Analysis and Control of the Stability of Embankments on Soft Soils: Juturnaíba and others Experiences in Brazil

R.Q. Coutinho, M.I.M.C.V. Bello

Abstract. In the design and evaluation of the behavior of embankments on soft clay foundations, geotechnical characterization, along with instrumentation for measurements involving pore pressure and displacements (vertical and horizontal), are required in order to efficiently elaborate a construction. This paper presents the results of stability analysis, and stability control, with emphases on studies carried out by the Geotechnical Group (GEGEP) of the Federal University of Pernambuco, Brazil. Five Brazilian cases are presented: the Juturnaíba trial embankments and Juturnaíba Dam construction, located in Rio de Janeiro; the access embankments for the Jitituba River Bridge in Alagoas; the failure of an embankment alongside highway BR-101-PE in Recife, Pernambuco; and the Sarapuí trial embankment, located in Rio de Janeiro (Ortigão 1980; Ortigão *et al.* 1983). With the exception of the Juturnaiba trial embankment, all of the cases in the stability analysis concerned failure involving embankments on soft clays confirmed the need to apply the Bjerrum (1973) correction factor to field vane tests measuring undrained strength. Effective stress stability analysis utilizing a normally consolidated effective stress parameters for strength measurements, presented results that can be considered satisfactory. Stability control was carried out using measurements of displacements, deformations, and pore pressures. Proposals were presented and analyzed, especially for horizontal displacements, demonstrating good results, and the potential for practical application. Recommendations for use are presented in the paper. The joint results of stability analysis, and stability control, showed the importance of having an FS > 1.3 to guarantee adequate behavior and security.

Keywords: embankments on soft soils, stability analyses, performance, monitoring, and stability control.

1. Introduction

The construction of embankments on soft clays raises an important geotechnical concern that has been studied by various authors, with the sum of their experiences adding to the overall understanding of soft soils when subjected to load increases (*e.g.* Bjerrum 1973; Tavenas & Leroueil 1980; Ladd 1991; Leroueil & Rowe 2000; Coutinho & Bello 2005; Almeida & Marques 2010). In Brazil, important research studies include those carried out by Ortigão (1980), Coutinho (1986) and Magnani de Oliveira (2006). In general, the design of embankments on soft soils should meet the basic stability requirements needed to resist rupture and displacement (vertical and horizontal), during and after construction, while remaining compatible with the objectives involved. Instrumentation is a tool used for monitoring and evaluation (including stability control) during the construction of embankments, where measurements are taken of horizontal and vertical displacements, along with pore-pressure.

This paper presents the results of stability analysis, and stability control, with emphasis on studies carried out by the Geotechnical Group (GEGEP) of the Federal University of Pernambuco, Brazil. Five Brazilian cases are presented: the Juturnaíba trial embankment (Case Study 1), and the Juturnaíba Dam construction (Case Study 2), both located in Rio de Janeiro; the access embankments of the Jitituba River Bridge (Case Study 3), located in Alagoas; the failure of an embankment alongside highway BR-101-PE (Case Study 4), located in Recife, Pernambuco; and the Sarapuí trial embankment, located in Rio de Janeiro (Case Study 5) (Ortigão 1980; Ortigão *et al.* 1983).

2. Behavior of Embankments On Soft Soil

When analyzing embankments on clay foundations, their behavior has commonly been considered to be entirely undrained during construction, with drainage and consolidation starting after construction ends. This approach has been widely utilized, and has generally performed well for conventional designs. Observations during construction have shown that while this approach may often provide reasonable designs, the actual behavior of embankments may be more complicated, and that conventional undrained analyses may over-predict pore pressures and lateral displacements. Thus, if one wishes to predict the actual behavior of an embankment located on clay, it is essential to have good knowledge of the mechanical behavior of natural clays, and to understand what might happen underneath an embankment during construction (Tavenas & Leroueil 1980; Leroueil & Rowe 2000; Coutinho & Bello 2011).

As in most geotechnical problems, understanding soil response only becomes possible when the corresponding stress path is known. Under embankments, the effective

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stress path can be deduced from pore pressure observations. Significant partial consolidation during construction has been reported by a number of investigators (*e.g.* Tavenas & Leroueil 1980; Ortigão 1980; Coutinho 1986; Leroueil & Rowe 2000).

If the behavior of the clay foundation under an embankment was perfectly undrained, the effective stress path for a point at or near the centerline would be as O'-U' in Fig. 1a (overconsolidation ratio, OCR < 2.5). As a consequence of the rapid consolidation during early stages of construction (very high coefficient of consolidation, c_v in the pre-consolidation condition), the effective stress path may be O'-P', and reach the limit state curve at P', at a vertical effective stress, σ'_v, close to the pre-consolidation stress, σ'_p, of the clay, corresponding to an increase in pore pressure much smaller than the increase in total stress ($\overline{B}_1 = \Delta u/\Delta\sigma_v < 1.0$; $\overline{B}_m = 0.6$). As the clay becomes normally consolidated, its coefficient of consolidation is reduced by a significant amount, and the behavior becomes essentially undrained. Due to the shape of the limit state curve of natural clays, further loading is associated with a stress path such as P'-A', under a vertical effective stress, which is essentially constant, equal to σ'_p. Such a stress path corresponds to an increase in pore pressure equal to an increase in total stress ($\overline{B}_2 = \Delta u/\Delta\sigma_v = 1.0$) during the second phase of loading.

If the embankment is built to a height in excess of that corresponding to point A (Fig. 1a) the effective stress path will continue up to F', on the strength envelope of the normally consolidated clay, resulting in local failure, possibly to the critical state C''. Between F' and C', the increase in excess pore pressure is larger than the increase in total stress ($\overline{B}_3 = \Delta u/\Delta\sigma_v > 1.0$) as shown in Fig. 1b. It should be noted that $\overline{B}_1$, $\overline{B}_2$ and $\overline{B}_3$ discussed above, are incremental values during different stages of loading and do not correspond directly to the conventional $\overline{B} = \Delta u/\Delta\sigma_v$ under the entire loading (for $\Delta\sigma_v = I\gamma H$). Hence a high value for $\overline{B}_3$ does not necessarily mean that the embankment is unstable. Pore pressure may develop even after construction is completed, *i.e.* when there is no increase in total stress, but when $\overline{B}$ may still be less than unity. The pore pressure generated during the construction of an embankment, and the corresponding stress path, have direct influence on settlements and lateral displacements.

3. Case Studies

3.1. Juturnaíba Dam Project – Case Studies 1 and 2

The Juturnaíba Dam Project, an earth-filled structure located in the Northern portion of the State of Rio de Janeiro, in Brazil, was built from 1981 to 1983 (Fig. 2a). The Project included the Juturnaíba trial embankment (Case Study 1) and the Juturnaíba Dam construction (Case Study 2). The two cases are located in areas with similar geotechnical characteristics. The foundation consisted basically of an organic clay deposit about 8 m thick, with SPT values (blows/length in cm) ranging from 0/111 to 1/33, typically 0/50, along its full depth, underlain by sand sediments with SPT values about 10/30, reaching a depth of 14 m. Visual classification and laboratory tests permitted division of the clayey deposit into six layers, with variations in organic and water content, ranging from light-grey silt clay, to brown clayey peat (Fig. 2b).

Figure 3 shows results involving natural water content, and Atterberg Limit values for the six layers of the profile. Variations of these results can be observed for each layer, and consequently in the plasticity index values. Results are presented in Fig. 4 concerning overburden effective stress (σ'_{vo}) and preconsolidation pressure (σ'_p) obtained by oedometer tests. The foundation deposit exhibits a condition of overconsolidation, with the upper part show-

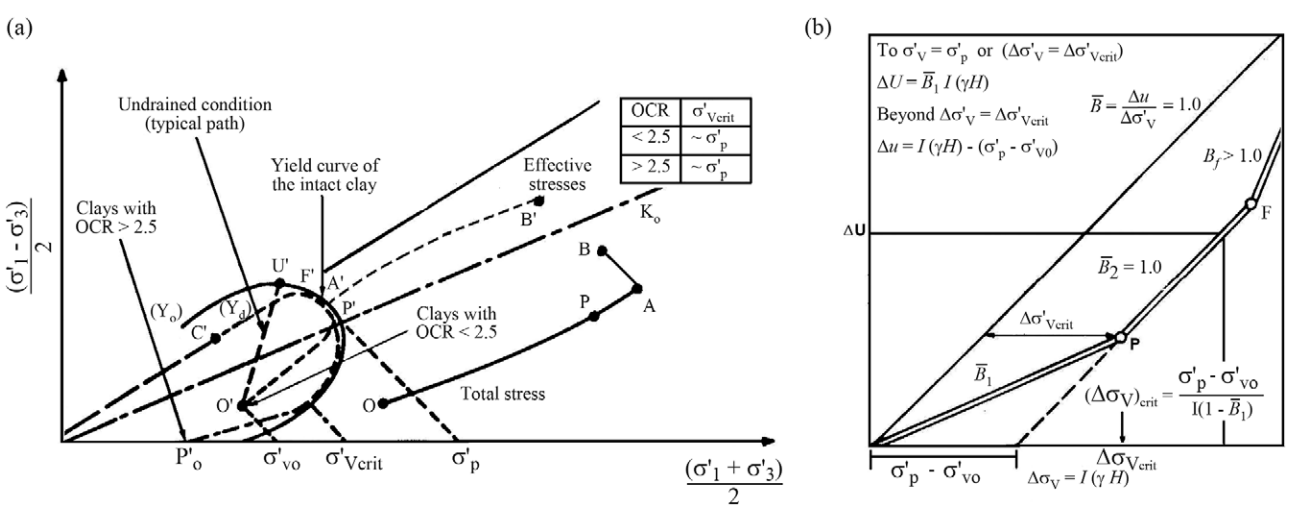


Figure 1 - (a) Total and effective stress path, and (b) increase of pore pressure under the centerline during stage construction of an embankment on soft clays (Coutinho 1986, from Tavenas & Leroueil 1980).

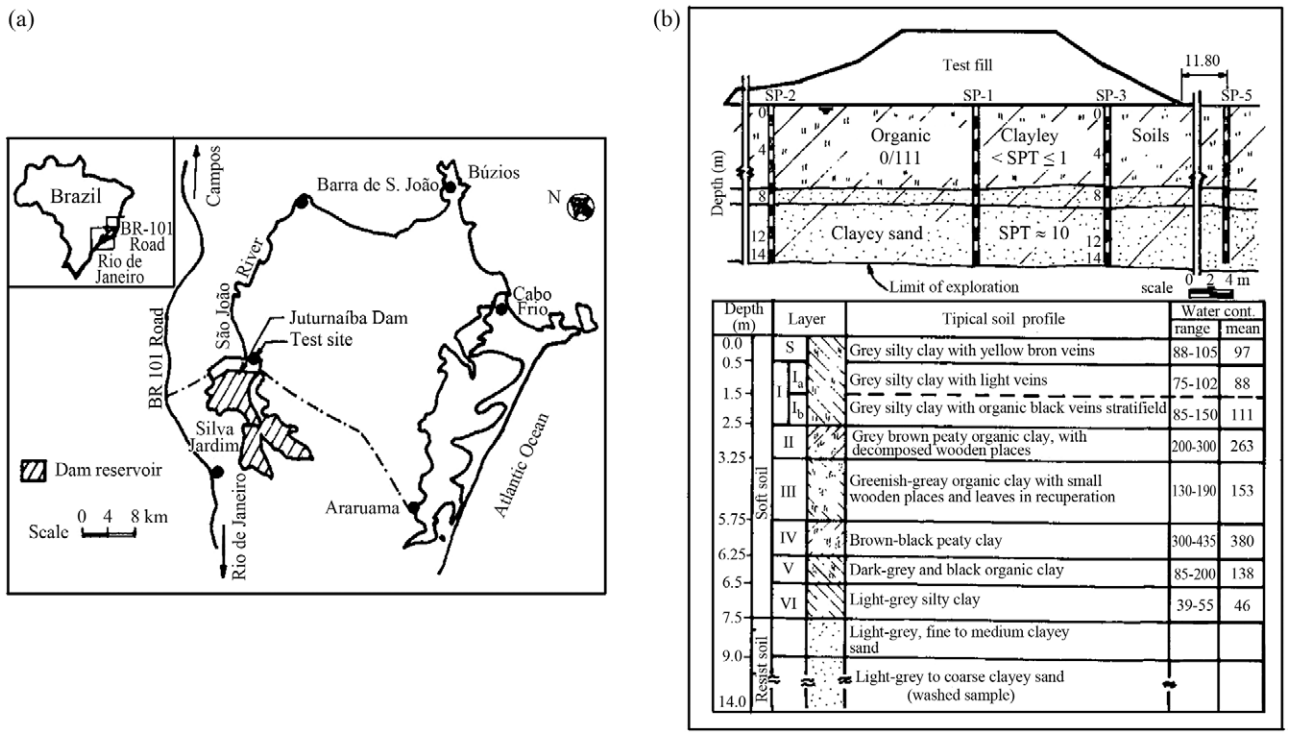


Figure 2 - Juturnaíba Dam Project: (a) Localization; (b) typical soil profile (Coutinho 1986; Coutinho & Lacerda 1987).

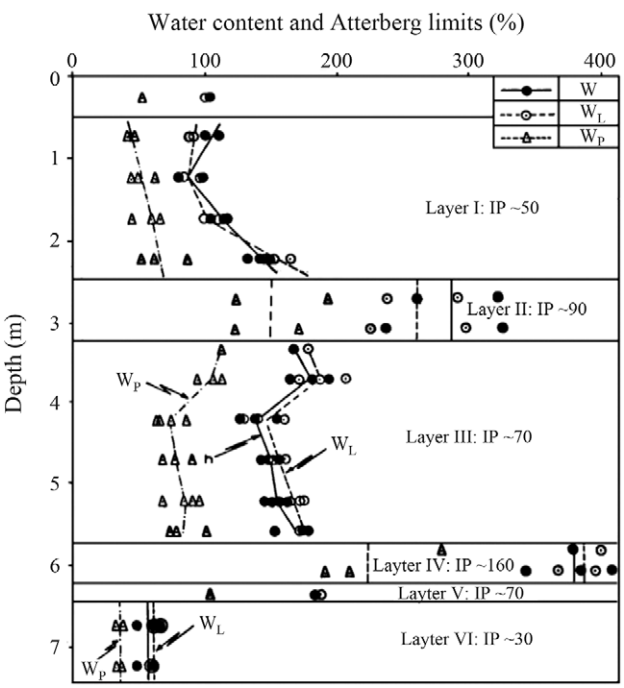


Figure 3 - Water content and Atterberg Limits of the Juturnaíba trial embankment (Coutinho 1986).

