of load increases (e.g. Bjerrum 1973; Leroueil & Rowe 2001).

In general, the project of embankments on soft soils should attend the basic requirements of stability against rupture and the settlement for consolidation, during and after the construction, compatible with its objective.

This work presents a study about a rupture of an embankment on soft clays occurred in an area besides of the federal road BR-101-PE, Recife, Brazil (Bello 2004; Coutinho & Bello 2007; Bello et al. 2006).

The geotechnical profile in this local has a thick layer of soft soil of about twelve meters of thickness. The embankment was constructed without geotechnical investigation project, accompaniment and technological control. After the rupture, to understand the process, a geotechnical investigation (in situ and laboratory tests) was performed to permit the total stress stability analysis / back-analysis. It was considered the circular and non-circular surfaces, the occurrence of cracking in embankment, and the three-dimensional effect.. The Data Base of Recife Soft Clays (Coutinho et al. 1998c) was used to complement the necessary technical information for the study.

CASE STUDY

Localization / Characteristics

Recife City is situated on the Northeastern coast of Brazil (Fig. 1) and presents a wide coastal plain and lowland area. Soft clay and organic soil deposits can be found in about fifty percent of the area. The importance of human action in the actual conformation of the deposit can be observed in successive embankments performed with time, preparing the land for utilization in supporting the foundations of buildings and other engineering workmanship, at least since the period of the Netherlands colonization (1630 - 1654). It is estimated that a total area of 25 million square meters of embankment was constructed.

The study area has approximately 10.000m^2 and it is located in the BR-101, Recife-PE, where were constructed 3 sheds. The rupture occurred in the greater shed. Figure 2 shows the position of the sheds and location of the geotechnical field investigations.

Initially, the area had been regularized with an embankment of about 1.0 m of thickness, and executed a gabion wall with height between 3.0 and 6.0m. During the construction, occurred a movement with the soft clay and peat expulsion for the neighboring area. It was observed cracks in the ground parallel to the support wall and in the structure of the shed. In function of these facts, it was requested an evaluation of these problems and recommendations of corrective measures. Considering peculiar problems involved, it was also recommended accompaniment / technological control and realization a geotechnical instrumentation to follow up vertical and horizontal displacements. This accompaniment was not realized in the local (decision of the owner) and after six months, the general rupture occurred (Fig 3).

Geotechnical investigation

In this study, it was performed two SPT soundings with water content measurement (W_n), two vane test soundings, undisturbed sampling shelby (only one sample) and correspondent laboratory tests - characterization, oedometer and triaxial UU-C compression (Fig. 1).

Figure 4 presented a typical geotechnical and natural water content (W) profile. Through standard penetration tests SPT was identified a thick layer of soft organic soil (SPT<1) until 12 m of thickness and a water level approximately 4 m of depth. The natural water content presented a maximum value (334%) in the depth of 7.0m. After this depth it is observed a decrease in this valor until 12.0m of depth (34%), and then becoming constant with depth.

It was collected one undisturbed sampling shelby of 4" of diameter in the 11.40 depth. The characterization tests classified the soil as composed of 27% de silte, 67% of clay and 6% of fine sand, presenting the following parameters: specific weights γ_{clay}= 11.9kN/m^3, natural water content (W) 223%, liquidity limit (W_L) 76%, plasticity limit (W_P) 57%, plasticity index (I_P) 19%, contraction limit (LC) 37%, organic content (TMO) 67%, pH = 6.66% and conductivity (μs/cm = 1640).

