Monitoring and Performance of Embankments on Soft Soil: Juturnaíba Trial Embankment and Other Experiences in Brazil

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

Abstract. The accurate performance of instrumentation is fundamental to the adequate use of the results obtained from analyzing the behavior of constructions of embankments on soft soils. When evaluating embankment behavior, the geotechnical instrumentation for pore pressure and vertical and horizontal displacements measurements can be an extremely useful tool, including when there are adjacent structures. This paper presents the results of monitoring and analysis of the performance of embankments on soft soil, carried out by the Geotechnical Group (GEGEP) of the Federal University of Pernambuco. Three Brazilian cases are presented: the Juturnaíba trial embankment and Juturnaíba Dam construction, in Rio de Janeiro; and the access embankments of the Jitituba River Bridge, in Alagoas. The analysis of embankment behavior was performed by using the traditional model (undrained condition during construction) and the Tavenas & Leroueil (1980) model (partially drained condition during construction). The results showed that it is possible to predict embankment behavior. They also showed the complexity of this topic and the importance of monitoring and evaluating each case in accordance with the location and the type of instrument, subsoil and embankment conditions.

Keywords: embankments, soft soils, monitoring, performance.

1. Introduction

The construction of an embankment on soft clay represents an important geotechnical problem and has been studied by various authors. Their papers form a body of experiences for a better understanding of soft soils bearing load increases (*e.g.* Bjerrum 1973; Tavenas & Leroueil 1980; Leroueil & Rowe 2000). In Brazil, important studies have been published by Ortigão (1980), Coutinho (1986), Pinto (1992), Almeida (1996), Massad (1999); Coutinho & Bello (2005), and Magnani de Oliveira (2006). In general, the design of embankments on soft soils should meet the basic requirements of stability against rupture and vertical and horizontal displacements, during and after construction, compatible with its objective. Trial embankments have been used to increase the understanding on the behavior of embankments on soft soils, as well as to support projects for which conventional procedures presented in studies do not seem sufficient for the adequate prediction of their behavior. Instrumentation is a tool for monitoring and evaluating the construction of embankments by measuring pore-pressures, vertical and horizontal displacements, etc.

This paper presents the results of monitoring and analysis of the performance of embankments on soft soil, carried out by the Geotechnical Group (GEGEP) of the Federal University of Pernambuco. Three Brazilian cases are presented: the Juturnaíba trial embankment and Juturnaíba Dam construction, in Rio de Janeiro; and the access embankments of the Jitituba River Bridge, in Alagoas. The topics on analysis and stability control are presented by Coutinho & Bello (2010).

1.1. Behavior of embankments on soft soil

When analyzing the behavior of embankments on clay foundations, it has commonly been assumed that such is perfectly undrained during construction and that drainage and consolidation start only after the end of construction. This approach has been widely used and has generally performed well for conventional designs. Observations in construction sites 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 overpredict pore pressures and lateral displacements. Thus, if one wishes to predict the actual behavior of an embankment on clay, it is essential to have a good knowledge of the mechanical behavior of natural clays and to understand what may happen under an embankment during construction (Tavenas & Leroueil 1980; Leroueil & Rowe 2000).

As in most geotechnical problems, it becomes possible to understand soil response only when the corresponding stress path is known. Under embankments, the effective 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). The pore pressure increase observed during

R.Q. Coutinho, D. Sc., Universidade Federal de Pernambuco, Recife, PE, Brazil. e-mail: rqc@ufpe.br.
M.I.M.C.V. Bello, M. Sc., Universidade Federal de Pernambuco, Recife, PE, Brazil. e-mail: isabelamcvbello@hotmail.com.
Submitted on December 29, 2010; Final Acceptance on December 15, 2011; Discussion open until July 31, 2012.

the first phase of loading under more than 30 embankments ($\overline{B}_1 = \Delta u/\Delta\sigma_v$) is plotted in Fig. 1 as a function of the normalized depth, z/D, with D being the thickness of the clay layer. Two observations were made by Leroueil *et al.* (1978): $\overline{B}_1$ is smaller than predicted when perfectly undrained behavior is assumed; and the $\overline{B}_1$ *vs.* z/D relationship has the shape of a consolidated isochrone, indicating that in these cases there is significant consolidation during the early stages of construction when the soil is overconsolidated.

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. 2a (OCR < 2.5). As a consequence of the rapid consolidation during early stages of construction (very high c_v in the preconsolidation 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 preconsolidation pressure, σ'_p, of the clay. 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 increase in total stress ($\overline{B}_2 = \Delta u/\Delta\sigma_v = 1.0$) during the second phase of loading.

The change in pore pressure generation during construction is thus associated with the soil yielding when the effective stress path reaches the limit state curve. This *in situ* vertical yield stress, (σ'_{vy} or σ'_{vcrit}), has been compared with the preconsolidation pressure measured in conventional 24-h oedometer tests, σ'_{pconv} (Morin *et al.* 1983; Leroueil 1996). The results can be summarized as follows: for overconsolidation ratios (OCRs, estimated on the basis of laboratory tests) between 1.2 and 2, there is good agreement between the two parameters; at lower OCRs, laboratory tests generally slightly underestimate *in situ* values, typically by 10%; for OCRs larger than 2, laboratory tests generally overestimate *in situ* values. The overestimation of σ'_{vy} by laboratory tests in overconsolidated clays with OCR > 2 can be explained by the shape of the limit state curve of natural clays and the fact that the coefficient of earth pressure at rest (Ko) is high in these materials (Fig. 2a) (Leroueil *et al.* 1978).