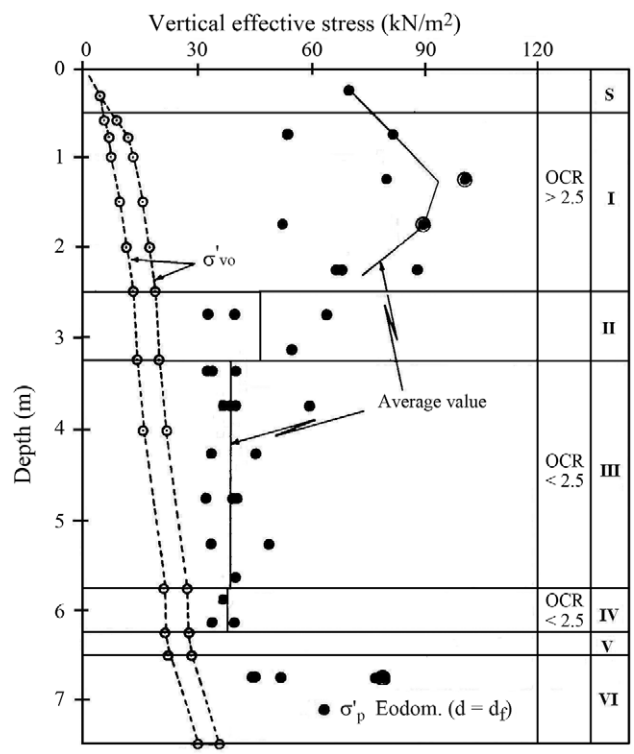


Figure 4 - σ'_{vo}, $\sigma'_{v\,crit}$, σ'_{v}, σ'_{p} laboratory values *vs.* depth – center of the Juturnaíba trial embankment (Coutinho 1986).

ing higher values (OCR > 2.5). Figure 5 shows the results of compressibility parameters, with values indicated for compression ratio (*CR*), swelling ratio (*SR*), void index (e_o), compression index (C_c), and organic content, differing for each layer.

Because 1.2 km of the length of this earth dam was built to rest on organic soft clay, geotechnical studies were

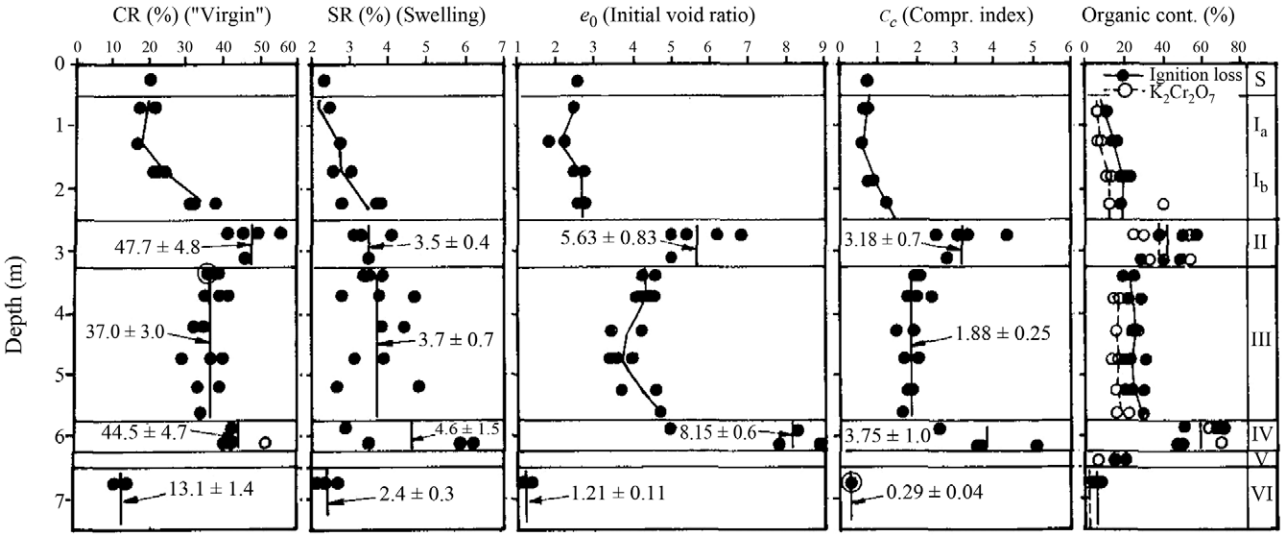


Figure 5 - CR, SR e_o and *Cc vs.* depth – oedometer tests for the Juturnaíba Dam (Coutinho & Lacerda 1987).

quite comprehensive, including laboratory and field investigations, along with construction of a trial embankment leading to failure (Case Study 1), which was instrumented as indicated in Fig. 6 (Coutinho 1986; Coutinho & Lacerda 1987; 1989).

The main purposes of the studies were to provide indications on the undrained strength and compressibility of the clay foundation, and methods to control stability during the construction. Joined by design studies, indications pointed out that the Juturnaíba Dam (Case Study 2) should be built in stages, with berms and flat slopes (Fig. 7). Dam monitoring consisted of placing settlement plates at the embankment-clay interface, with piezometers inside the organic clay, and inclinometers at the slope berm (Coutinho *et al.* 1994; Lucena 1997).

3.2. Access embankments for the Jitituba River Bridge – Case Study 3

Case Study 3 presents the study of the access embankments for the Jitituba River Bridge, located on highway AL-413-Alagoas, with the bridge being built before the access embankments. The geotechnical profile presents a soft clay organic layer between two sand layers, with the thickness of the organic clay increasing towards the direction of the river, to a maximum of 12 m (Fig. 8). Due to the existence of a geotechnical profile composed of soft soil layers, and considering the construction sequence involving the embankments of the Jitituba River Bridge (before the execution of the access embankments), analyses of the vertical and horizontal displacement, and the consequent effects on the pilings of the bridge were recommended. The solution adopted consisted of constructing embankments in

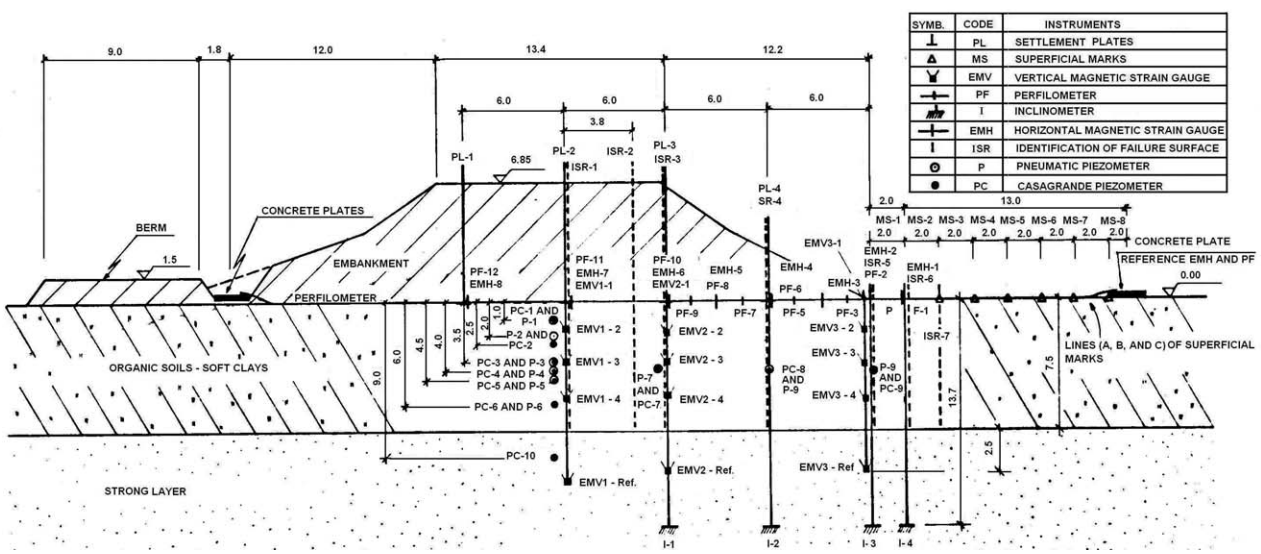
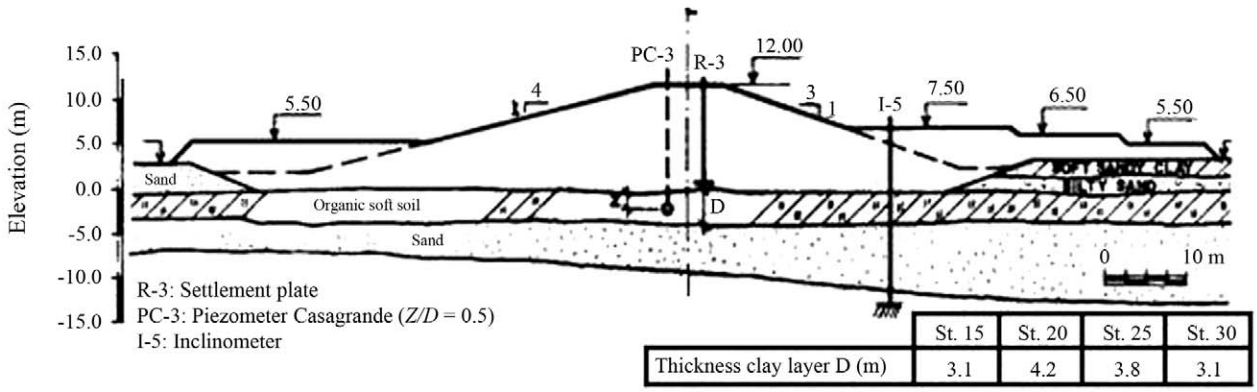


Figure 6 - Instrumentation of the Juturnaíba trial embankment (Coutinho 1986).



	St. 15	St. 20	St. 25	St. 30
Thickness clay layer D (m)	3.1	4.2	3.8	3.1

Figure 7 - Transversal geotechnical profile and instrumentation – Juturnaíba Dam (Coutinho & Lacerda 1989).

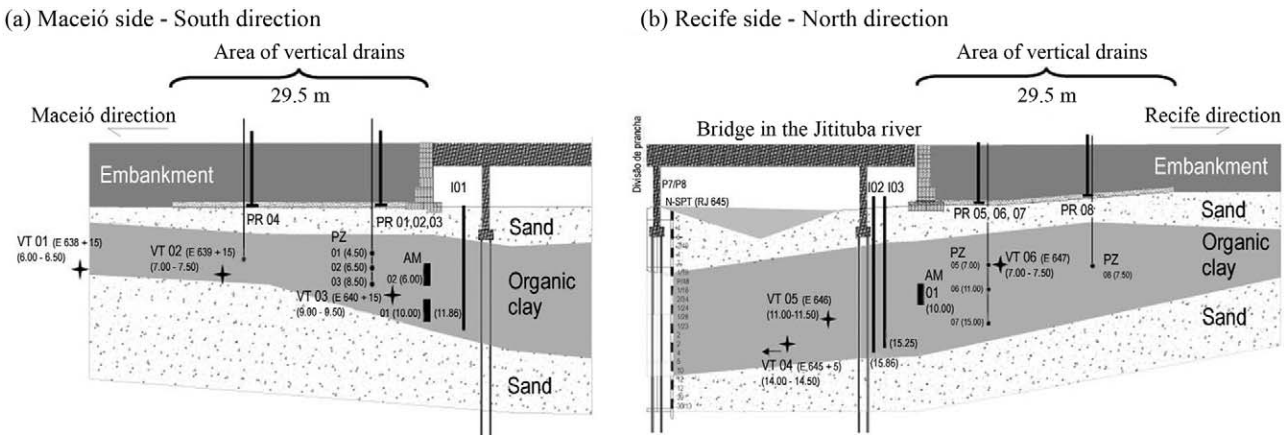


Figure 8 - Longitudinal section, geotechnical profile, and locations of the field investigations for the basic project of the access embankments for the bridge on the Jitituba River (Cavalcante *et al.* 2003; 2004).

stages, allied to the use of prefabricated vertical drains, and geotechnical instrumentation (Casagrande piezometers, settlement plates, and inclinometers) to control and monitor project performance (Cavalcante 2001; Cavalcante *et al.* 2003; 2004). Behavior of the access embankments was analyzed in terms of measurements of pore-pressures, and vertical and horizontal displacements, by applying models proposed in the literature, and by comparison with other case studies of embankments on soft soils.

The research study of the access embankments of the Jitituba River Bridge (Case Study 3) was made possible due to a partnership with Gusmão Engineers Associated.

3.3. Failure of an embankment alongside highway BR-101-PE – Case Study 4

Case Study 4 involves the rupture of an embankment on soft clays that occurred in an area alongside Federal Highway BR-101 - in Pernambuco (Bello 2004; Bello *et al.* 2006; Coutinho & Bello 2005). Figure 9 shows the position of the sheds, and location of the geotechnical field investigations.

Results of the geotechnical profile, natural water content, overconsolidation ratio OCR, and field vane undrained strength *vs.* depth are shown in Fig. 10. Variations can be observed in the results for each soft layer. The geotechnical profile initially presents a fill layer, followed by three layers of soft clays (13 m thick) with different organic content/water content, and finally a silty sand layer. The natural water content, W_n, presented a maximum value (334%) at 7 m of depth, becoming constant (34%). The undrained shear strength S_u demonstrates variation with depth, and a minimum value of 17 kPa at 13 m depth. The OCR value in general is close to 1. The embankment was constructed without any geotechnical investigation project, monitoring, or technological control. After the failure, in order to understand the process, *in situ* and laboratory tests were performed so as to permit total stress stability analysis/back-analysis. The Data Base for Recife Soft Clays (Coutinho *et al.* 1998) was used to complement the technical information needed to carry out the study.