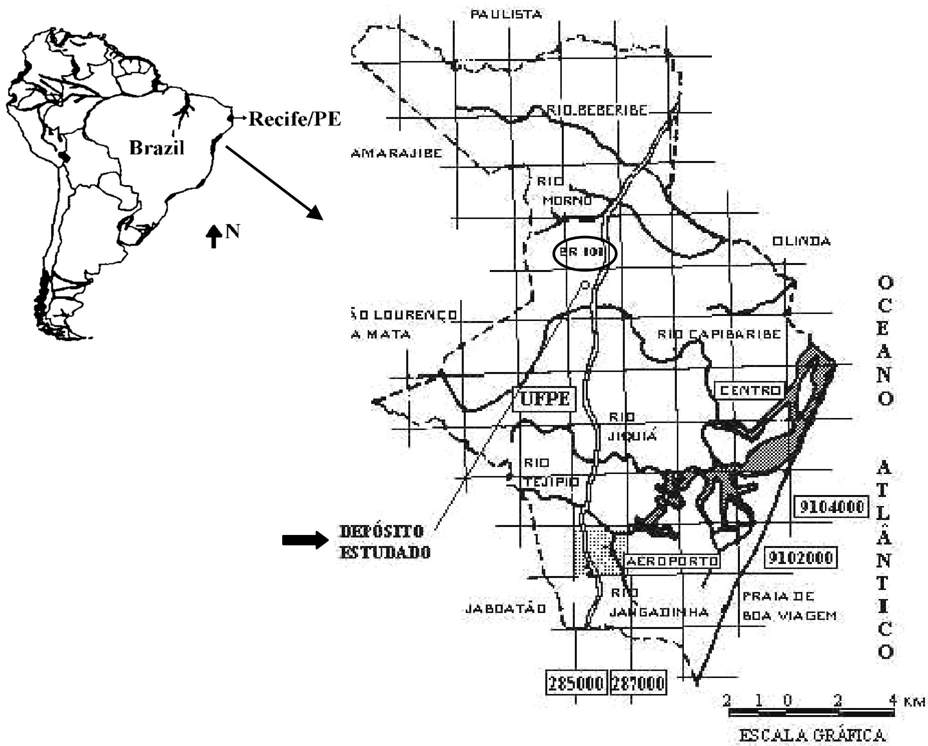


Figure 1. Localization of the site student (Bello 2004).

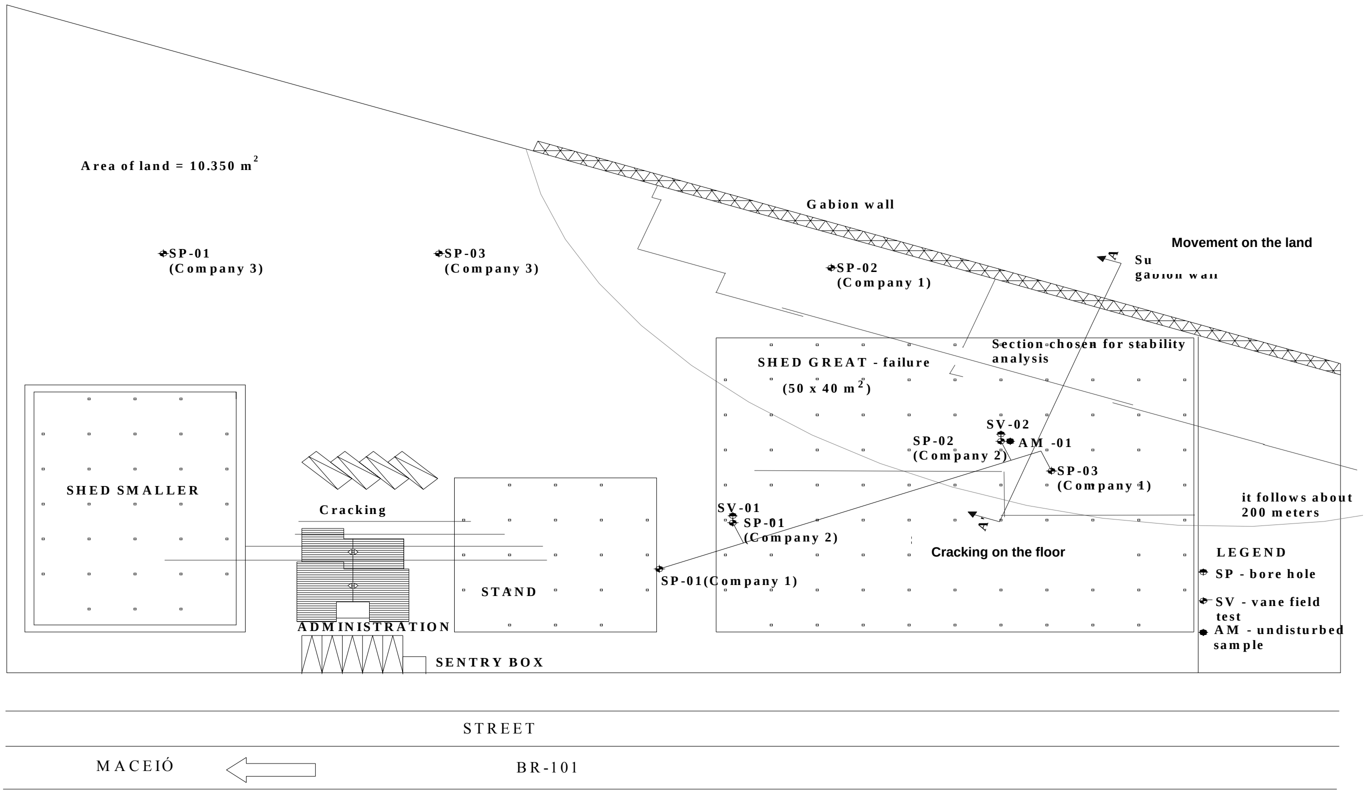


Figure 2. Situation, localization of SPT soundings, vane field tests and undisturbed sampling (Bello 2004).