If the embankment is built to a height that exceeds the corresponding to point A (Fig. 2a) the effective stress path will continue up to F', on the strength envelope of the normally consolidated clay, where there is local failure and then 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. 2b. 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

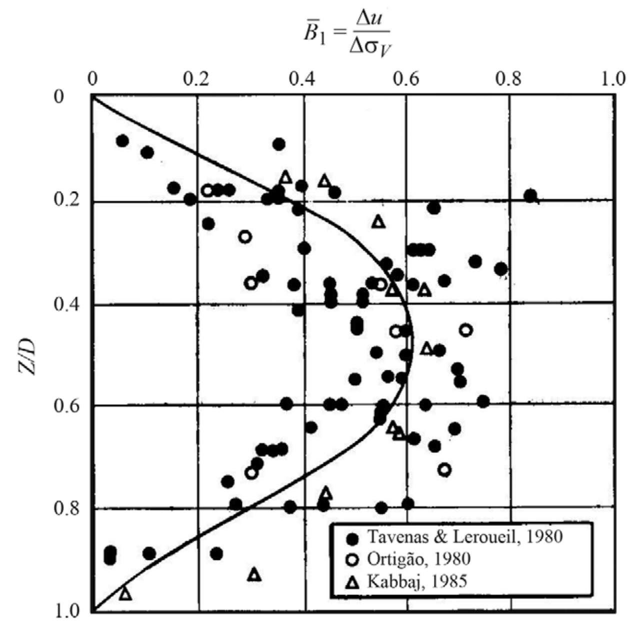


Figure 1 - Compilation of observed excess pore pressures in clay foundation in the first phase of embankment construction (Leroueil & Rowe 2000).

correspond directly to the conventional $\overline{B} = \Delta u/\Delta\sigma_v$ under the entire loading (where $\Delta\sigma_v = \gamma H$). Hence a high value of $\overline{B}_3$ 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 $\overline{B}$ may still be less than unity. The pore pressure generated during the construction of an embankment and the corresponding stress path has a direct influence on settlements and lateral displacements.

As indicated by Folkes & Crooks (1985) and Leroueil & Tavenas (1986) the behavior of an embankment on soft clay is not expected to be unique. It has been observed that there are: some field cases where the behavior is essentially undrained; many cases like those discussed here where there is some yielding after partial dissipation of pore pressure; and there have been some cases in which yielding was not reached during construction. In this latter situation, the pore pressures rapidly decrease after the end of construction.

Isochrones shown are given by equation:

$$\overline{B}_1 = \overline{B}_m\left[1-\left(\frac{z}{\Lambda}-1\right)^2\right], \text{For } \overline{B}_m = 0.6 \qquad (1)$$

where z = the distance from the upper drainage boundary; Λ = the drainage path (Λ = 0.5D) and $\overline{B}_m$ = the maximum pore pressure ratio.

1.2. Cases studied

This paper presents results of monitoring and performance of embankments on soft soil carried out by the

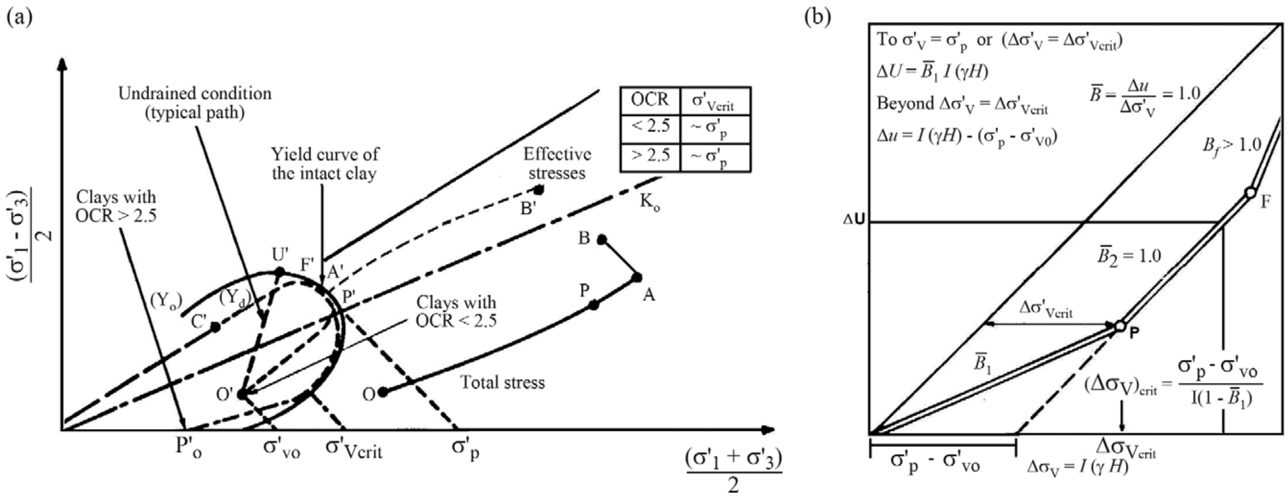


Figure 2 - (a) Total and effective stress paths, and (b) increase of pore pressure under the centerline during stage construction of an embankment - clays with OCR < 2.5 (Coutinho 1986, from Tavenas & Leroueil 1980).