The research work for the embankment alongside highway BR-101-PE (Case Study 4) was made possible thanks to a partnership with Gusmão Engineers Associated.

3.4. Sarapuí trial embankment – Case Study 5

An extensive research program concerning the behavior of embankments located on soft soils, sponsored by the Brazilian Highway Research Institute (IPR), was conducted together with COPPE-UFRJ, at the Federal University of Rio de Janeiro. It included field and laboratory investigations, theoretical analyses, and large-scale field trial embankments on soft soils, and dark grey deposits. The Sarapuí trial embankment (Study Case 5) was the first instrumented embankment taken to failure (Ortigão 1980; Ortigão *et al.* 1983). It was situated in a very flat swampy area, covering a surface area of about 150 km^2 around Guanabara Bay. At the site, the clay deposit is about 11 m thick, and overlies sand and gravel layers. As may be seen from Fig. 11, the liquid limit varies from 120%-160% near the ground surface, to 90%-100% at the bottom of the de-

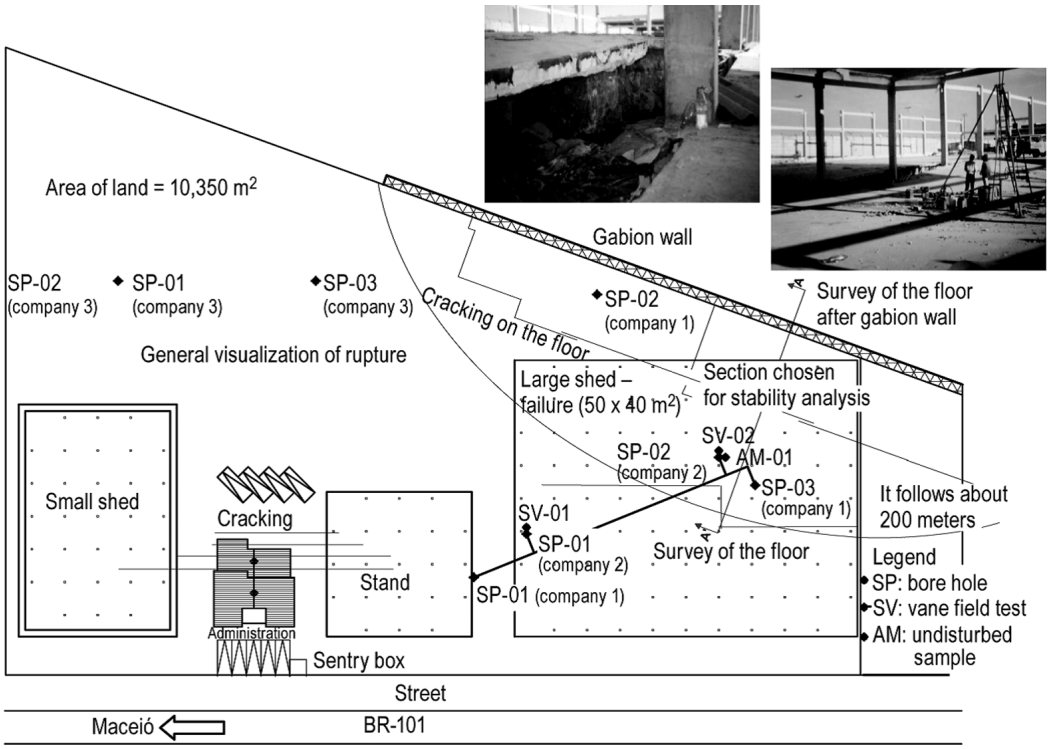


Figure 9 - Situation and localization of SPT soundings, vane field tests and undisturbed sampling – embankment alongside highway BR-101-PE (Bello 2004).

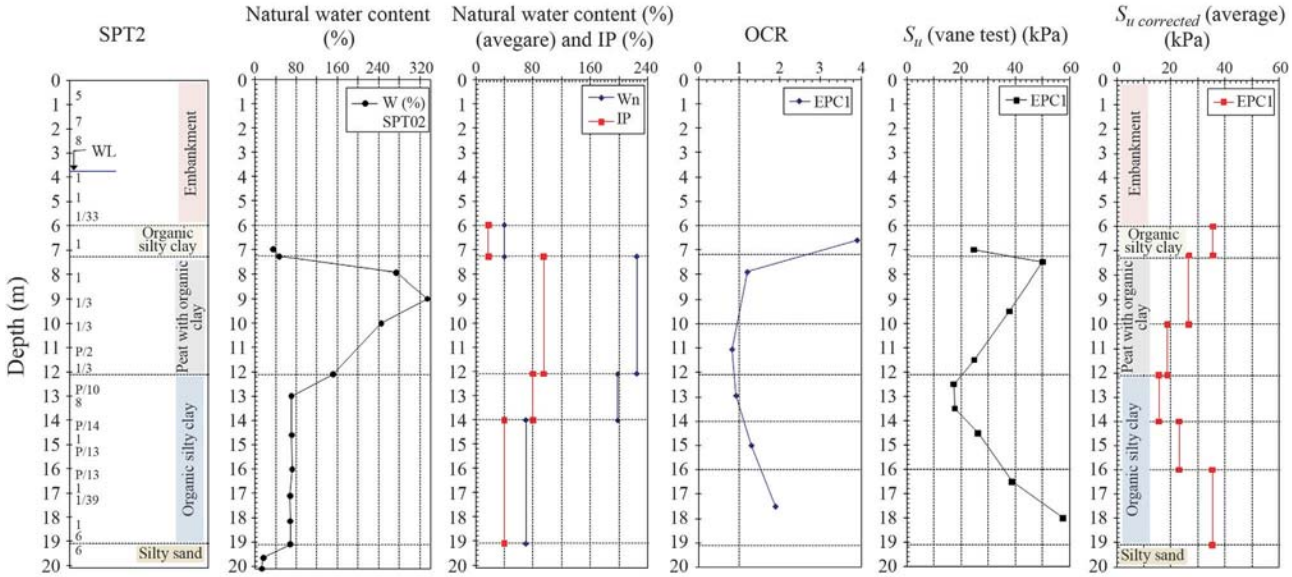


Figure 10 - Profile of embankment alongside highway BR-101-PE – Section AA (Bello 2004).

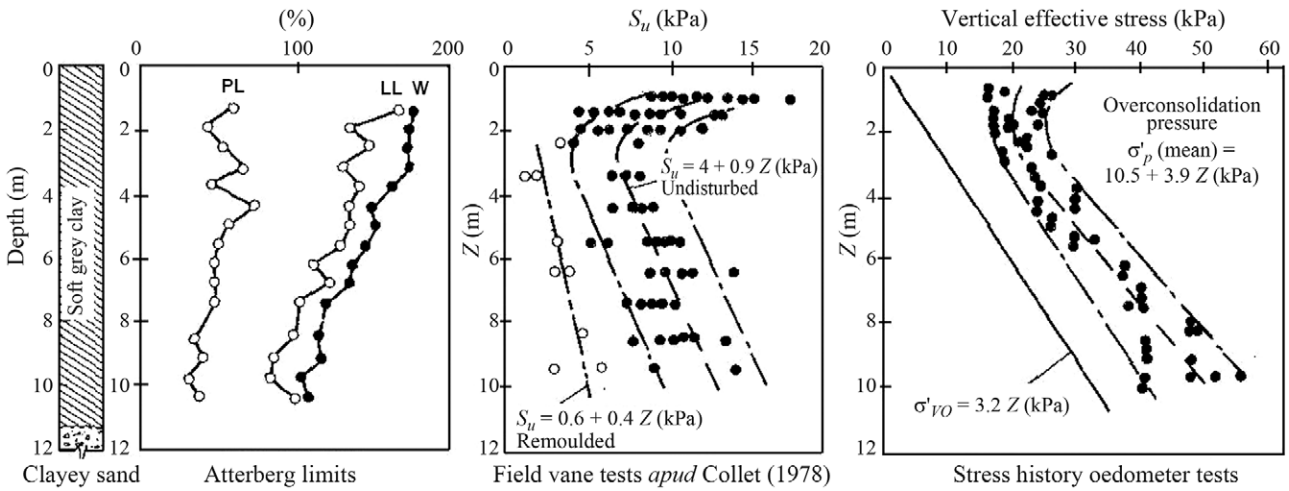


Figure 11 - Summary of Geotechnical properties, Rio de Janeiro soft gray – clay (Ortigão *et al.* 1983).

posit. The natural water content is slightly higher than these liquid limit values. The sensibility of this deposit is low, ranging from 2-4, with an average of 2.6. The plastic limit decreases from 60%-80% near to top, to 50%-60% at the bottom. The results of pre-consolidation pressure (σ'_p) indicated the presence of an upper clay crust, extending down to a depth of 2.5 m. The undrained shear strength (S_u) values seem to initially decrease with the depth, until reaching 2.5 m; below this level, field vane tests indicate increasing S_u values.

4. Stability Analysis

Ladd (1991) defined three types of stability analysis for embankments on soft soils: (a) total stress analysis (TSA); (b) undrained strength analysis (USA), and (c) effective stress analysis (ESA).

Total stress analysis is often used in single-stage construction analysis, and is usually based on the undrained strength profile prior to construction. In undrained strength analysis, the in situ undrained shear strength is computed as a function of the pre-shear effective stress. This analysis is often used in evaluating the stability of embankments that are constructed in stages.

Evaluation of mobilized undrained shear strength S_u, in an embankment constructed in one stage, can be carried out using several approaches: (a) the field vane test approach, with the Bjerrum (1973) correction factor, μ; (b) pre-consolidation pressure S_u/σ'_p = 0.19 (plasticity index PI = 10%) to about 0.28 (PI = 80%). The upper values often correspond to organic clays; (c) recompression and SHANSEP approaches; (d) the direct simple shear test; (e) the unconfined and unconsolidated undrained compression test, and (f) piezocone penetration tests, and Marchetti dilatometer tests. To gain confidence in the results of stability analyses, it is recommended that at least two of these approaches be considered in practical applications. In the case of an embankment constructed in several stages, the selection of strength can be obtained using several approaches: (a) field vane test approach, without the Bjerrum (1973) correction; (b) CPTU tip resistance approach; (c) vertical effective stress approach (S_{uv}/σ'_{vo} = 0.25); and (d) SHANSEP approach (Leroueil & Rowe 2000).

Skempton (1957) suggested the general correlation for S_u be determined from the field vane shear test (VST), as a function of the plasticity index. All of the data concerns normally consolidated (NC) clays. A linear depiction of this data results in Eq. (1).

$$S_{u\,(VST)} / \sigma'_{vo} = 0.11 + 0.0037\,PI \qquad (1)$$

Coutinho (1986) and Coutinho *et al.* (2000) offered a general discussion concerning field vane testing, and presented results of undrained shear strength for some Brazilian soft clay deposits. Results of the S_{uv}/σ'_p ratio *vs.* plasticity index from Juturnaíba and Recife research site deposits RRS1 and RRS2, and from other Brazilian clays, are presented in Fig. 12. It can be seen that, in general, Brazilian soft clays with PI < 80% are in agreement with correlations proposed in the literature. If all soils are considered, including organic soils, a modified Skempton (1957) correlation

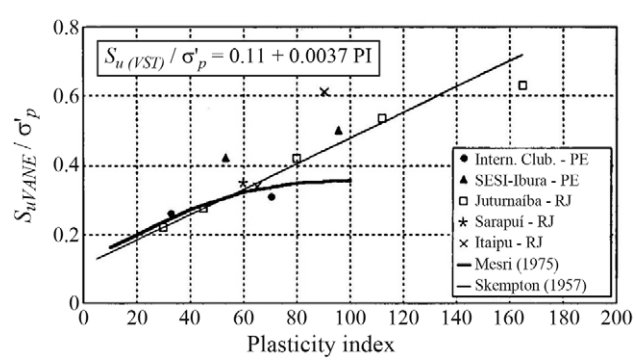


Figure 12 - S_{uVST}/σ'_p *vs. PI* correlation (mod. Skempton 1957, Bjerrum 1973) including values for Brazilian clays (Coutinho *et al.* 2000).

may also be valid for OC clays, using pre-consolidation stress (σ'_p) in place of the overburden effective stress (σ'_{vo}).

In the effective stress analysis approach, mobilized strength parameters are close to those for normally consolidated clay. Theoretical and practical difficulties involving effective stress analysis have been observed, including concerns regarding accurate measurement of pore pressure along the failure surface (Tavenas *et al.* 1980; Ortigão 1980; Coutinho 1986).

In the stability analysis of embankments on soft soils, investigation of the critical shape of the failure surface (circular and non-circular) is important.

This item presents results of the stability analysis performed on the Juturnaíba trial embankment (Case Study 1), and the embankment alongside highway BR-101-PE (Case Study 4). Results of stability analysis performed on the Sarapuí trial embankment (Case Study 5) were used in the discussion as a complement for the Brazilian experiences. Others cases can be seen in Magnani de Oliveira *et al.* (2010) and Almeida *et al.* (2010).

4.1. Total stress analysis results – Case Study 1

Coutinho (1986) performed a total stress stability analysis on the Juturnaíba trial embankment (Case Study 1) to obtain the minimal Safety Factor, SM. The principal analysis was performed using the Modified Bishop method, which takes the circular surface of the rupture into account. The Modified Janbu method was utilized in a complementary analysis (back-analysis) which took into account the specific shape of the failure surface. In the study, a total of seven hypotheses were established considering the S_u profile obtained from the *in situ* vane test (average ± standard deviation), and the cracking of the embankment (Fig. 13). The fill strength parameters (cohesion c = 29,1 kN/m^2, and friction angle ϕ = 29°), were determined from direct shear strength. The analysis was performed for a height of 6.85 m, at which point the failure of the foundation occurred, and also for a height of 8.85 m, to be able to evaluate the behavior of the embankment after failure, which occurred while the embankment was still under construction.