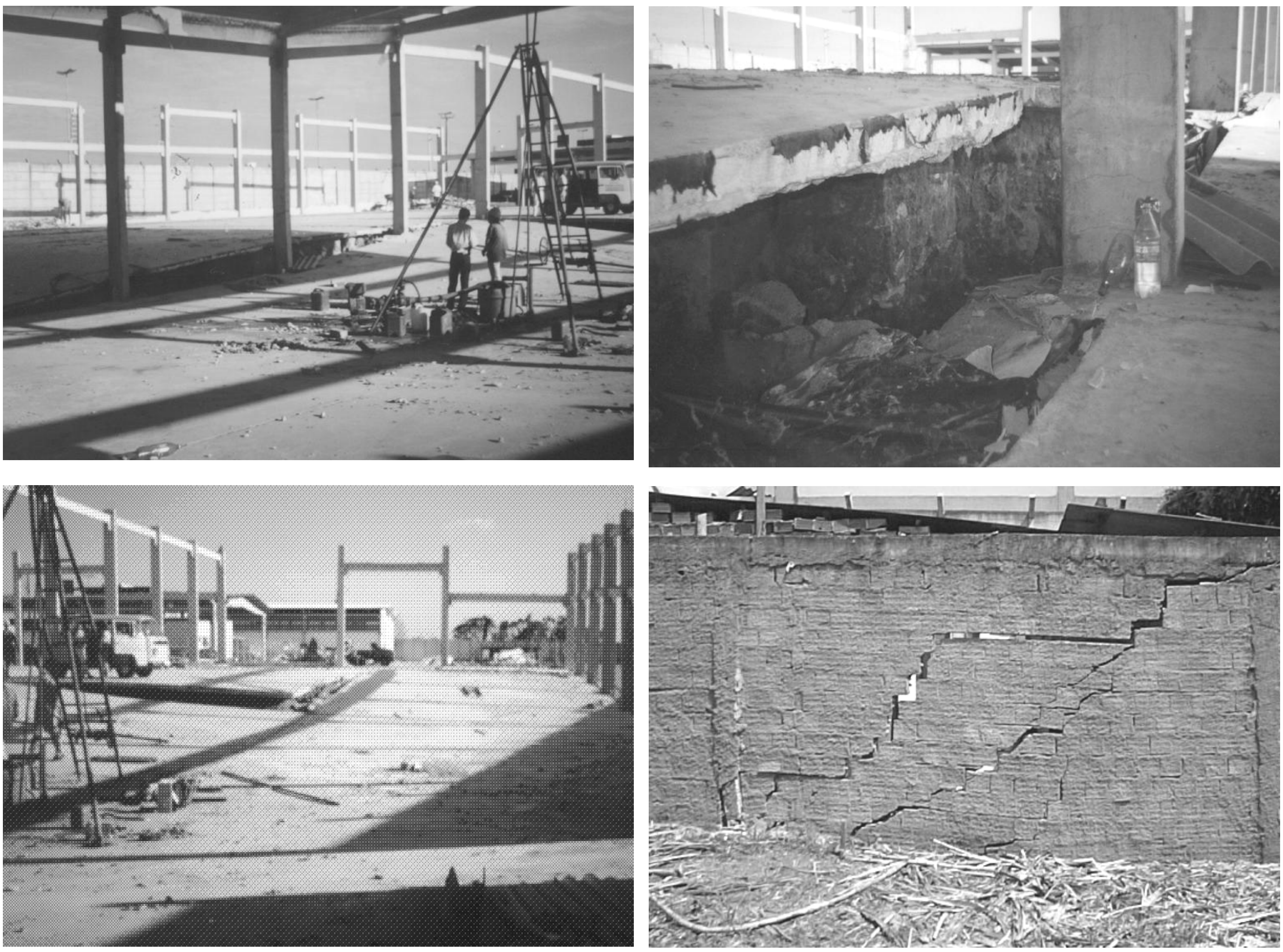


Figure 3. General visualization of rupture occurred – BR-101 (Bello 2004).

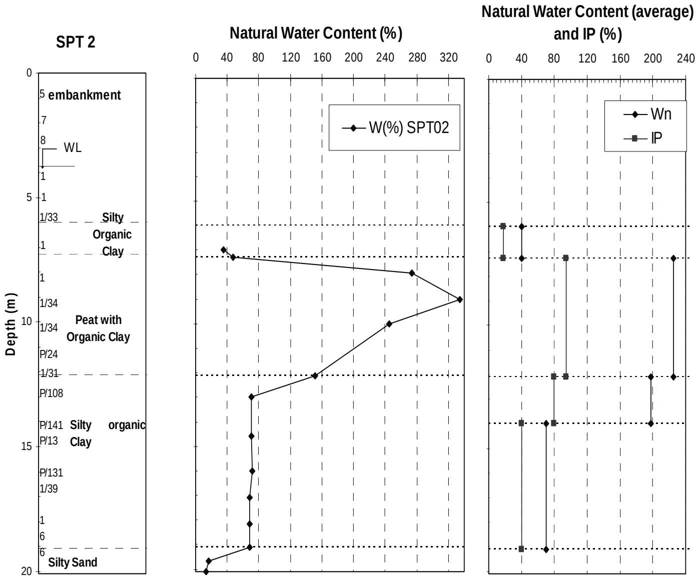


Figure 4. Natural Water Content (Bello 2004).