Geotechnical Group (GEGEP) of the Federal University of Pernambuco. The study realized on the Juturnaíba Dam project was used as the basis for this paper.

The Juturnaíba Dam project, an earth-fill structure located in the north of the state of Rio de Janeiro, was built in 1981-1983 (Fig. 3a). The foundation consisted basically of an organic clay deposit about 7.5-8 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 to a depth of 14 m. Visual classification and laboratory tests permitted a subdivision of clayey deposit into six layers, with varying organic and water content, ranging from light-grey silt clay to a brown clayey peat (Fig. 3b).

Because a 1.2 km length of this earth dam was supposed to rest on organic soft clay, geotechnical studies were quite comprehensive, including laboratory and field investigations and the construction of a trial embankment led to failure (first case), which was instrumented as indicated in Fig. 4 (Coutinho 1986, Coutinho & Lacerda 1987; 1989).

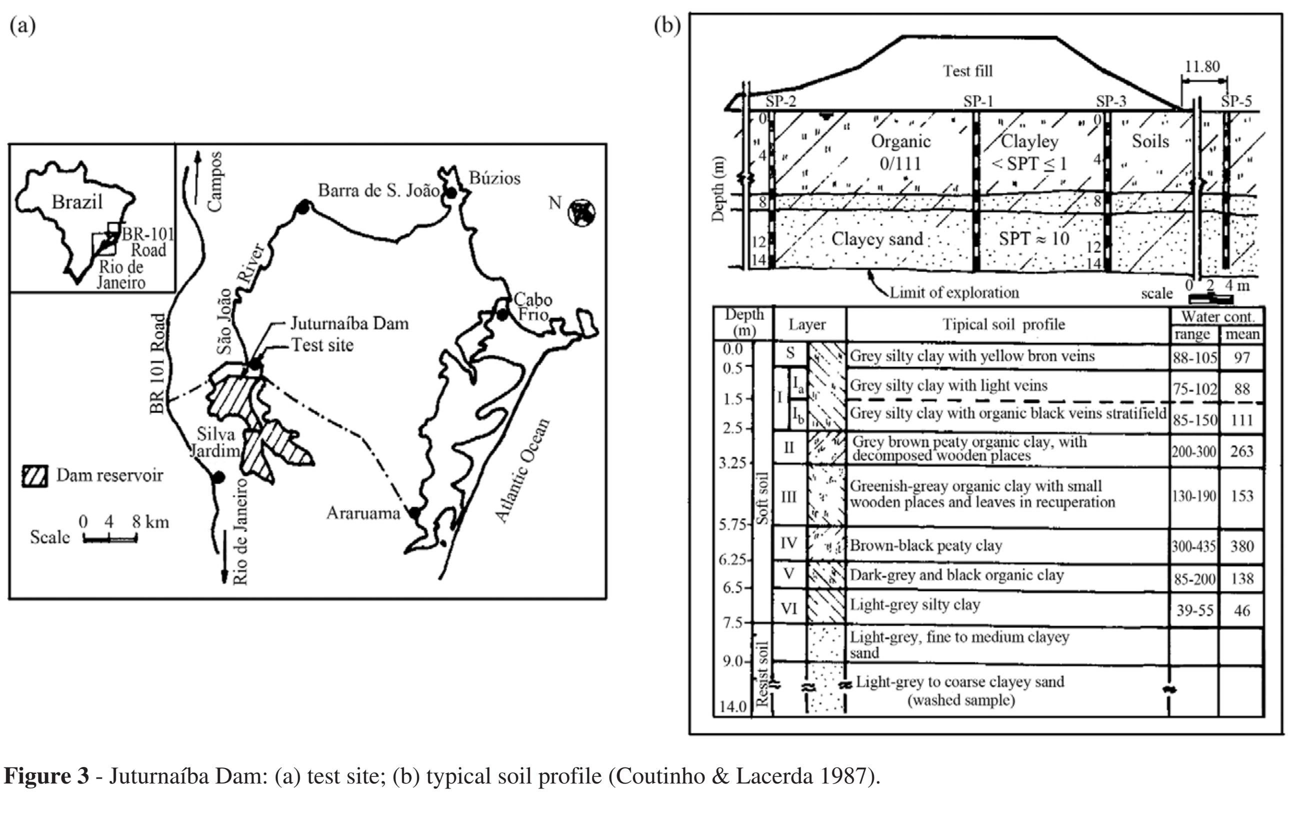


Figure 3 - Juturnaíba Dam: (a) test site; (b) typical soil profile (Coutinho & Lacerda 1987).