Figure 14 shows S_u values obtained in the field vane test, and in the triaxial UU and CIU ($\sigma'_c \cong \sigma'_{oct}$ *in situ*) laboratory tests, as well as the mean values and the range of field vane shear tests results, with the equations for each of the six soil layers. The following points emerged by analyzing and comparing data: (a) Individual values, and linear regression of the S_u field vane test (Fig. 14a) were basically distinct for each layer. In the tests with "remolded" soil, S_u values were low, with sensitivity near or equal to 10, showing great dispersion and little variation among the layers; b) S_u values from UU and CIU laboratory tests are very similar, and fall close to mean vane shear strength results (Fig. 14b). Results from CIU tests present smaller dispersion than the UU values; c) S_u results in layer III from the triaxial tests are practically constant with the depth, which agrees well with results of the maximum past preconsolidation pressure (Fig. 5); d) The Mesri (1975) proposal for the "mobilized" $S_u = 0.22\sigma'_p$ showed lower results, as expected, than those obtained directly from the triaxial and vane tests (Fig. 14b).

Table 1 shows the summary of the analysis results (minimal Safety Factor, SM values). The stability analysis for the 6.85 m embankment height (Juturnaíba failure condition) considering the average Su from the field vane test performed before construction, without the Bjerrum correction, obtaining values of 1.069, 1.001 and 0.960 for SM, depending on the consideration of embankment cracking in hypotheses 1, 4 and 5 respectively.

With consideration of the S_u vane range (mean values ± standard deviation) and the strength of the embankment (0% cracking), the results of the minimal Safety Factor obtained were 1.274 and 0.888, respectively. The use of the Bjerrum correction would show very low results (SM << 1.0), considering the average high plasticity of the soft deposit, and consequently, the very low correction factor. Back-analysis carried out using the failure surface observed, showed satisfactory results for SM, displaying values close to the preliminary analysis values (in the order of 5 to 9% higher).

Figure 15 presents the minimal Safety Factor results for embankment heights considering the hypothesis of average S_u from the field vane test, and the effect of embankment cracking on the results. An appreciable reduction in SM value can be observed with the continuity of loading, particularly at embankment heights over 5.65 m (SM = 1.31). The influence of cracks in the embankment on

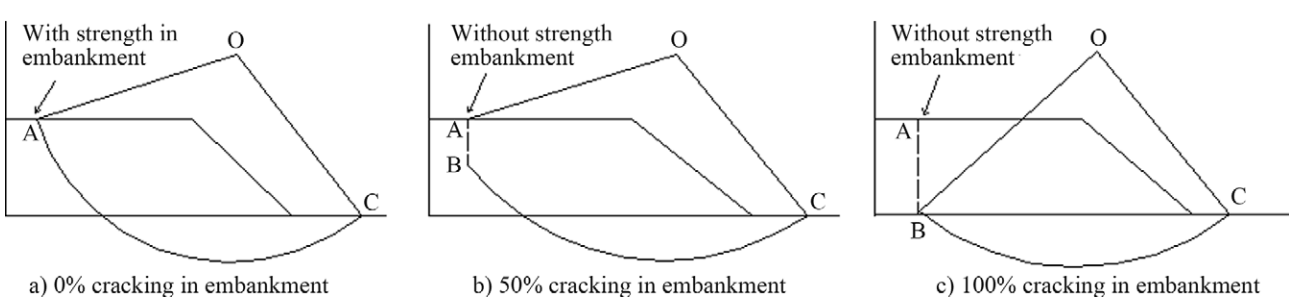


Figure 13 - Consideration of strength of the embankment in the stability analysis (Bello 2004).

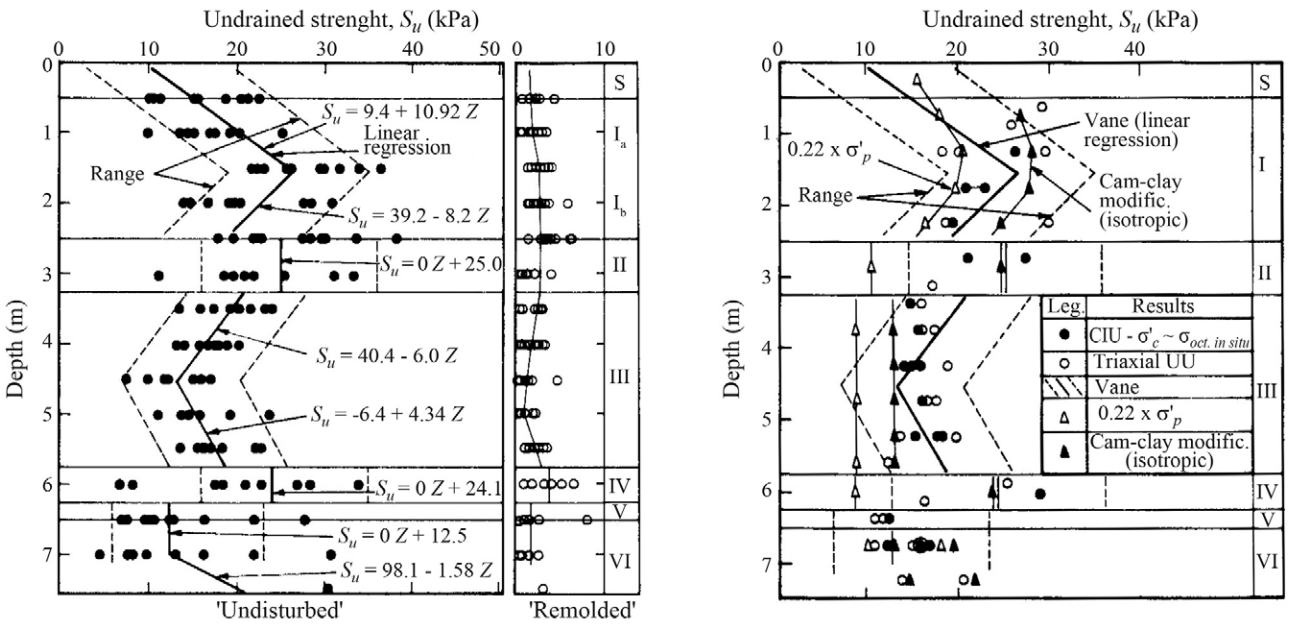


Figure 14 - Undrained strength values S_u *vs.* depths – Juturnaíba trial embankment: (a) S_u from field vane tests; (b) S_u triaxial tests (Coutinho & Lacerda 1989).

the SM values was in the order of 10%. The stability analysis for the 8.85 m high embankment (construction post failure) demonstrated SM results of around 1.0, confirming the rupture condition.

The dissimilar behavior involving the Juturnaíba foundation, which did not need the Bjerrum correction, can be attributed to the organic condition of the soil deposits, extensive drainage, and significant deformation from the increase in stress during the construction phase (see Coutinho 1986; Coutinho & Bello 2011). The analysis also shows that the Mesri (1975) proposal does not adequately represent the mobilized strength. Sandroni (1993) presents another organic soil case (Camboinhas trial excavation-RJ) where application of the Bjerrum (1973) S_u correction proposal is not necessary.

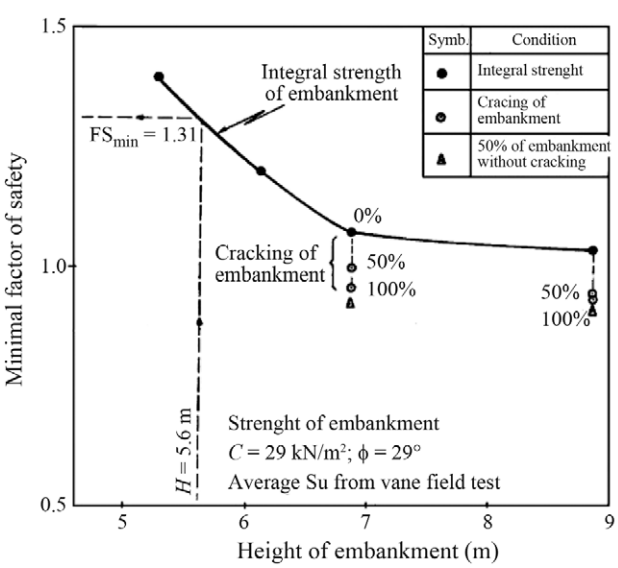


Figure 15 - Summary of the total stability analysis results for the Juturnaíba trial embankment (Coutinho 1986).

To understand the failure process better, after construction of the 8.85 m high embankment was completed, the embankment was excavated to be able to observe its condition, along with that of the foundation during the work. Figure 16 represents what was observed, demonstrating a shared failure for both sides, with a slick side zone in the foundation, and a loose zone in the embankment. Because of this type of shared failure, what was not observed was the traditional movement of embankment mass in one horizontal direction (during the failure of the 6.85 m embankment).

In the Juturnaíba trial embankment, the critical circle predicted was very similar to the failure surface observed *in situ*, and only slightly dislocated towards the left. It was observed that a circular surface (or similar shape) tangent to the resistant layer can represent the failure surface (Coutinho & Bello 2011).

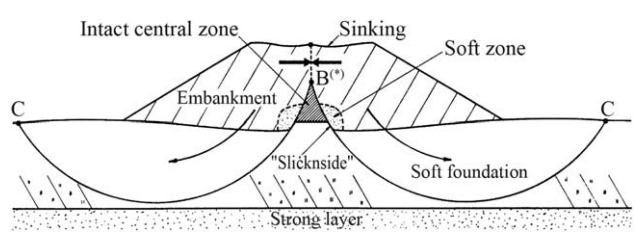


Figure 16 - Shared failure of the Juturnaíba trial embankment (Coutinho 1986).

Table 1 - Summary of the total stress stability analysis, SM_{min} calculated (modified from Coutinho & Bello 2007).

Case study	Hypothesis	(%) Cracking of embankment				S_u		Circular surface		Non circular surface			
								Analysis		Back-analysis			
		0	50	100	Correction	Average	Standard deviation	Bishop	Spencer	Bishop	Janbu	Spencer	Morgenstern-Price
(Case 1) Juturnaíba trial embankment – Coutinho (1986)	1	X	-	-	-	X	-	1.069	-	1.165	1.205	-	-
	2	X	-	-	-	X	(-) X	0.888	-	-	-	-	-
	3	X	-	-	-	X	(+) X	1.274	-	-		-	-
H_{emb} = 6.85 m (Hypothesis 1 to 7)	4	-	X	-	-	X	-	1.001	-	1.046	-	-	-
	5	-		X	-	X	-	0.960	-	1.007	-	-	-
	6	-	X	-	-	X	(+) X	1.200	-	-	-	-	-
	7	-	-	X	-	X	(+) X	1.195	-	-	-	-	-
H_{emb} = 8.85 m (Hypothesis 8)	8	-	X	-	-	X		0,948					
(Case 4) Embankment alongside highway-101-PE – Bello (2004)	1	X	-	-	X	-	-	1.045	1.048	-	1.240	1.232	1.232
	3	-	X	-	X	-	-	1.000	0.995	-	1.128	1.141	1.141
	7	-	X	-	X	-	-	1.082	1.076	-	1.153	1.186	1.186

The stability study was also performed using empirical methods (Load Capacity Equation; the Sliding Wedge Method; Pillot and Moreau Chart; Pinto Chart, and representative use of S_u. Table 2 shows results obtained from the Juturnaíba and Sarapuí trial embankments. The results were satisfactory, which encourages application of these methods for preliminary results, especially the load capacity and sliding wedge methods.

The three-dimensional effect was also studied using the Azzouz *et al.* (1983) proposal. In the Juturnaíba trial embankment, the results from the three-dimensional minimal Safety Factor (SM_{3D}) were on the order of 10% higher than the bi-dimensional value.

The initial Sarapuí embankment analysis suggested that the first indication that failure was being approached occurred when the embankment height was 2.5 m. Until then the instruments had not shown any signs of imminent failure. On the following day, the embankment height was lifted to 2.8 m and after was raised to 3.1 m. The height of embankment of 2.8 m was considered that produced the failure. The total stability analysis suggested that Bjerrum (1973) correction factor, μ, may not be applicable (Ortigão 1980; Ortigão *et al.* 1983). Later Sandroni (1993) presented a discussion about the Sarapuí embankment demonstrating the Bjerrum (1973) proposal applicable when the failure height embankment was reevaluated, and the three-dimensional effect was considered (μ is 0.7 for SM of 1.45).

4.2. Total stress analysis results – Case Study 4

Bello (2004) performed total stability analysis of the embankment alongside highway BR-101-PE. The study seeks to comprehend the failure, and confirm the necessity to correct S_u from the vane field test on Recife soft clays. Sub-layers were defined, with varying soils and respective S_u values from field vane tests, corrected by the Bjerrum (1973) proposal. SM calculations were made using the Modified Bishop, Janbu, Spencer and Morgenstern-Price methods. Table 1 shows a summary of the minimal Safety Factor results obtained from stability analysis, together with back-analysis for three hypotheses regarding the cracking of the embankment, and the use of the corrected S_u value. In the stability analysis, the SM values are in the range of 0.995 to 1.082 for the circular surface condition, depending on the strength of the embankment. Hypothesis 3 (embankment 50% cracking and S_u corrected) presented SM equal to 1.00, explaining the rupture (Table 1 and Fig. 17). The influence of embankment cracks on the SM values ranged from 10% to 15%. In the back-analysis, SM results indicated a range of values close to those for the preliminary stability analysis (around 15% higher). This difference may be due to the difficulty of defining the failure surface for this case while in the field. The predicted critical circle was very similar to the failure surface observed *in situ*, and dislocated only slightly towards the left.

Table 2 - Summary of the S_u (back-analysis) and SM values obtained from empirical methods (Coutinho & Bello 2007).

Method	Height of embankment in a rupture (m)			Representative S_u value (kPa)			S_u (kPa) back-analysis (SM = 1)			SM representative S_u (kPa) value		
	(1)	(2)	(3)	(1)	(2)	(3)	(1)	(2)	(3)	(1)	(2)	(3)
Load capacity equation (Terzaghi 1943)	2.5	6.85	6.0	9.0	19.0	20.59	9.4	19.7	19.63	1.07	0.96	1.05
Sliding Wedges (NAVFAC 1971)							9.7	-	19.08	*	-	1.06
Pillot & Moreau (1973)							9.5	17.3	20.52	*	1.06	1.10
Abacus of Pinto (1966)							-	-	18.00		-	1.14

*The author did not calculate SM for representative S_u value.
Number of authors: (1) Sarapuí (Ortigão 1980); (2) Juturnaíba (Coutinho 1986); (3) Recife (Bello 2004).
Specific weight of embankment: (1) γ_{emb} = 18.4 kN/m^3; (2) γ_{emb} = 15.8 kN/m^3; (3) γ_{emb} = 18.0 kN/m^3.