The water content value was very higher to the one of the liquidity limit (LL), probably due to previous drying of the organic clay samples before the realized of the tests for determined LL and IP.

In the vertical oedometer tests was obtained the parameters: compression index (C_C=1.9), swell index (C_S=0.3), initial void ratio (e_0=4.564) and preconsolidation pressure (σ'_p=42.5kPa).

In accordance considered for Lunne et al. (1997), (see Coutinho et al 1998a), the quality of the sample was poor to very poor ($\varepsilon_{\sigma'vo}$= 13%). The disturbed effected in the σ'_p, indicated a false subconsolidation condition of the clay (OCR=0.71).

It was performed triaxial UU and CIU tests to define the strength parameters (Su=20kPa, c'=12kPa e ϕ'=30º).

The undrained shear strength obtained from field vane test varies between 17 and 50 kPa (Fig. 5a). The lowest value (~17 kPa) corresponded to the range of 12 – 14m depth and the highest Su value (~50 kPa) corresponded to the range of 7.2-10m of depth. Due to the presence of roots and materials in decomposition, a correction was used in the Su value for the depth of 7.0 m (from 70 to 50 kPa). The Sensitivity results are also presented in the Figure 5, showing values in the range of 3 to 15, increasing with depth. The soil can be classified as medium to high sensitivity, according to the classification of Skempton & Noythey (1952).

ESTIMATED OF LIQUIDITY LIMIT AND PLASTICITY INDEX

It has been observed in Recife softy clays that the water content value is very next to the liquidity limit value (Coutinho et al. 1998c).
The Chart of Plasticity with the results of tests of laboratory for Recife soft clays and Juturnaíba organic deposits (Rio de Janeiro) (Coutinho et al. 1998b) was used for estimated the IP values, considering LL=W_n values and classification of the ground that constitutes each layer (Fig. 6). With the LL values considered LL=W_n and the range of results for each soil type an average IP values were estimated (Table 1).

In this study, the determination of the natural water content in each meter during the SPT tests and the Data Base of Recife Soft Clays (Coutinho et al. 1998c) were important for knowledge of the estratigrafy and complementation of necessary technical information.

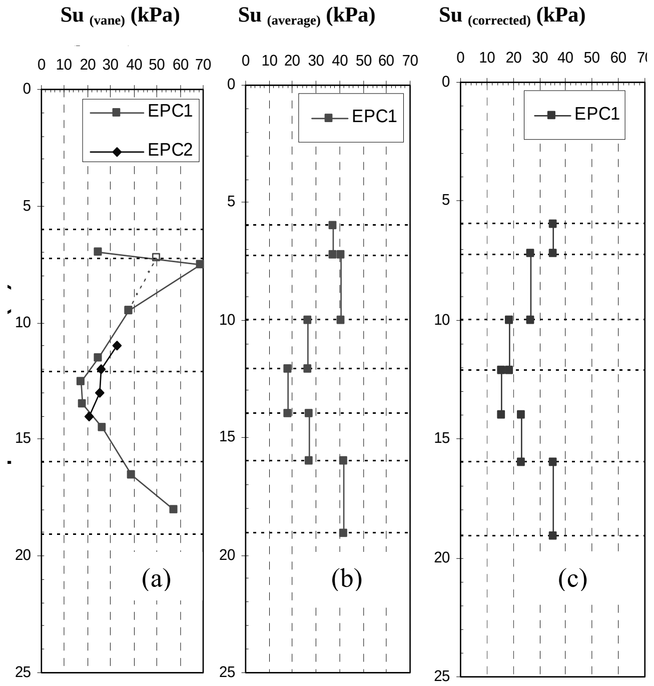


Figure 5. Undrained Strength and corrected by Bjerrum proposal (1973) (Bello 2004; Coutinho & Bello 2005).