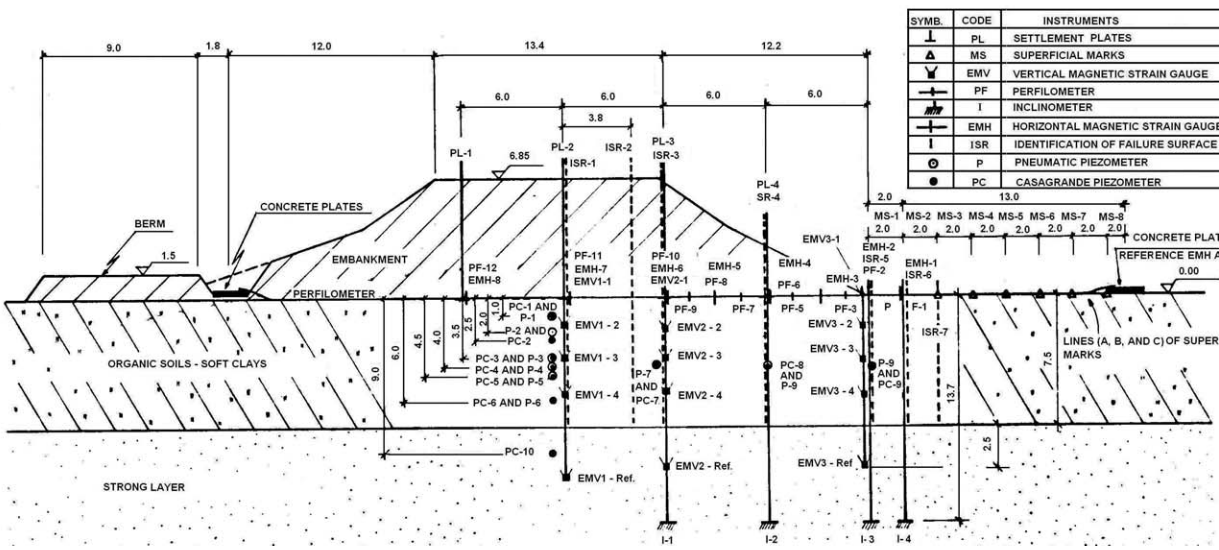


Figure 4 - Instrumentation of the Juturnaíba trial embankment (Coutinho 1986) - First case.

Figure 5 shows results of the water content and Atterberg Limits for six layers of the profile. It can be observed the variation of these results for each layer and consequently in the plasticity index values. Figure 6 presents the results of the overburden effective stress (σ'_{vo}) and preconsolidation pressure (σ'_p) obtained by the oedometric tests. The foundation deposit presents overconsolidation condition with the upper part showing higher values (OCR > 2.5). Figure 7 shows the compressibility parameters, and it can be seen that compression ratio (CR) and swelling ratio (SR) are distinct for each layer. Values of initial void index (e_o), compression index (C_c), and organic content are also different for each layer. The main purposes of these studies were to provide indications on the undrained strength and compressibility in the clay foundation and on methods to control stability during construction.

The design studies indicated that the dam should be built in stages with berms and flat slopes and the 1.2 km length being divided in three sections (II, III.2 and V). Dam monitoring (second case) consisted of placing settlement

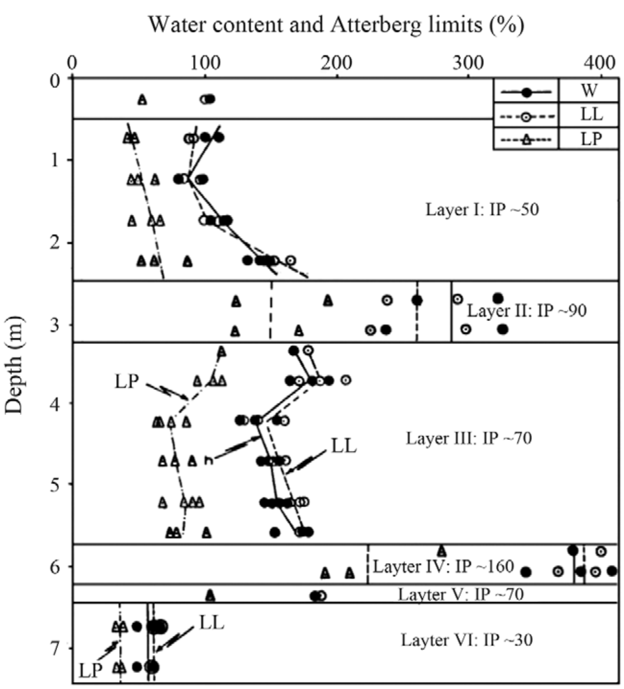


Figure 5 - Water content and Atterberg Limits of the Juturnaíba trial embankment (Coutinho 1986).

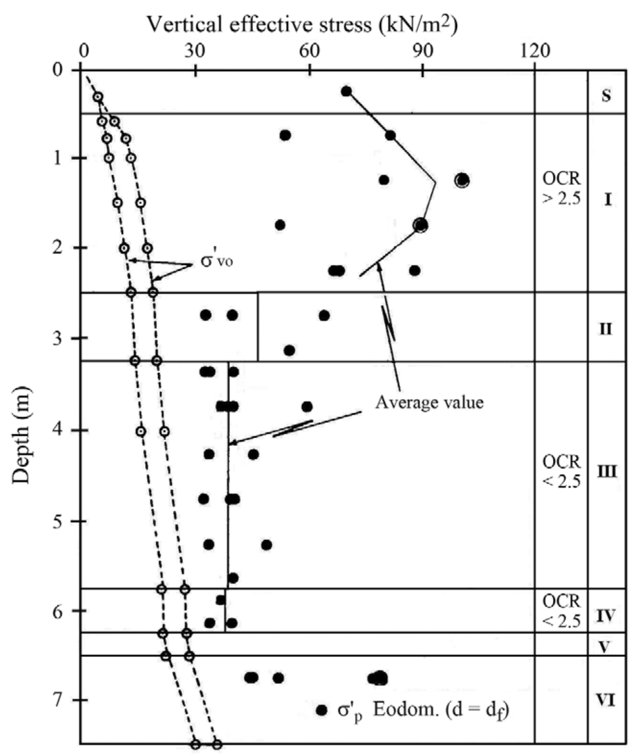


Figure 6 - σ'_{vo}, $\sigma'_{v\,crit}$, σ'_v and σ'_p laboratory values *vs.* depth - center of the Juturnaíba trial embankment (Coutinho 1986).