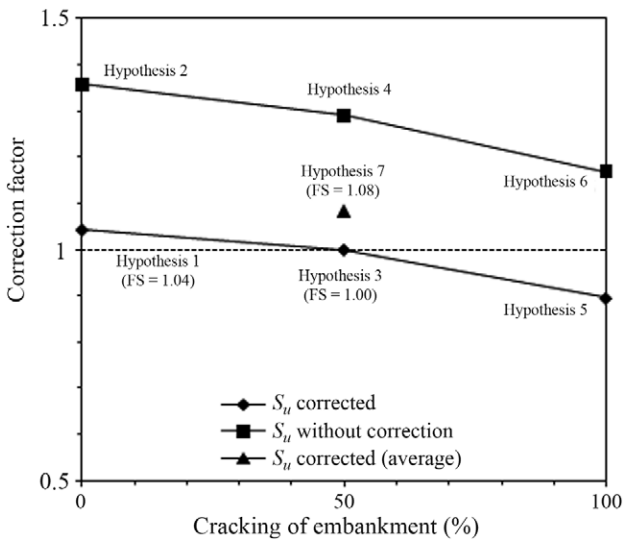


Figure 17 - Results of stability / back analysis (circular surface) - Bishop Method, embankment alongside highway BR-101-PE (Bello 2004).

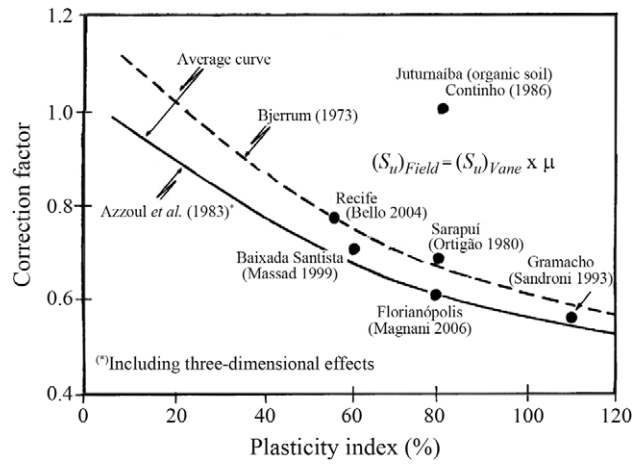


Figure 18 - Correction factors from back-analysis of rupture embankments (Bello 2004; Coutinho & Bello 2005; Coutinho 2006; Sandroni 2006).

The stability study was also performed using empirical methods (Load Capacity Equation; the Sliding Wedge Method; Pillot and Moreau Chart; Pinto Chart), and using a representative Su value for each case. Table 2 shows results obtained for the embankment alongside highway BR-101-PE. The results were satisfactory, which encourages application of these methods for preliminary results, particularly the load capacity and sliding wedge methods.

The three-dimensional effect was also studied using the Azzouz *et al.* (1983) proposal. The results showed a small increase in the three-dimensional SM value, on the order of 5% for the embankment alongside highway BR-101-PE.

4.3. Bjerrum (1973) correction factor, μ - Study Cases 1 and 4

Figure 18 presents the Bjerrum (1973) and Azzouz *et al.* (1983) proposal for the correction factor, μ, to be applied in Su values from field vane test, in order to obtain the undrained strength for design. The results from Brazilian experiences are also shown. It can be seen that the Brazilian results (high plasticity clays) validated the Bjerrum proposal (in general the two proposals). The Juturnaíba trial embankment results lie outside of the proposal, as it involves a highly organic soil foundation (see item 4.1). Stability analysis for the embankment alongside highway BR-101-PE showed the need for the correction factor, μ to be applied in S_u values from field vane tests, in order to obtain the undrained strength for design involving the Recife soft clays. In this case, the average result obtained was μ = 0.8.

4.4. Effective stress analysis results – Case Study 1

Coutinho (1986) carried out effective stress stability analysis on the Juturnaíba trial embankment in order to obtain the minimal Safety Factor SM, for the height that caused failure of the embankment (H_{emb} = 6.85 m). This

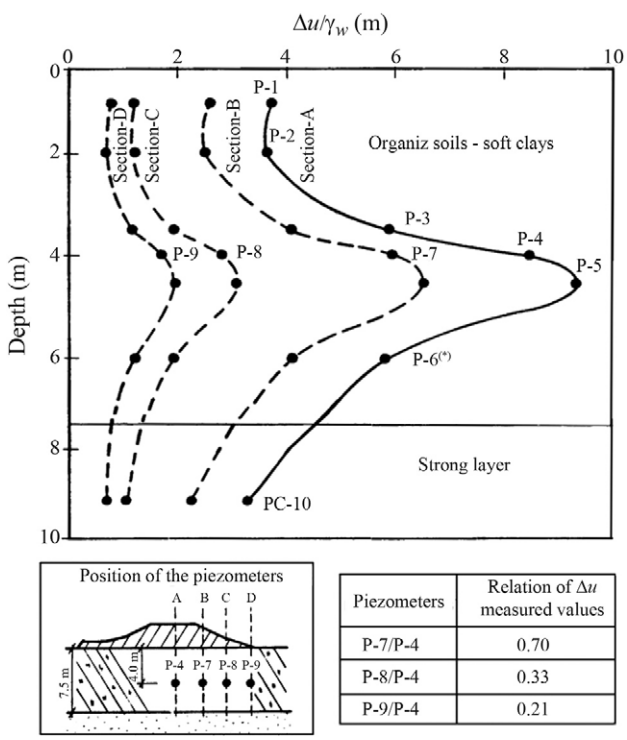
analysis was performed basically through use of the Modified Bishop method, using preconsolidated, and normally consolidated effective strength parameters, and pore pressure measurements obtained through the use of pneumatic piezometers (Figs. 4 and 19). The fill strength parameters (cohesion $c = 29$ kN/m^2 and friction angle $\phi = 29°$) were determined from direct shear strength.

Analysis considering normally consolidated effective stress parameters for strength ($c = 0$ and ϕ' = average of 39°; ϕ' different in each layer of soft soil) presented satisfactory results, particularly when cracking of the embankment was considered to simulate the failure. The SM value obtained ranged from 0.95 to 1.23. In the case of 50% cracking of the embankment, the SM value equaled 1.05.

The predicted critical circle for the effective analysis was distinct for the failure surface observed in situ. The predicted circle presented a smaller extension in area and maximum depth. The observed failure surface showed values for the minimal Safety Factor greater than the corresponding ones obtained in the study of SM. For the case of 50% of cracking of the embankment, and the same effective strength, the SM value was 1.169.

Estimation of pore pressure values at points without piezometers, and the difficulties of measuring the pore pressure at the moment of failure, can cause a reduction in the accuracy of effective stress analysis.

Ortigão (1980) and Ortigão *et al.* (1983) present results of effective stability analysis of the Sarapuí trial embankment. A Minimal Safety Factor well below 1.0 was obtained for failure conditions from the effective stress analysis, considering normally consolidated effective stress parameters for strength. This case shows the difficulties in obtaining good results.



Piezometers	Relation of Δu measured values
P-7/P-4	0.70
P-8/P-4	0.33
P-9/P-4	0.21

Figure 19 - Estimation of pore pressure *vs.* depth – pneumatic piezometer (Coutinho 1986).

5. Stability Control

Field control of the stability of embankments on soft foundations is frequently a means of reducing, or even avoiding the risk of an undesirable failure, and also enables construction on a more rational, and economical basis. Stability control of an embankment can be carried out during construction using the measurement of displacements, deformations, and pore pressures.

This item presents results of stability control carried out on the Juturnaíba trial embankment (Case Study 1), the Juturnaíba Dam (Case Study 2); the access embankments for the Jitituba River Bridge (Case Study 3), and the Sarapuí trial embankment (Case Study 5).

5.1. Pore pressure

Stability control through observation of pore pressures can be carried out using interstitial pressure measurements in an effective stability analysis. In this method, it is necessary to obtain the effective strength parameters, adequate measurement of pore pressures, and the time needed to analyze the results. Results of an effective stability analysis are presented in item 4.3.

The results of increases in pore pressures measured near the middle of a soft deposit of foundation under the center of the embankment may show substantial increases in values when nearing failure. In this case, pore pressure parameter $\overline{B}_3$ ($\Delta u/\Delta\sigma_v$) presents values greater than 1.0 (Fig. 1). According to Tavenas & Leroueil (1980), and Leroueil & Rowe (2000), this condition would be a signal of local failure. The Juturnaíba trial embankment (Case Study 1) showed this behavior, and the result ($\overline{B}_3 > 1.0$) was considered to be a signal of the beginning of failure (Coutinho 1986; Coutinho & Bello 2011).

5.2. Horizontal displacements

Table 3 shows a summary of the stability control proposals for horizontal displacements presented and discussed in this study. In the analysis, two different conditions were considered: (a) embankments designed to be stable: the Juturnaíba Dam (Case Study 2) and the access embankments of the Jitituba River Bridge (Case Study 3); (b) trial embankments induced to rupture: Juturnaíba (Case Study 1) and Sarapuí (Case Study 5). In the stability control process, it is recommended that all behavior be analyzed, not just the value of the measurements, and in practice, more than one stability control proposal be used. Other experiences can be seen in Magnani de Oliveira *et al.* (2010) and Almeida *et al.* (2010).

Table 3 - Summary of the stability control proposals from horizontal displacements.

Analysis methods			Classification
	Maximum value (Y_{max}) *vs.* time (Kawamura 1985)		Convergent behavior → Stable Divergent behavior → Unstable
Y_{max} *vs.* time	Maximum value normalized in function of the thickness of the clay level (Y_{max}/D) *vs.* time		Convergent behavior → Stable Divergent behavior → Unstable
	Velocity of the maximum value normalized ($\Delta Y_{max}/D$)/Δt *vs.* time (Cavalcante 2001; Cavalcante *et al.* 2003)		<< 0,2% / day → Stable > 0.2% / day → beginning to be Unstable
Angular distortion *vs.* time	Maximum value (vd) *vs.* time Construction end value (Ortigão 1980; Coutinho 1986; Cavalcante 2001; Cavalcante *et al.* 2003)		< 3% - Convergent behavior → Stable > 3% - Divergent behavior → beginning to be Unstable
	Velocity of the maximum value (vd) *vs.* time (Almeida *et al.* 2001)		vd ≥ 1.5%/day → Tendency to be unstable 0.5% ≤ vd ≤ 1.5% → Alert, especial attention vd ≤ 0.5%/day → Stable, to continue monitoring
Displaced vertical volume / Displaced horizontal volume	(Vv/Vh) or (ΔVv/ΔVh) *vs.* time	Sandroni *et al.* 2004	(ΔVv/ΔVh > 5) → Stable (3 < ΔVv/ΔVh < 5) → Medium (alert) (ΔVv/ΔVh < 3) → Unstable
		Johnston 1973	3.5-4.2 < ΔVv/ΔVh ≤ 20 → Stable (ΔVv/ΔVh ~ 2.4-1.8) → Unstable
	H_{emb} *vs.* Vh	Sandroni *et al.* 2004	H_{emb} *vs.* Vh increase significantly → Unstable
Y_{max}/D *vs.* SM	(Bourges & Mieussens 1979; Coutinho 1980); Cavalcante 2001; Cavalcante *et al.* 2003)		> 1.8% → Unstable = 1.0% → Stable (SM~1.5) < 0.8%→Stable minimum horiz. displacements

5.2.1. Tendency for horizontal displacements

Analysis of the horizontal displacements was carried out with three considerations: (a) maximum horizontal displacements (Y_{max}) *vs.* time; (b) maximum horizontal displacements, normalized in function of the thickness of the clay layer (Y_{max}/D) *vs.* time; (c) and velocity of the maximum horizontal displacements normalized ($\Delta Y_{max}/D$)/Δt *vs.* time.

(a) Figures 20a and 20b present the evolution of the maximum horizontal (lateral) displacements (Y_{max}) through time, considering the access embankments of the Jitituba River Bridge, and Juturnaíba trial embankment. The main objective of this analysis is to evaluate the possibility of creep rupture (Kawamura 1985). Using this model, rupture from undrained creep is associated with the divergent behavior in the evolution of displacements through time, while the convergent behavior would indicate consolidation and stabilization.

The tendency observed in the access embankments of the Jitituba River Bridge is clearly convergent during and af-

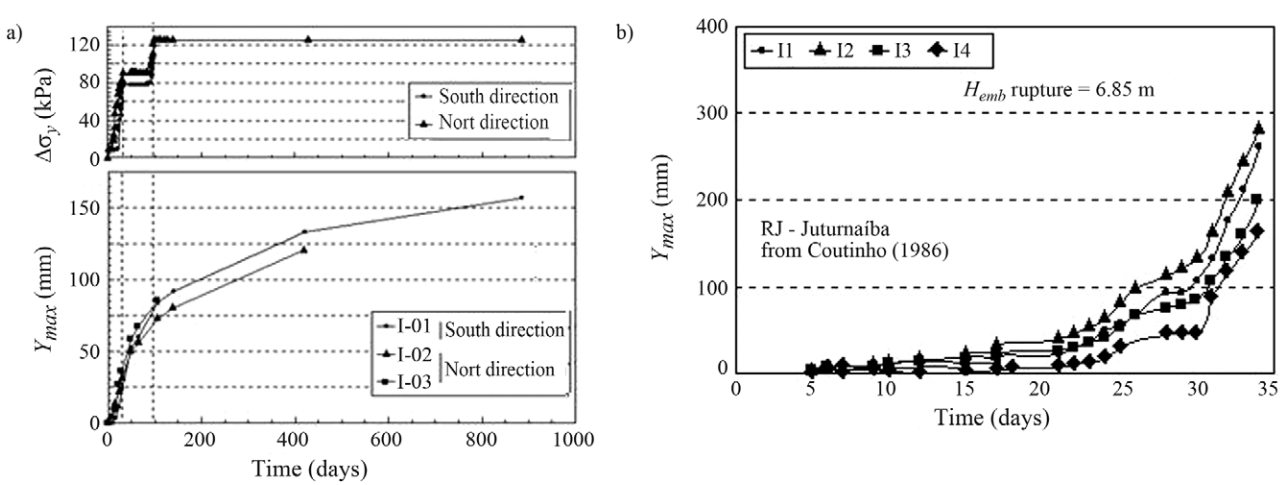


Figure 20 - Maximum horizontal displacements through time: (a) Juturnaíba trial embankment (Coutinho 1986); (b) access embankments of the Jitituba River – up to 885 days (Cavalcante *et al.* 2003).

ter construction, thus indicating the stabilization condition. The maximum horizontal displacement measured just after the end of construction (140 days) was on the order of 80-97 mm, and in the long term (885 days), on the order of 155 mm. In Juturnaíba Dam construction was observed convergent behavior and stabilization condition. Inclinometer I-3 showed values of Y_{max} on the order of 180 mm in 650 days (H_{emb} = 10.70 m) (Lucena 1997; Cavalcante *et al.* 2003).