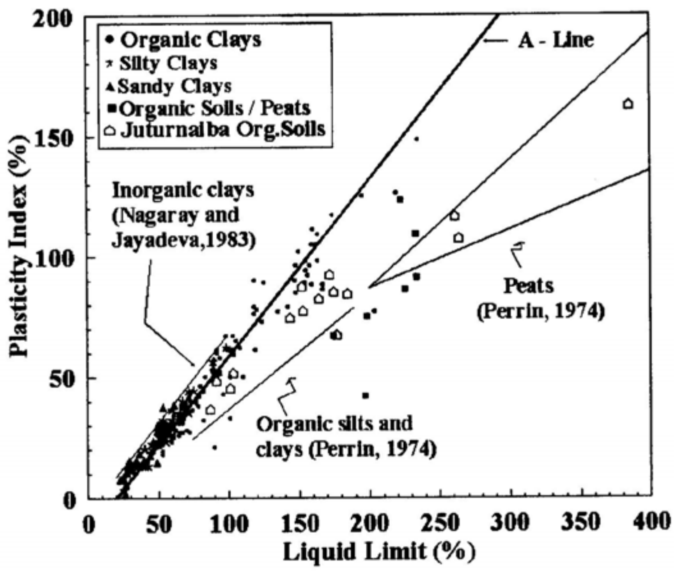


Figure 6. Plasticity chart – Recife and Juturnaíba soft soil results (from Coutinho et al. 1998b)

Table 1. Values estimated of liquidity limit and index plasticity (Bello 2004).

Depth (m)	Description of layer	Wn=LL (%)	IP (%)
6 – 7.2	Silt Organic Clay	40	18
7.2-10	Peat with Organic Clay	225	95
10-12.1	Peat with Organic Clay	198	80
12.1-14	Silt Organic Clay	70	40
14-16	Silt Organic Clay	70	40
16-19.1	Silt Organic Clay	70	40

STABILITY ANALYSIS AND BACK–ANALYSIS

For performing the stability analysis and back-analysis was considered the soil profile presented in the Figure 7. The program GEO SLOPE allows established sub layers with different type of soils and shear strength values. This way, the Su averages values obtained from field vane tests (Fig. 5a) were used for defined the sub layers (Fig. 5b) and after corrected second proposal of Bjerrum (1973) (Fig. 5c).

It is important to register that it is the first investigation in the Northeast Region of Brazil clays about the applicability of the Bjerrum Su correction. In fact, very few studies have been performed in Brazil and South America, where there are different types of soft deposits of those considered by Bjerrum work. This type of investigation is very important for local and can contribute for regional and international research and practical works.

In the analysis was considered embankment without cracking, 50% and total cracking. This study had an objective of verify the influence in the FS of occurrence of the cracking embankment during the rupture process.

The stability analysis was done considering circular surface. After this initial analysis was performed a back-analysis considering the field observations in the moment of rupture.

It was possible to determine some probable points (initial and final) of the rupture surface and probably, the rupture surface tangencies the minor strength layer (depth 12 to 14m) (Fig. 6).

The localization of the landslide surface was not complete, because not carried through studied topographical and internal instrumentation, but it is considered reasonable adequate for an investigation. In the back-analysis was admitted circular and no circular surface.

The calculation of the FS_{min} was done using the Modify Bishop, Janbu, Spencer and Morgenstern-Price methods. It was established 7 hypotheses for the calculation of the FS_{min} value, considering cracking of embankment and Su value correction (Tab. 2).

Table 2. Hypothesis about considerations of strength of embankment and foundation (Bello 2004)

HYPOTHESE	EMBANKMENT	FOUNDATION
1ª	without cracking	Su corrected
2 ª	without cracking	Su without corrected
3 ª	50% cracking	Su corrected
4 ª	50% cracking	Su without corrected
5 ª	100% cracking	Su corrected
6 ª	100% cracking	Su without corrected
7 ª	50% cracking	Su corrected (average EPC1 e EPC2)

The stability analysis was done considering de AAs et al. (1986) corrected. This study this objective of verify the use of this corrected in Recife soft clays end compared the results with the Bjerrum proposal results (see Tab. 3).