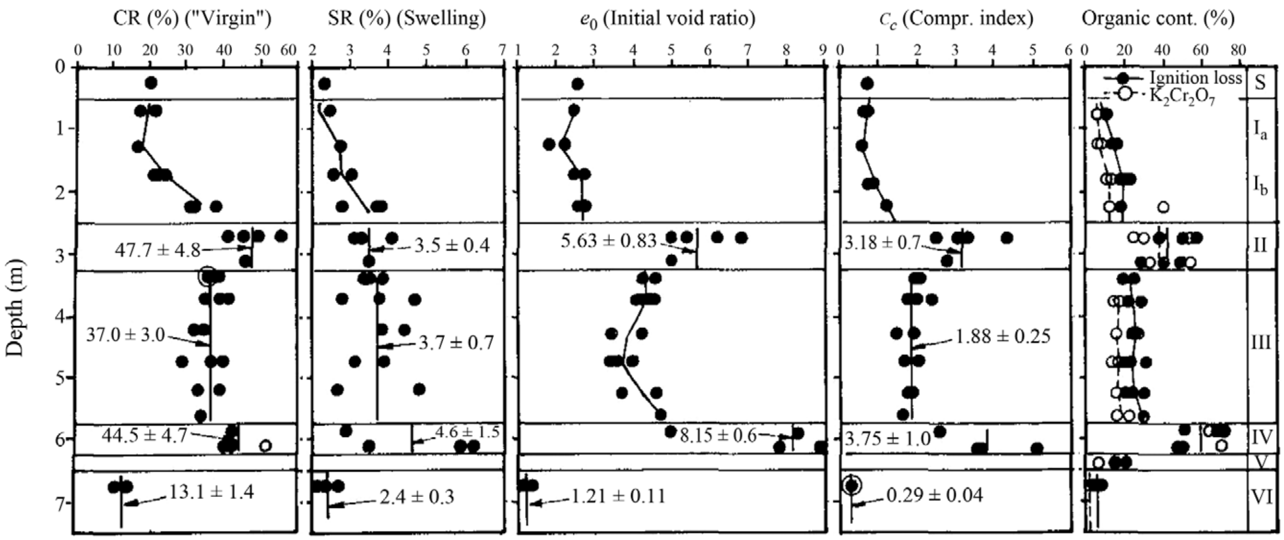


Figure 7 - CR, SR e_o and *Cc vs.* depth - oedometer tests in the Juturnaíba Dam (Coutinho & Lacerda 1994).

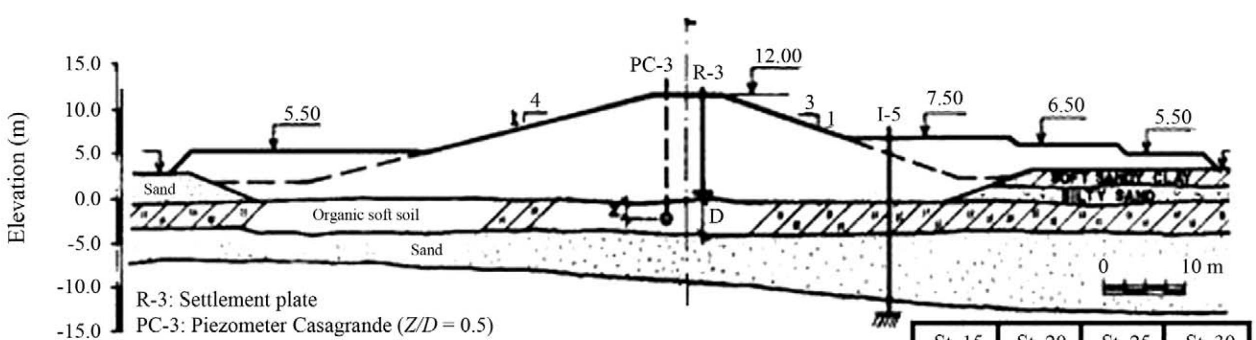
plates at the embankment clay interface, piezometers inside the organic clay, and inclinometers at the slope berm. Eleven stakes (15 to 60) were instrumented and Fig. 8 examplifies (Coutinho *et al.* 1994; Lucena 1997).

The third case presents the study on the access embankments for the Jitituba River Bridge, located on the Alagoas - 413 Highway. This bridge was built before the access embankments. Due to the existence of a soft soil layer (12 m thick) and to the construction sequence of the bridge, there was a need to analyze the vertical and horizontal displacements and the consequent efforts on the piles of the bridge (Fig. 9). The 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 solution adopted consisted of constructing the embankments in stages, along with the use of prefabricated vertical drains and geotechnical instrumentation (Casagrande piezometers, settlement plates and inclinometers) to control and monitor the performance of the project (Cavalcante 2001; Cavalcante *et al.* 2003; 2004). The research studies on this case were made possible due to the partnership with Gusmão Engineer Associated.