The tendency observed in the Juturnaíba trial embankment is clearly divergent, particularly after 30 days (H_{emb} = 5.60 m; FS = 1.31), with inclinometer I-3 showing a maximum stable horizontal displacement value on the order of 100 mm, and the last reading corresponding to H_{emb} just before failure, with Y_{max} values on the order of 270 mm. The Sarapuí trial embankment also presented divergent behavior, being more evident after 25 days (H_{emb} = 2.5 m). Inclinometers I-2 and I-4 showed values limit of Y_{max} on the order of 100 mm. In the failure (H_{emb} = 2.8 m), the value of Y_{max} was on the order of 300 mm (Coutinho 1986; Cavalcante *et al.* 2003).

(b) Figure 21 presents the maximum horizontal displacement normalized as a function of the thickness of the clay layer (Y_{max}/D) *vs.* time for the access embankments of the Jitituba River Bridge, and for the Juturnaíba trial embankment. The tendency observed is in agreement with what is proposed, and with the design conditions of each embankment. The convergent behavior in access embankments of the Jitituba River bridge becomes more evident after 140 days, with Y_{max}/D values of 0.6% (H_{emb} = 4.8 m), and 0.9% (H_{emb} = 7.0 m) after 170 days. The Juturnaíba Dam construction showed convergent behavior (stable) with higher values in inclinometers I-1, I-3 and I-4, with Y_{max}/D values of 1.8%, 3.4% and 3.6% for H_{emb} of 10.7 m (Lucena 1997; Coutinho *et al.* 1994; Cavalcante *et al.* 2003).

The divergent behavior of the Juturnaíba trial embankment becomes more evident after 30 days (H_{emb} = 5.60 m; FS = 1.31), with inclinometer I-3 showing Y_{max}/D values of around 1.22% (limit of stability), and for the last reading corresponding to Hemb just before failure, the Y_{max}/D value is 2.75% (beginning to be unstable). The Sarapuí trial embankment showed divergent behavior (inclinometers I-2 and I-4), with values limit of Y_{max}/D of 0.7% and 0.9% (H_{emb} = 2.50 m), and Y_{max}/D values of 1.5 and 1.7% for H_{emb} of 2.8 m (unstable).

(c) The divergent and convergent behavior of the Y_{max}/D curve *vs.* time directly relates to the velocity of the horizontal deformation. Cavalcante *et al.* (2003) verified the possibility of evaluating the unstable or stable situation of the foundation soil through the velocity of the horizontal deformation. Figure 22a shows the results of the velocity of maximum horizontal displacement normalized $(Y_{max}/D)/\Delta t$ *vs.* time for a stable embankment (the access embankments of the Jitituba River bridge), presenting maximum values on the order of 0.024%/day. Figure 22b shows the results of $(Y_{max}/D)/\Delta t$ with time for an embankment induced to rupture (Juturnaíba trial embankment). It is observed that after 30 days, a large increment occurs in the rate of variation of the $(Y_{max}/D)/\Delta t$ value, showing a stable limit value of around 0.20%/day (H_{emb} = 5.6 m), with the last reading corresponding to H_{emb} just before failure, the $(Y_{max}/D)/\Delta t$ value is 0.5%/day. Coutinho (1986) and Cavalcante *et al.* (2003) found similar divergent behavior results in the Sarapuí trial embankment. After 25 days, a stable limit value around 0.20%/day (H_{emb} = 2.5 m) was shown, and for the failure (H_{emb} = 2.8 m), the $(Y_{max}/D)/\Delta t$ value is 0.5%/day. Lucena (1997) and Coutinho *et al.* (1994) showed $(Y_{max}/D)/\Delta t$ results for the Juturnaíba Dam, with stable behavior and values of around 0.03%/day. The maximum $(Y_{max}/D)/\Delta t$ value observed at the recommended limit for stable condition (SM = 1.30) was about 10 times greater in the embankment induced to failure, in comparison with the embankments constructed to be stable.

5.2.2. Angular distortion with time

Analysis of angular distortion was carried out considering two proposals: (a) maximum angular distortions *vs.*

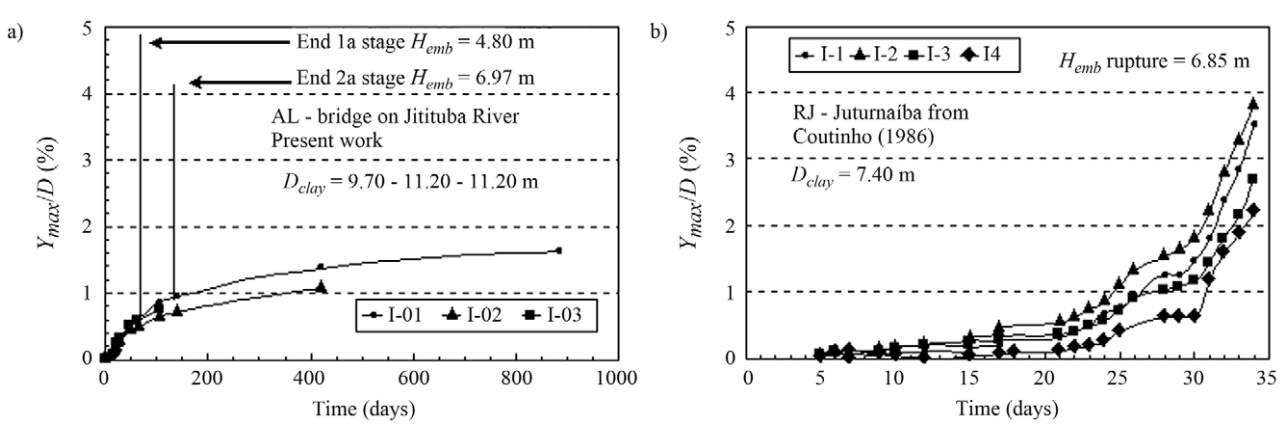


Figure 21 - Relation Y_{max}/D (horizontal maximum displacements / thickness of the clay layer) with time - a) stable embankments; b) embankments induced to the rupture (Cavalcante 2001; Cavalcante *et al.* 2003).

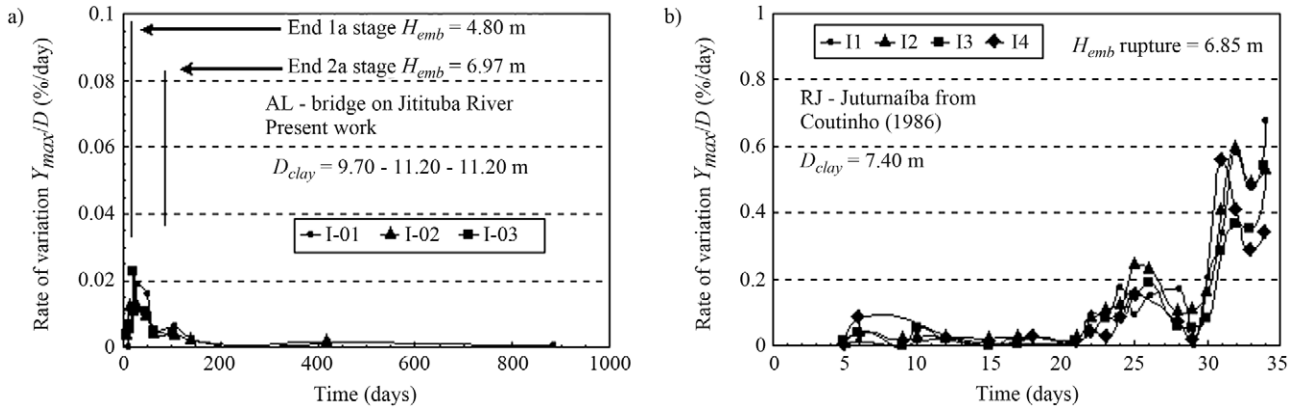


Figure 22 - Rate of variation of the relation Y_{max}/D (horizontal maximum displacements / thickness of the clay layer) through time: a) Stable embankments – access embankments of the Jitituba River Bridge; b) embankments induced to rupture - Juturnaíba trial embankment (Cavalcante 2001; Cavalcante *et al.* 2003).

time and maximum construction end values; and (b) velocity of maximum angular distortion (vd) *vs.* time.

(a) Figure 23 presents the results of maximum angular distortions (10^{-2} radians or %) through time, for the access embankments of the Jitituba River Bridge. The convergent stable behavior is similar to that observed in the analysis of the maximum horizontal displacement. The curve showed a stable maximum value of 2% for just after the end of construction (140 days), and in the long term 3.5% for 885 days.

The tendency observed in the Juturnaíba trial embankment is clearly divergent behavior (Fig. 24). Coutinho (1986), in the analysis of the behavior of the Juturnaíba and Sarapuí trial embankments, found the maximum angular distortions to show divergent behavior for values in the rupture, measuring around 12 and 17%, respectively. The embankments remained stable during construction, with the maximum angular distortion value lower than 4% in the Juturnaíba trial embankment, and 3% in the Sarapuí trial embankment.

(b) Figure 25 presents the value for the maximum rate of angular distortion (vd) *vs.* time for the access embankments of the Jitituba River Bridge. The maximum value observed was 0.07%/day during the initial construction period, showing a much lower value than the vd < 0.5%/day limit proposed by Almeida *et al.* (2001) for a stable situation (Table 3).

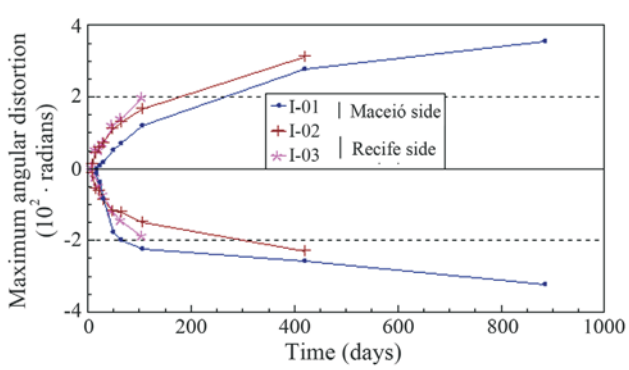


Figure 23 - Maximum angular distortion through time, I-01, I-02, I-03, access embankments of the Jitituba River Bridge (Cavalcante *et al.* 2003).

Table 4 shows the vd results obtained for the Juturnaíba trial embankment. The values were relatively small (vd < 0.6%/day), and the embankment remained stable to an embankment height of 5.60 m (SM = 1.31). However, in considering Table 3, inclinometer I-2 was practically within the stable limit when beginning to show the alert signal. When the embankment height was increased from 5.60 to 6.10 m, a significant increase in vd occurred for all inclinometers, particularly for I-2, showing extremely high values (H_{emb} = 6.40 m), indicating imminent collapse. The failure occurred with H_{emb} = 6.85 m.

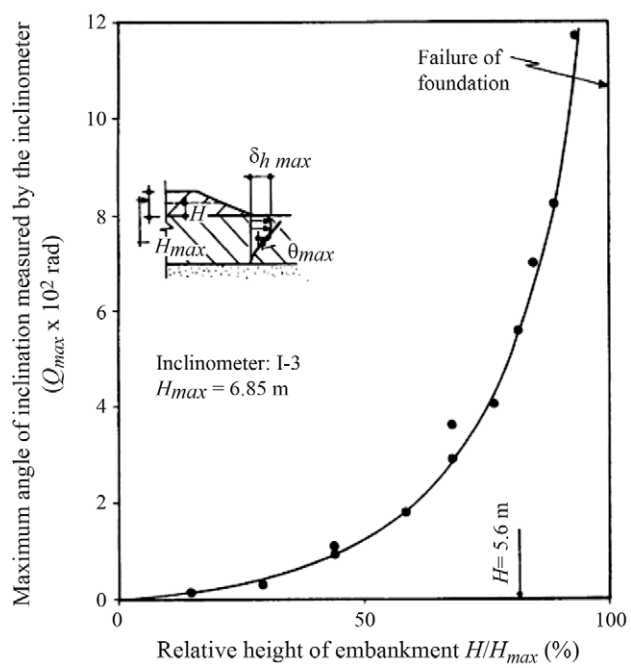


Figure 24 - Maximum angular distortion *vs.* the height of embankment,- Juturnaíba trial embankment (Coutinho 1986).

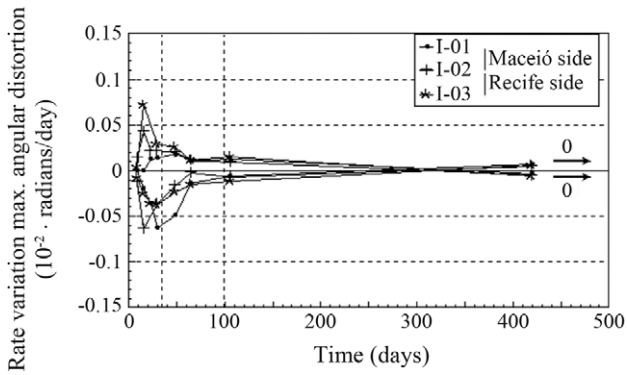


Figure 25 - Rate of variation of the maximum angular distortion with time, access embankments of the Jitituba River Bridge (Cavalcante *et al.* 2003).