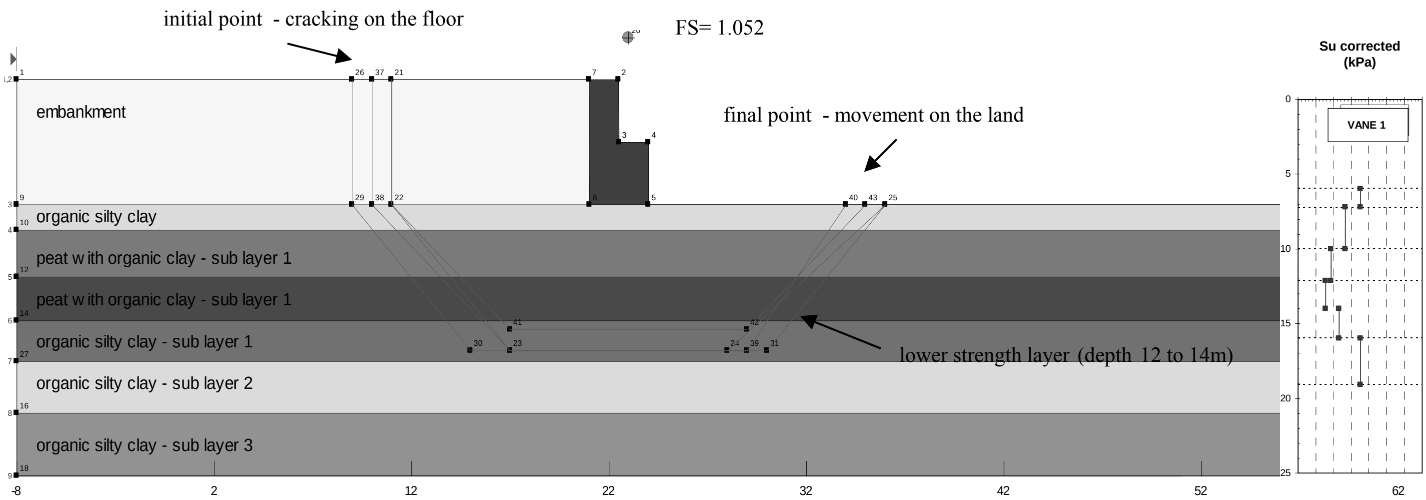


Figure 7. Determination of no circular surface - back-analyze: 100% cracking of embankment - Su correction (Bello 2004).

Table 3. Su from Bjerrum proposal and Su Aas et al. proposal.

Depth (m)	IP (%)	OCR	Su (kPa)	μ BJERRUM (1973)	Su_{corrig} (kPa) BJERRUM (1973)	μ AAS et al. (1986)	Su_{corrig}(kPa) AAS et al. (1986)
6 – 7,2	18	3,90	37,36	0,95	**35,49**	0,38	**14,19**
7,2-10	95	1,20	40,91	0,65	**26,59**	0,40	**16,36**
10-12,1	80	0,82	26,11	0,70	**18,63**	0,70	**18,28**
12,1-14	40	0,92	18,32	0,85	**15,58**	0,80	**14,65**
14-16	40	1,30	27,18	0,85	**23,01**	0,72	**19,57**
16-19,1	40	1,89	41,52	0,85	**35,29**	0,45	**18,68**

RESULTS AND DISCUSSION

The summary of FS_{min} results obtained from stability analysis and back-analysis are summarized in Figure 8 and Table 4.

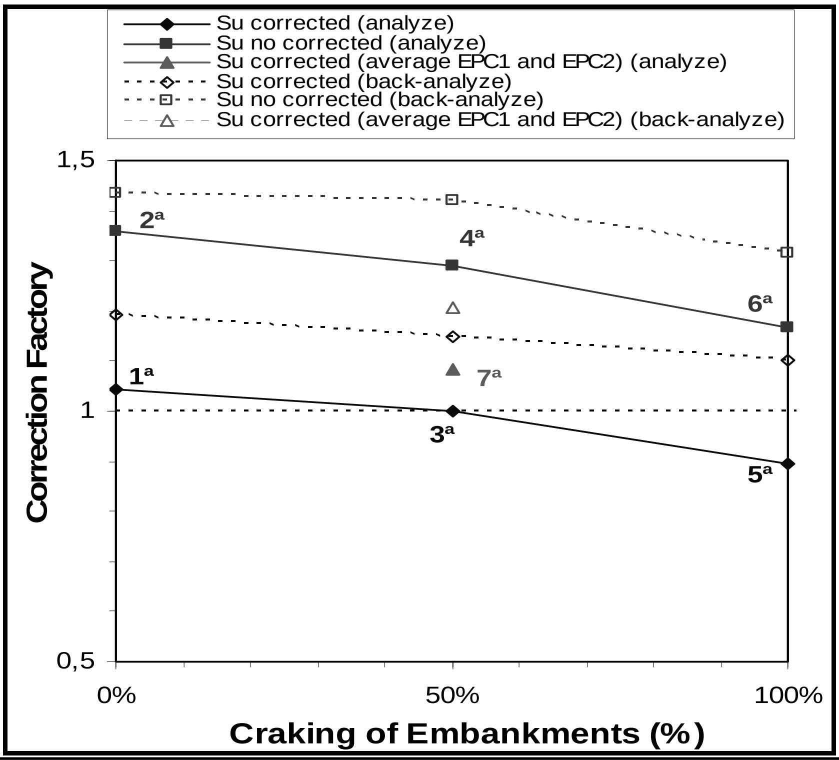
In the stability analysis, the results of FS_{min} values are in the range of 0.896 to 1.356 for the circular surface condition.

Hypothesis 2 (embankment without cracking / Su without correction) presented higher value of FS_{min},. This situation is not indicated for use in design. Hypothesis 5 (embankment 100% cracking / $Su_{correct}$) present the lesser value of FS.