2. Instrumentation

The accurate performance of instrumentation is fundamental to the adequate use of its results obtained during embankment construction. The general objectives of the instrumentation are: to evaluate the general behavior of the embankment; to obtain signs of imminent rupture thus allowing a control methodology to be adopted during construction; and to evaluate the behavior of instrumentation, by comparing measurements obtained from more than one instrument at the same location.

In order to evaluate the degree of consolidation and the strength of the clay, it is desirable to instrument the clay foundation with piezometers and settlement gauges. This is particularly important in stage construction as it estimates when the following stage may be constructed and to what level. When there are services, structures or bridge piles close to the embankment, it can be important to monitor the



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

Figure 8 - Geotechnical profile and instrumentation of Juturnaíba Dam (Coutinho *et al.* 1994) - Second case.

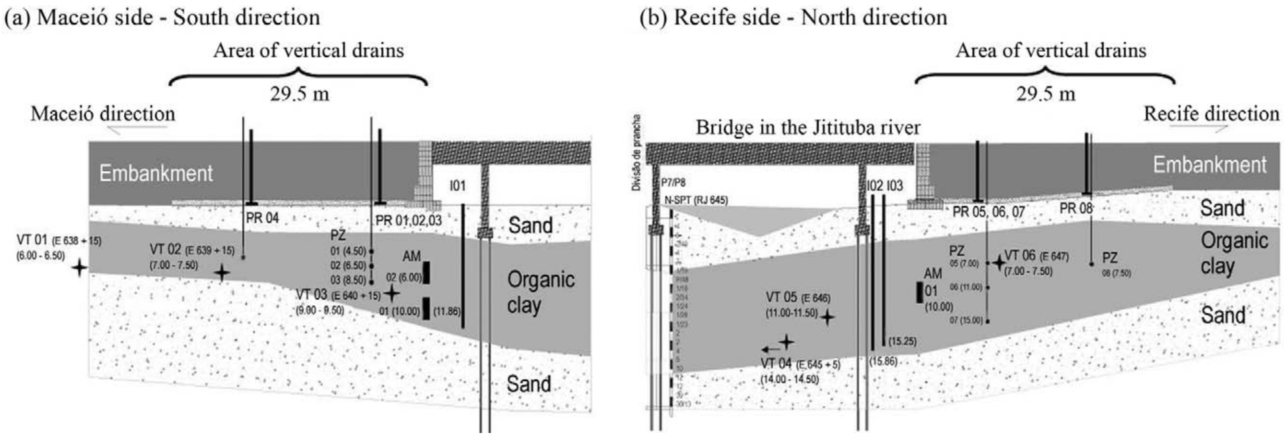


Figure 9 - Longitudinal section, geotechnical profile and the location of the field investigations of the basic project of the access embankments of the bridge on the Jitituba River (Cavalcante 2001; Cavalcante *et al.* 2004).

lateral displacements using inclinometers (Leroueil & Rowe 2000).

In most cases, the behavior of the embankment is monitored with respect to the following variables: vertical displacements (at surface and depth); horizontal displacements (at surface and depth); pore pressure; and total stress in the embankment (not common).

As a practical consideration, the observation of the pore pressures generated during construction under the centerline of an embankment can generally be used to calculate the vertical yield stress of the clay at the level of the piezometers. It can also give an indication of local shear failure when the ratio $\Delta u/\Delta\sigma$ becomes larger than 1.0. The strain between two deep settlement gauges can be used in conjunction with pore pressure measurements to define an *in situ* effective stress-strain curve that can then be compared with the compression curve assumed for the sublayer considered. When the clay deposit and the consolidation conditions are relatively simple, Asaoka's (1978) method can be used during the consolidation process to evaluate the approximate magnitude of the final settlement as well as to determine a representative coefficient of consolidation (Leroueil & Rowe 2000).

Figure 4 shows the instrumentation used in Juturnaíba trial embankment. The following instruments were used:

• Measurement of vertical displacements: 4 settlement plates placed throughout the instrumented section (Pli); 12 magnetic strain gauges, placed in the foundation deposit throughout the instrumented section (EMVi); 1 continuous pipe at the base of the embankment with 12 points for measurements with a full-profile settlement gauge (Pfi); 18 surface marks installed on the soil surface in the central region of the failure zone (Msi). A bench-mark (RNP) was utilized - it was installed at a location far from the region of influence of the displacements.

• Measurement of horizontal displacements: 4 inclinometer tubes (Ii); 8 measuring points, using a horizontal magnetic strain gauge, distributed throughout a continuous tube at the base of the embankment (EMHi); 18 surface marks placed on the soil surface in the central region of the failure zone (Msi).

• Measurement of pore pressure: 9 pneumatic (Pi) and 10 Casagrande piezometers (PCi), placed in the foundation deposit.

• Identification of the failure surface: 7 pipes were placed to help defining the failure surface (ISRi); 4 inclinometer pipes (Ii).

Two water level measuring gauges and one Casagrande piezometer were installed in the underlying sand layer, in front of the embankment, outside the zone of influence of the construction. The instruments were arranged in order to concentrate them in a single section, with redundancy towards the amount and the type of instrument, thus allowing relevant information at surface and depth in the foundation soil to be obtained. Initial measures were taken so as to evaluate the behavior of the instruments with regard to repeatability of the measurement technique. The accuracy of the measurements was evaluated by comparing results from different systems for a given position.