5.2.3 Relationship between the variation of vertical volume, and the variation of horizontal volume

Sandroni *et al.* (2004) presented a stability control proposal with analysis and discussion on this topic. The proposed methodology (Table. 3) is based on the displaced vertical volume (Vv) and the displaced horizontal volume (Vh) involving two relations: (a) Vv/Vh or dVv/dVh *vs.* time (t) or embankment height (H_{emb}); and (b) Vh *vs.* H_{emb}. In the methodology, it is recommended that analysis be given to the behavior of all the relations proposed, in order to be able to verify the condition of stability. Johnston (1973) developed earlier studies and observed a range of 3.5-4.2 < Vv/Vh ≤ 20, corresponding to partial drainage behavior, without failure of embankments; in the embankment which presented failure, Vv/Vh was in the range of 2.4-1.8 (Table 3).

(a) Figure 26a shows the results of Vv/Vh and dVv/dVh *vs.* H_{emb} obtained from the Juturnaíba trial embankment. The values for Vv/Vh were higher that 14 until H_{emb} = 3.25 m, decreasing to 7 between H_{emb} = 3.25 and 5.6.m (SM = 1.31), presenting the stability limit. The values between H_{emb} = 5.6 and 6.4 m decrease to 4.5, presenting a signal of failure. In the case of dVv/dVh, values were higher than 14 in the beginning of loading, decreasing to 4.0 when the embankment height was increased from 3.25 m to 5.60 m. Finally, at H_{emb} = 6.4 m, the results of this relation equaled 2.0, presenting a signal of failure.

Table 4 - Angular distortion rate, vd in the Juturnaíba trial embankment (Coutinho 1986; Almeida *et al.* 2001).

H (m)	I-1 (%/day)	I-2 (%/day)	I-3 (%/day)	I-4 (%/day)	I-5 (%/day)
4.65	0.3	0.6	0.5	0.5	0.5
5.60	0.3	0.6	0.4	0.4	0.4
6.10	1.3	2.3	1.3	1.4	1.6
6.40	2.7	2.5	3.1	1.4	2.4

In the Sarapuí trial embankment, the Vv/Vh results were higher than 7.0, until H_{emb} = 2.5 m. An abrupt decrease to values under 3.0 was observed. The dVv/dVh values ranged from 1.0 to 2.0 for H_{emb} = 2.5 m, indicating that the beginning of the rupture that occurred at the 2.5 m embankment height (Sandroni *et al.* 2004).

This proposal was applied to the access embankments of the Jitituba River Bridge (Cavalcante *et al.* 2003). The ΔVv/ΔVh results ranged from 8.4 to 28.0 (Southern direction) and 4.2 to 28.0 (Northern direction), which in general was classified as a stable condition, and only in some intervals, during the first phase of the construction, did the embankment present a situation classified as medium/alert.

(b) Another unstable condition is when the H_{emb} *vs.* Vh relation shows divergent behavior, with a significant increase in inclination. Figure 26b shows the results of H_{emb} *vs.* Vh obtained from the Juturnaíba trial embankment. Above the embankment height of 5.60 m (FS = 1.3), the behavior changed significantly, showing results that correspond to the beginning of a possible failure process, which occurred shortly afterwards, with H_{emb} = 6.85 m.

5.2.4 Relationship between horizontal displacements and safety factor (or embankment height)

For use as a stability control proposal, Bourges & Mieussens (1979) related the maximum horizontal dis-

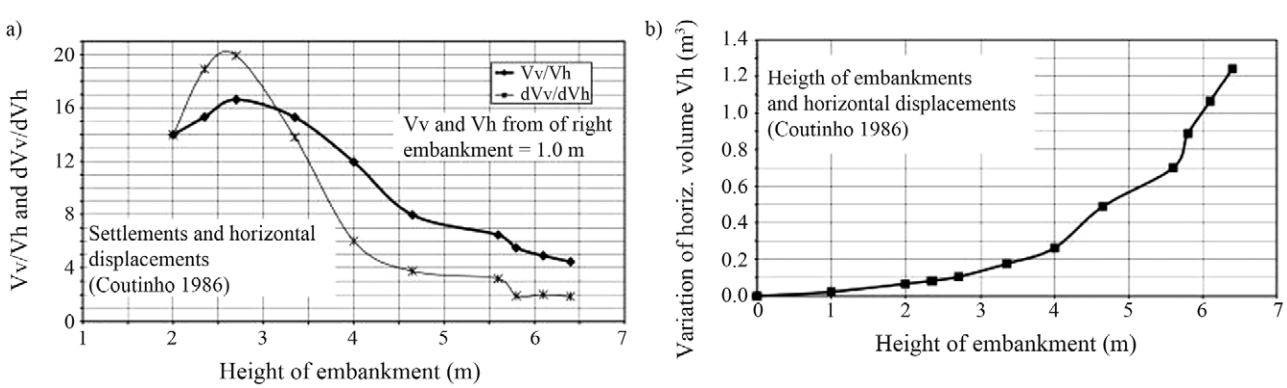


Figure 26 - Juturnaíba trial embankment: a) failure around 35 days; b) evolution of horizontal volume with the height of embankment (Coutinho 1980; Sandroni *et al.* 2004).

placement corresponding to the end of construction, and normalized it in function of the thickness of the clay layer (Y_{max}/D) *vs.* embankment height (H_{emb}), or the Minimal Safety Factor (SM).

Coutinho (1986), in the stability control for the Juturnaíba trial embankment, discussed the use of the Y_{max}/D *vs.* relative height of embankment, H/H_{max} (%). Figure 27 presents the observed results, where divergent behavior can be seen, including a sharp increase after H_{emb} = 5.6 m, and a maximum value of 3% (H_{emb} = 6.4 m) just before failure. In this case, it is recommended that the relation be less than 1.5% in order to have a stable condition. Ortigão (1980) and Ortigão *et al.* (1983) in the Sarapuí trial embankment, found Y_{max}/H_{emb} results of 2.7% for an embankment height of 2.8 m (failure condition).

Cavalcante *et al.* (2003) presented and discussed Y_{max}/D *vs.* SM results obtained for a number of Brazilian embankments on soft clays (Fig. 28). It can be observed that the tendency is for SM values to decrease when the Y_{max}/D values increase. For the stable Jitituba embankment, the maximum values for the Y_{max}/D relation for the 1st and 2nd stages of construction were 0.54% and 0.32%, respectively. Lucena (1997) and Coutinho *et al.* (1994) in the Juturnaíba Dam construction, encountered Y_{max}/H_{emb} results of 1.93%, 0.90%, and 2.09% (inclinometers I-1, I-2 and I3) for an embankment height of 6.0 m (1st stage), demonstrating marginally stable behavior (Fig. 28).

Considering these cases, Cavalcante *et al.* (2003) proposed values for stability control during the construction phase: (a) $Y_{max}/D > 1.8\%$ indicates the proximity of rupture situations (SM ~ 1.0); (b) Y_{max}/D = 1.0% is generally adopted in practical application as the safety factor (SM ~ 1.5); (c) $Y_{max}/D < 0.8\%$ indicates limit of horizontal displacement.

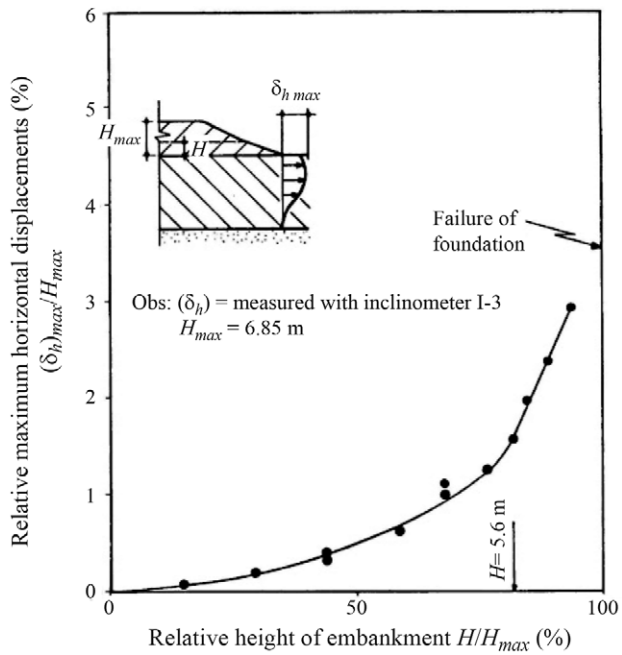


Figure 27 - Relation between the horizontal displacement and the relative height of the embankment, I-3 (Coutinho 1986).

5.2.5. Summary of the stability control (horizontal displacements) results

Table 5 presents a summary of the results obtained in the analysis of stability control from horizontal displacements using all of the proposals from Table 3. Two different conditions were considered: (a) a trial embankment induced to rupture: Juturnaíba (Case Study 1) and Sarapuí (Case Study 5); (b) embankments designed to be stable: the Juturnaíba Dam (Case Study 2) and the access embankments of the Jitituba River Bridge (Case Study 3). The be-

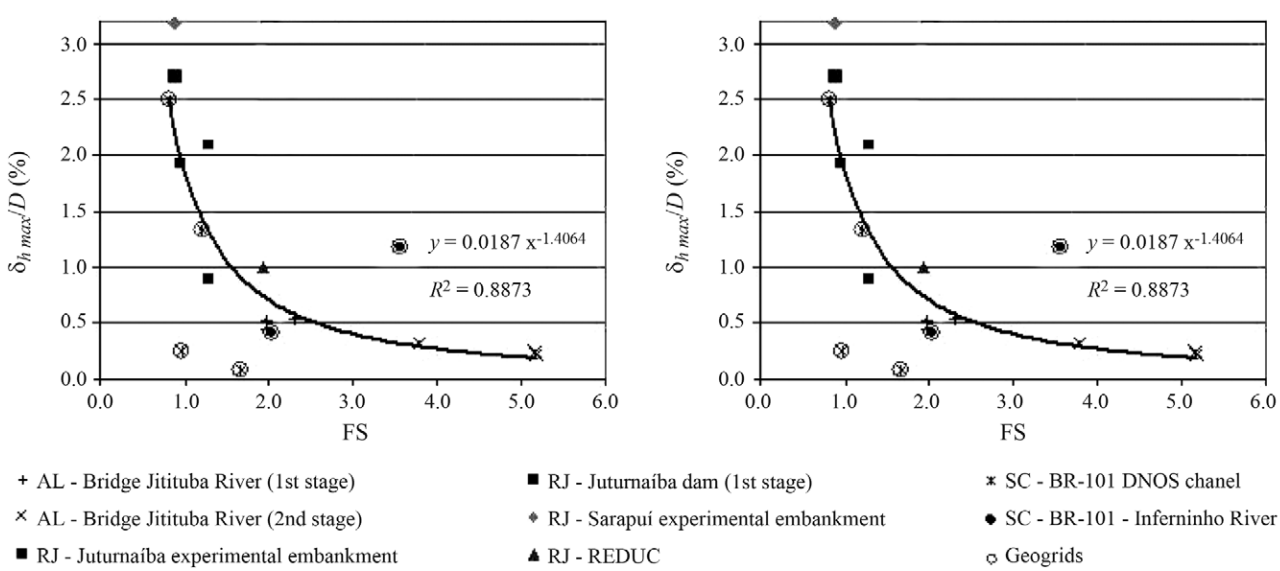


Figure 28 - Y_{max}/D *vs.* FS (or SM relation): Brazilian embankments (Cavalcante *et al.* 2003).

Table 5 - Summary of the stability control results from horizontal displacements proposals.

Analysis methods		Results / classification			
		(Case Study 1) Sarapuí trial embankment	(Case Study 2) Juturnaíba trial embankment	(Case Study 3) Juturnaíba Dam	(Case Study 4) Access embankments of the Jitituba River Bridge
Horizontal displacement *vs.* time	Maximum value normalized in function of the thickness of the clay level (Y_{max}/D) *vs.* time	*I2 and I4 - Divergent* H_{emb} = 2.5 m - 0.7-0.9% Safe Limit H_{emb} = 2.8 m - 1.5-1.7% Unstable	*I3 - Divergent* H_{emb} = 5.6 m - 1.22% Safe Limit H_{emb} = 6.4 m - 2.75% Beginning to be Unstable	*I1, I3 and I4 Convergent* H_{emb} = 10.7 m 1.8; 3.4; 3.6% Stable	*Convergent* H_{emb} = 4.8 m - 0.6% H_{emb} = 6.97 m - 0.9% Stable
	Velocity of the maximum value normalized ($\Delta Y_{max}/D$) *vs.* time (Cavalcante 2001; Cavalcante *et al.* 2003)	*I3 and I4* H_{emb} = 2.5 m - 0.20%/day Safe Limit H_{emb} = 2.8 m - 0.5%/day Unstable	*I1, I2, I3 and I4* H_{emb} = 5.6 m - 0.25%/day Safe Limit H_{emb} = 6.4 m - 0.5%/day Beginning to be Unstable	0.030% / day Stable	0,024% / day Stable
Angular distortion *vs.* time	Maximum value (vd) *vs.* time Construction end value (Ortigão 1980; Coutinho 1986; Cavalcante 2001; Cavalcante *et al.* 2003)	*I3 and I4 - Divergent* 3.0% - Safe Limit 12% - Unstable	*I3 - Divergent* H_{emb} = 5.6 m - *3%* Safe Limit H_{emb} = 6.4 m -12% Beginning to be Unstable	-	*Convergent behavior* Stable < 2%
	Velocity of the maximum value (vd) *vs.* time (Almeida *et al.* 2001)	H_{emb} = 2.5 m -1.00%/day Safe Limit H_{emb} = 2.8 m - 3.5%/day Unstable	H_{emb} = 5.6 m - 0.4%/day Safe Limit H_{emb} = 6.4 m- 3.1%/day Beginning to be Unstable	-	0.07% / day Stable
Displaced vertical volume *vs.* Displaced horizontal volume	(Vv/Vh) or (ΔVv/ΔVh) *vs.* H_{emb} or time (Sandroni & Lacerda 2001; Sandroni *et al.* 2004	ΔVv/ΔVh: H_{emb} = 2.5 m - 7.0 - Safe Limit H_{emb} = 2.8 m = 1.0- Unstable Vv/Vh: H_{emb} = 2.5 m - 7.0 - Safe Limit H_{emb} = 2.8 m = 3.0- Unstable	ΔVv/ΔVh: H_{emb} = 5.6 m - 4.0 - Safe Limit H_{emb} = 6.4 m = 2.0 - Beginning to be Unstable Vv/Vh: H_{emb} = 5.6 m - 7.0 - Safe Limit H_{emb} = 6.4 m = 4.0 - Beginning to be Unstable	-	Southern direction (8.4 to 28.0) Stable Northern direction (4.2 to 28) Medium to Stable
	Vh *vs.* H_{emb}	-	H_{emb} = 5.6 m - 0.6 Safe Limit H_{emb} = 6.4 m - 1.25 Beginning to be Unstable	-	-
Y_{max}/D x *SM or* Y_{max}/D x H_{emb} (Bourges & Mieussens 1979; Coutinho,1980 Cavalcante 2001; Cavalcante *et al.* 2003)		*I3 and I4 - Divergent* H_{emb} = 2.8 m - 3.0% Unstable	*I3 - Divergent* H_{emb} = 5.6 m - 1.5% Safe Limit H_{emb} = 6.4 m - 3.0% Beginning to be Unstable	*I1, I2 and I3* H_{emb} = 6.0 m (1st Stage) 1.93%; 0.90%; 2.09% Marginally Stable	0.44 to 0.54% Stable (minimum horizontal displacements)