Hypothesis 3 (50% cracking in the embankment and $Su_{corrected}$) presented FS=1. This condition can be able to explain the rupture.

The back-analysis FS_{min} results considering only no circular surface were satisfactory and indicated range of values close to the preliminary stability analysis, presenting slightly higher values (around 15 %). This difference can be due to the difficulty in defining the real failure surface in the field.

The critical circle of preview landslide was similar to the observed one. The maximum depth of the landslide was adequately previewed by the analysis method, correspondent to the end of the layer with Su minimal (Fig. 9). The hypothesis 5 (embankment 100% cracking / $Su_{corrected}$) presented FS=1.05. This condition also can be able to explain the rupture.



1ª, 2ª, 3ª, 4ª, 7ª - Hypothesis about strength of embankment and correction of Su.

Figure 8. Comparison of results of stability analysis / back-analysis (circular surface) – Bishop Method (Bello 2004)

Table 4. Summary of the FS_{min} calculated in the GEO SLOPE Program (Bello 2004; Coutinho & Bello 2005).

Hypothesis	Cracking of embank. (%)			Su Correction		CIRCULAR SURFACE				NO CIRCULAR SURFACE		
						ANALYSIS		BACK-ANALYSIS		BACK-ANALYSIS		
	0	*50*	*100*	*yes*	*no*	*BISHOP*	*SPENCER*	*BISHOP*	*SPENCER*	*JANBU (corrected)*	*SPENCER*	*MORGENSTERN -PRICE*
1	X			X		1.045	1.048	1.192	1.190	1.240	1.232	1.232
2	X				X	1.356	1.357	1.475	1.472	1.480	1.397	1.397
3		X		X		1,000	0.995	1.149	1.141	1.128	1.141	1.141
4		X			X	1.297	1.290	1.474	1.462	1.475	1.481	1.481
5			X	X		0.896	0.899	1.100	1.103	1.037	1.052	1.052
6			X		X	1.168	1.168	1.316	1.316	1.244	1.336	1.336
7		X		X		1.082	1.076	1.205	1.195	1.153	1.186	1.186

The influence of cracks in the embankment on the FS_{min} values was in the order of 10 – 15% showing lower results.

The correction of Su was much more important in this study. The stability study was also done using empirical methods (Equation of Load Capacity; Method of the Sliding Wedges; Chart of Pillot and Moreau, 1973; Chart of Pinto, 1966) and the results were satisfactory presenting FS close to 1.0 for the failure condition, using a representative corrected Su Bjerrum proposal (Tab. 5). Details can be seen in Coutinho & Bello (2007).

Due to the dimensions of the "failure surface", the three-dimensional effect (Azzouz et al. 1983) showed an insignificant increase (in the order of 5% in relation to the bi-dimensional FS) in the critical factor of safety. It was utilized the Azzoz et al. (1983) proposed (Eq. 1).

$$\frac{FS^T}{FS} = \left[1 + 0,7\left(\frac{DR}{2L}\right)\right] \qquad (1)$$

One possible solution of stabilization could be the construction of a balance berm. The FS_{min} obtained for H_{berm} = 2.5m for the Hypothesis 3 (50% cracking in the embankment and $Su_{corrected}$) was 1.665 showing the increase of 65%. This simple solution could have avoided the rupture.

The results of the stability analysis / back-analysis confirmed the necessity of the correction of S_u obtained by field vane test for the Recife soft clays using the Bjerrum (1973) proposal for design work, extending the results of Brazilian clays, with the exception of Juturnaíba organic soils deposit (Fig. 10). Due to the characteristic of the clay deposit studied (very soft, very high plasticity and the presence of organic content), which is different of those investigated by Bjerrum (1973). The result can be considered a contribution for local, national (Brazil) and international research and practical work.

Table 5. Resumo de Su (retroanálise) e do FS obtidos através dos métodos expeditos (Bello 2004)

MÉTODOS	Su (kPa) – retroanálise (FS=1)	FS (Su = 20,59kPa)
Capacidade de carga	19,63	1,05
Cunhas deslizantes	19,08	1,06
Pillot e Moreau	20,52	1,10
Ábacos de Pinto	18,00	1,14

CONCLUSIONS

This paper presented the geotechnical investigation and stability analysis of a practical case of a failure of an embankment on soft organic soil in Recife, Brazil (Bello 2004; Coutinho & Bello 2005).

The Data Base of Recife Soft Clays showed to be a good tool to complement technical information in a practical work.

The results of the stability analysis / back-analysis confirmed the necessity of the correction of S_u obtained by field vane test for Recife soft clay, which is a different type of deposit of those considered by Bjerrum (1973), representing a very soft clay deposit with very high plasticity and presence of organic content. The result contributes for local, national (Brazil) and international research and practical work.

The results also showed that the rupture could really have been avoided in an adequate design procedure.