Coutinho (1986) evaluated the performance of the instruments used for measuring the vertical displacement (settlement plates, vertical magnetic strain gauge, full-profile settlement gauge and superficial marks) and horizontal displacement (horizontal magnetic strain gauge, inclinometer and superficial marks) in the Juturnaíba trial embankment. Tables 1 and 2 show the sensitivity, precision, reliability and accuracy of the instruments for vertical and horizontal displacement, respectively. The formation of groups on the accuracy of the results of vertical displacements can be observed, namely settlement plates, surface marks and vertical magnetic strain gauge. Accuracy was in the order of ±5 to 7 mm; and the accuracy of the full-profile settlement gauge was of the order of ±17 mm. The accuracy of the measurements of horizontal displacements for all instruments was of the order of ±5 mm.

Table 1 - Sensitivity, precision, reliability and accuracy of vertical displacements - Juturnaíba (Coutinho 1986).

Instruments		Settlement plates	Perfilometer	Vertical magnetic strain gauge	Surface marks
Sensitivity (mm)	1	~7	1	1	
Precision (mm)	± 2	± 10	± 1	± 2	
Reliability	Very good	Good	Good	Excellent	
Accuracy (mm)	5	17	7	5	
Deviation in relation of average observed values (mm)	Average	Position (2)	3.0	5.0	5.0
		Position (3)	8.9	6.7	16.5
	Standard deviation	Position (2)	2.3	4.3	4.8
		Position (3)	4.1	4.8	6.3
	Range 90% (mm)	Position (2)	6.8	12.1	12.9
		Position (3)	15.6	14.6	26.9

Note: Positions (2) and (3) - see Fig. 4.

Table 2 - Sensitivity, precision, reliability and accuracy of horizontal displacements-Juturnaíba (Coutinho 1986).

Instruments		Surface marks		Inclinometers		Horizontal magnetic strain gauge	
		MS (1)	MS (2)	IN (1)	IN (2)	EMH (1)	EMH (2)
Sensitivity (mm)		1		1:10,000 rad		1	
Precision (mm)		± 3		± 2		± 2	
Reliability		Excellent		Very good		Good	
Accuracy (mm)		5		5		5	
Deviation in relation of observed values (mm) - Pos (3)	Average	3.0	6.0	1.0	3.0	4.0	8.0
	Standard deviation	1.0	2.5	1.0	2.7	1.2	2.3
	Range (90%)	4.6	10.1	2.6	7.4	6.0	11.8

Notes: a) Values for height of embankment: (1) until 4.0 m; (2) greater than 4.0 m. b) Position (3) - see Fig. 4.

Figure 10 shows the summary of the vertical and horizontal displacements measured by different instruments at the base of the Juturnaíba trial embankment during construction, before the failure. These measurements show the occurrence of significant displacements of the foundation surface, which increase gradually as the height of the embankment increases. The maximum vertical values were greater than 500 mm at a height of 6.4 m just before failure. In the case of horizontal displacements, the maximum value was 200 mm and there is a position in the base of the embankment at which a change of the displacement direction occurred, causing tensile stresses.

The measurement of pore pressure taken before the construction of the trial embankment started, showed a deviation of less than ±1 kPa for both instruments (pneumatic and hydraulic Casagrande). Figures 11 and 12 present pore pressure isochronous measured under the center of the embankment and the results of the comparative study between the two types of piezometers, respectively. The excess of pore pressure (Δu) values measured, using the pneumatic piezometers, were always higher than the corresponding Casagrande's values.

As the height of the embankment increases, to near the failure of the foundation, the values measured by pneumatics piezometers in the center of the soft layer present significant increases, unlike the values measured with the Casagrande piezometers. In general, the values obtained for the ratio between measurements from the two piezometer types were in the range of 0.8 to 1.0 for heights of up to 4.65 m (Fig. 12). As the embankment height increases, that ratio decreases (range of 0.8 to 0.5) until the embankment reaches failure. The average value is in the order of 0.75.

Measurements from inclinometer, failure surface indicator (ISR's) and failure visual signs (cracks) were used to localize the failure surface (Fig. 13). Considering all the points observed it was verified that a circular surface tan-