havior tendency, limit values for stable condition (safe limit) and values on unstable condition were presented. The results in general agreed with the stability control proposals for horizontal displacements, showing potential for use in practical work, depending on each problem. It was seen that it is important for more than one proposal be employed, in order to have additional confidence in decisions, and that all behavior has to be analyzed, not just the measurement values. The trial embankments induced to rupture studied in this paper shown that is possible to predict the rupture, with the Juturnaíba embankment showing more anticipating and clearly change from stable to unstable condition. The joint results of stability analysis and stability control show the importance of having SM > 1.3 to guarantee adequate behavior and security.

Ladd (1991) presents and discusses the use and interpretation of displacement data from the field for use in stability control. It is pointed out that this requires experience and judgment, along with the use of different graphs for each problem. The authors recommend the use of many of the graphs / proposals shown in this paper. It must also be remembered that the behavior of a soft clay foundation may be brittle or ductile. In soils with brittle behavior, such as sensitive clays, the rupture can be abrupt and difficult to anticipate. In soils with ductile behavior, the process tends to be more gradual, with greater possibly for advanced warning.

6. Final Comments and Conclusions

This paper presents the results of stability analysis and stability control, with emphases on studies carried out by the Geotechnical Group (GEGEP) of the Federal University of Pernambuco, Brazil. Five Brazilian cases are presented: the Juturnaíba trial embankments and Juturnaíba Dam construction, located in Rio de Janeiro; the access embankments of the Jitituba River Bridge in Alagoas; the failure of an embankment alongside highway BR-101-PE, located in Recife, Pernambuco; and the Sarapuí trial embankment, located in Rio de Janeiro.

Some of the approaches presented and discussed here were used for evaluation of mobilized undrained shear strength S_u in an embankment constructed in one stage. In the total stability analysis, cases of failure in Brazilian embankments on soft clays (the exception being the Juturnaíba trial embankment) show the need for application of the Bjerrum (1973) correction factor to the field vane test measurements for undrained strength. The presence of organic soil layers in the Juturnaíba foundation, combined with strong drainage and deformation/increases from effective stress during the construction, seem to be a possible explanation for the "different" behavior.

Effective stress stability analysis performed on the Juturnaíba trial embankment presented satisfactory results, considering normally consolidated effective stress parameters of strength, particularly when cracking of the embankment was considered to induce failure.

Stability control is one of the important steps in the design and construction of embankments on soft soils, and can be carried out through the measurement of displacements, deformations or pore pressures. Proposals were presented and analyzed, particularly for horizontal displacements, showing potential for use in practical work, depending on each problem. Due to the limits of each proposal, and many variables involved in the process, it is recommended that more than one proposal be used to obtain more confidence in decisions, and that all behavior be analyzed, not just the value of measures. The Juturnaíba trial embankments induced to rupture showed that would be possible to avoid the rupture, with reasonable anticipating and clearly change from stable to unstable condition.

The joint results of stability analysis and stability control show the importance of having SM > 1.3 to guarantee adequate behavior and security.

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References

Almeida, M.S.S.; Oliveira, J.R.M.S. & Spotti, A.P. (2001) Previsão e desempenho de aterro sobre solos moles: Estabilidade, recalques e análises numéricas. In: Encontro das Argilas Moles Brasileiras. COPPE/UFRJ e ABMS, Rio de Janeiro e São Paulo, pp. 166-191.

Almeida, M.S.S. & Marques, M.E.S. (2010) Aterros Sobre Solos Moles. Editora Oficina de Textos, São Paulo, 254 pp.

Almeida, M.S.S.; Marques, M.E.S. & Lima, B.T. (2010) Overview of Brazilian construction practice over soft soils. Márcio Almeida (ed) New Techniques on Soft Soils. Oficina de Textos, São Paulo, pp. 247-268.

Azzouz, A.S.; Baligh, M. & Ladd, C. (1983) Corrected field vane strength for embankment design. ASCE JGE, v. 109:5, p. 730-734.

Bello, M.I.M.C.V. (2004) Estudo de Ruptura em Aterros Sobre Solos Moles – Aterro do Galpão localizado na Br-101-PE. M.Sc. Thesis, Universidade Federal de Pernambuco, Recife, 207 pp. (in Portuguese).

Bello, M.I.M.C.V.; Coutinho, R.Q. & Gusmão, A.D. (2006) Estudo de ruptura de um aterro sobre solos moles localizado em Recife, Pernambuco. Proc. XIII COBRAMSEG, Curitiba. v. 4, pp. 2115-2120.

Bjerrum, L. (1973) Problems of soil mechanics and construction of soft clays and structurally unstable soils. 8th ICSMFE, Moscow, v. 3, pp. 111-159.

Bourges, F. & Miussens, C. (1979) Déplacement latéraux à proximité des remblais sur sols compressibles – Méthod de prévision. Bull. Liaison Labo. P. et Ch., n. 101, p. 73-100.

Cavalcante, S.P. (2001) Análise de Comportamento de Aterros sobre Solos Moles – Aterros de Encontro da Ponte sobre o Rio Jitituba- AL. M.Sc. Thesis, Universidade Federal de Pernambuco, Recife, 232 pp.

Cavalcante, S.P.; Coutinho, R.Q. & Gusmão, A.D. (2003) Evaluation and control of stability in field of embank on soft soils. Proc. XII PCSMGE, Cambridge, pp. 2649-2660.

Cavalcante, S.P.P.; Coutinho, R.Q. & Gusmão, A.D. (2004) Analysis of behavior of embankments on soft soils geotechnical investigations and instrumental access embankments of the Jitituba River Bridge. Prof. 15th ICCHGE, New York, paper 2.83, pp. 1-9.

Coutinho, R.Q. (1986) Aterro Experimental Instrumentado Levado à Ruptura Sobre Solos Orgânicos-Argilas Moles Barragem de Juturnaíba. D.Sc. Thesis, COPPE/Universidade Federal do Rio de Janeiro, Rio de Janeiro, 634 pp.

Coutinho, R.Q. (2006) Characterization and Engineering Properties of Recife Soft Clays – Brazil. Charact. and Eng. Properties of Natural Soils, Singapor. v. 3, pp. 2049-2100.

Coutinho, R.Q.; Oliveira, J.T.R. & Oliveira, A.T.J. (1998) Geotechnical site characterization of Recife soft clays. Proc. ISC'98 - I International Symposium on Site Characterization, ISSMGE/ASCE/CGS, Balkema, Atlanta, v. 2, pp. 1001-1006.

Coutinho, R.Q.; Almeida, M.S.S. & Borges, J.B. (1994) Analysis of the Juturnaíba embankment dam built on soft clay. ASCE, Geot. Spec. Public. N. 40, v. 1, p. 348-363.

Coutinho, R.Q. & Lacerda, W.A. (1987) Characterization-consolidation of Juturnaíba organic clays. Proc. ISGESS, Mexico City, México, v. 1, pp. 17-24.

Coutinho, R.Q. & Lacerda, W.A. (1989) Strength characteristics of Juturnaíba organic clays. Proc. 12th ICSMFE, Rio de Janeiro, v. 3, pp. 1731-1734.

Coutinho, R.Q.; Oliveira, A.T.J. & Oliveira, J.T.R. (2000) Palheta: Experiência, Tradição e Inovação. Conferência IV Seminário de Engenharia de Fundação Especiais e Geotecnia / Seminário Brasileiro de Investigação de Campo, São Paulo, v. 3. pp. 53-80.

Coutinho R.Q. & Bello M.I.M.C.V. (2005) Geotecnia do Nordeste – Aterro sobre Solos Moles. ABMS/ Núcleo Nordeste, Recife, Editora Universitária / UFPE, v. 3, pp. 111-153.

Coutinho, R.Q. & Bello, M.I.M.C.V. (2007) The empirical methods for stability analysis in total tensions for embankments on soft soils. Proc. XIII Conferencia Panamericana de Mecánica de Suelos e Ingeniería Geotécnica, Isla Margarita, pp. 868-873.

Coutinho, R.Q. & Bello, M.I.M.C.V. (2010) Analysis and control of the stability of embankments on soft soil: Juturnaíba and other experiences in Brazil. M. Almeida (ed) New Techniques on Soft Soils. Oficina de Textos, São Paulo, pp. 247-268.

Coutinho, R.Q. & Bello, M.I.M.C.V. (2011) Monitoring and performance of embankments on soft soil: Juturnaíba trial embankment and other experiences in Brazil. Soils and Rocks, v. 34:4, p. 353-378.

Johnston, I.W. (1973) Discussion – Session 4. In: Field Instrumentation in Geotechnical Engineering, Halsted Press Book, John Wiley, New York, pp. 700-702.

Kawamura, K. (1985) Methodology for landslide prediction. Proc. XI ICSMFE, San Francisco, v. 3, pp. 1155-1158.

Ladd, C.C. (1991) Stability evaluation during staged construction. JGE, ASCE, v. 117:4 p. 540-615.

Leroueil, S & Rowe, R.K. (2000) Embankments Over Soft Soil and Peat. Geotechnical and Geoenviromental Engineering Handbook. R.K. Rowe (ed) Kluwer Academic Publishers, Norwell, USA, pp. 463-498.

Lucena, M.H.L. (1997) Análise do Comportamento da Fundação da Barragem de Juturnaíba – Techos III-2 e V. M.Sc. Thesis, Universidade Federal University de Pernambuco, Recife, 219 pp. (in Portuguese).

Magnani de Oliveira, H.; Almeida, M.S.S & Ehrlich, M. (2010) Reinforced test embankments on Florianopolis very soft clay. In: Almeida, M.S.S. (ed) New Techniques on Soft Soils. Oficina de Textos, São Paulo, pp. 65-76.

Magnani de Oliveira, H. (2006) Comportamento de Aterros Reforçados sobre Solos Moles Levados à Ruptura. D. Sc. Thesis, COPPE/ Universidade Federal do Rio de Janeiro, Rio de Janeiro, 495 pp. (in Portuguese).

Mesri, G. (1975) Discussion: New design procedure for stability of soft clays. ASCE JGE, Division v. 101:GT4, p. 409-412.

NAVFAC DM-7 (1971) Design Manual of Soil Mechanics, Foundations and Earth Structures, Naval Facilities Engineering Command, Washington, D.C.

Ortigão, J.A.R. (1980) Aterro Experimental Levado à Ruptura sobre Argila Cinza do Rio de Janeiro. D.Sc. Thesis, COPPE/ Universidade Federal do Rio de Janeiro, Rio de Janeiro, 715 pp. (in portuguese).

D.Sc. Thesis, COPPE/UFRJ, Rio de Janeiro.

Ortigão, J.A.R.; Werneck, M.L.G. & Lacerda, W.A. (1983) Embankment failure on clay near Rio de Janeiro. Journal of Geotechnical Engineering ASCE, v. 109:11, p. 1460-1479.

Pillot, G. & Moreau, M. (1973) La Stabitité des Remblais sur sol Mous – Abaqus de Calcul. Editions Eyrolles, Paris, 151 pp.

Pinto, C.S. (1966) Capacidade de carga de argilas com coesão linearmente crescente com a profundidade. Jornal de Solos, v. 3:1, p. 21-44.

Sandroni, S.S. (1993) Sobre o uso do ensaio de palheta *in situ* em aterros sobre argilas moles. Solos & Rochas, v. 16:3, pp. 207-213.

Sandroni, S.S. (2006) Sobre a prática brasileira de projeto geotécnico de aterros rodoviários em terrenos com solos muito moles. In: Congresso Luso-Brasileiro, Curitiba, pp. 507-512.

Sandroni, S.S. & Lacerda, W. (2001) Discussão sobre Controle de Estabilidade em Aterros sobre Solos Moles. Encontro Propriedades das Argilas Moles Naturais Brasileiras, COPPE/UFRJ-ABMS, Rio de Janeiro e São Paulo, pp. 245-257.

Sandroni, S.; Lacerda, A.W. & Brandt, J.R.T. (2004) Método dos volumes para controle de campo da estabilidade de aterros sobre argilas moles. Solos & Rochas, v. 27:1, p. 37-57.

Skempton, A.W. (1957) Discussion. Proc. Institution of Civil Engineers, v. 7, p. 305-307.

Tavenas, F. & Leroueil, S. (1980) The behavior of embankments on clay foundations. Canadian Geotechnical Journal, v. 17:2, p.236-260.

Tavenas, F.; Trak, B. & Lerouiel, S. (1980) Remarks on the validity of stability analysis. Canadian Geotechnical Journal, v. 17:1, p. 61-73.

Terzaghi, K. (1943) Theoretical Soil Mechanics, Wiley, New York, 510 pp.