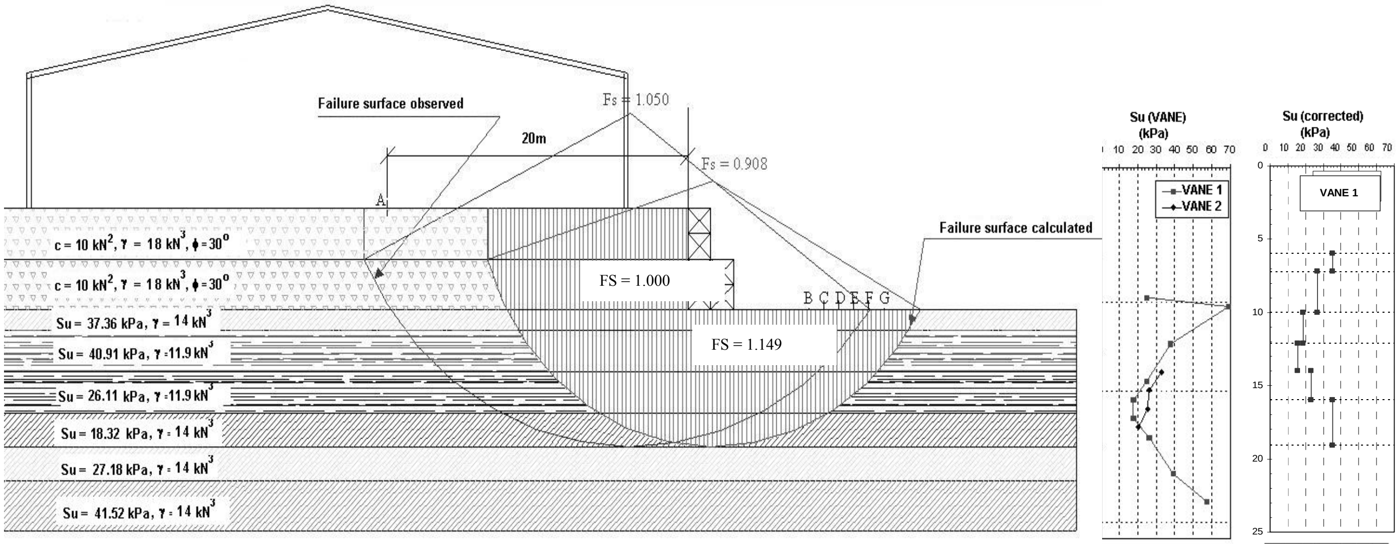
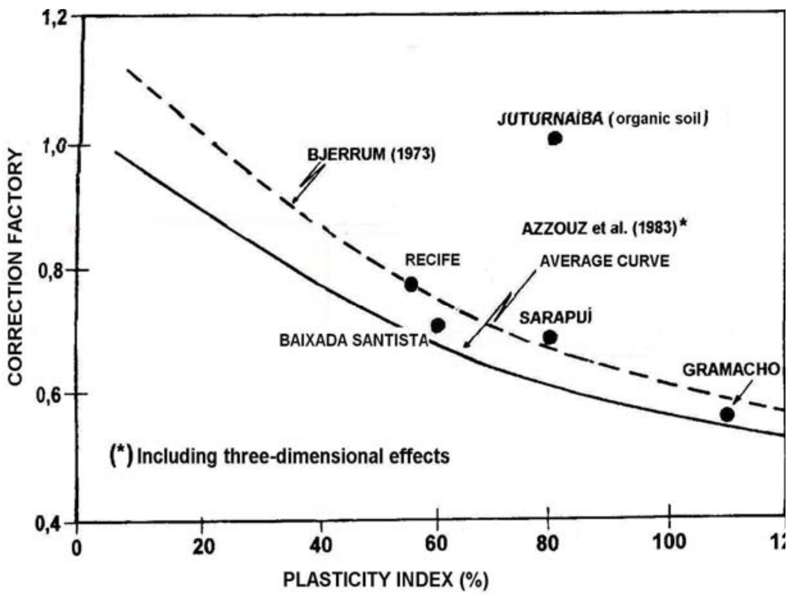
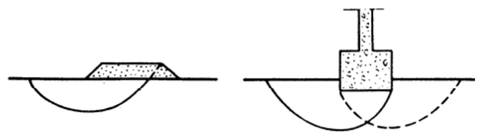


Figure 9. Localization of calculated and observed surface - 50% cracking of embankment - S_u correction (Bello 2004)





$$(S_u)_{FIELD} = (S_u)_{VANE} \cdot \mu$$

Figure 10. Factors of correction from back-analysis of rupture embankments (Coutinho 2000; Sandroni 1993; Massad 1999; and Bello 2004; Coutinho & Bello 2005; Coutinho 2006).

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