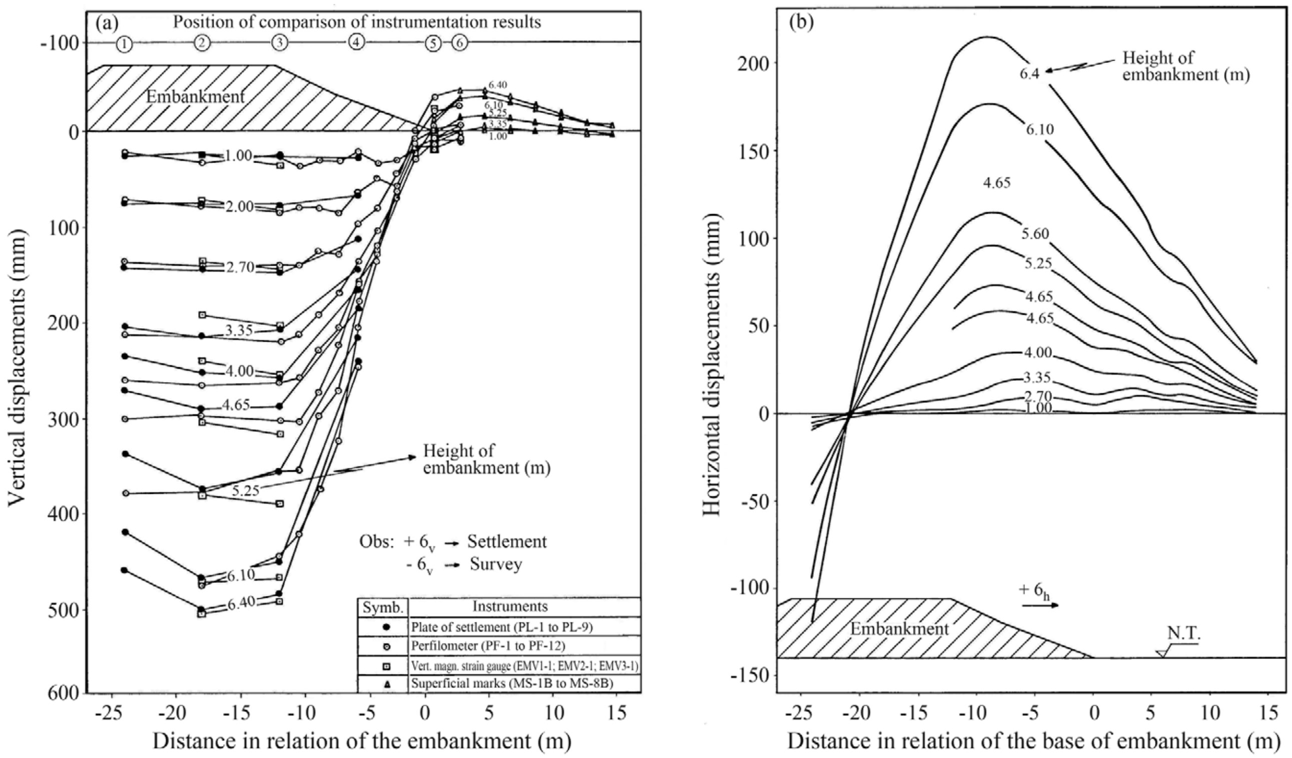


Figure 10 - Comparison between displacements measured in the base of the Juturnaíba trial embankment during construction, before the failure: (a) vertical and (b) horizontal (Coutinho 1986).

gent near to the resistant layer can represent the failure surface. These results were close to those indicated by the measurements from the inclinometers. A mechanism of failure of the planar type, in blocks, also seems to explain the phenomenon of the failure. Examples of possible failure surface are presented in Fig. 13.

Figure 14 presents the results of the geotechnical instrumentation obtained in the first 140 days in the access embankments of the bridge over the Jitituba River. After this period, the construction was paralyzed. Later, paving was carried out and the operation of the bridge was permitted, and only measurements of horizontal displacements could be performed.

3. Pore Pressure

As to the conventional design of embankment on clay foundations, it has been assumed that the behavior is perfectly undrained during the construction. For this case the pore pressure generated Δu can be given by Eq. (2).

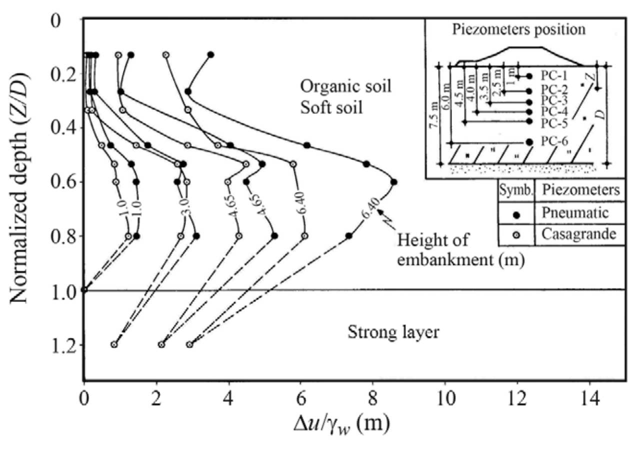


Figure 11 - Pore pressure *vs.* depth - Casagrande and pneumatic piezometers- Juturnaíba trial embankment (Coutinho 1986).

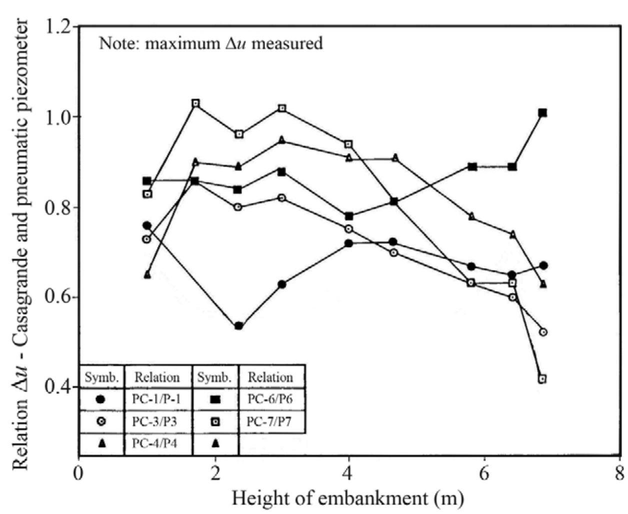


Figure 12 - Comparison among pore pressure measured by Casagrande and pneumatic piezometers - Juturnaíba trial embankment (Coutinho 1986).

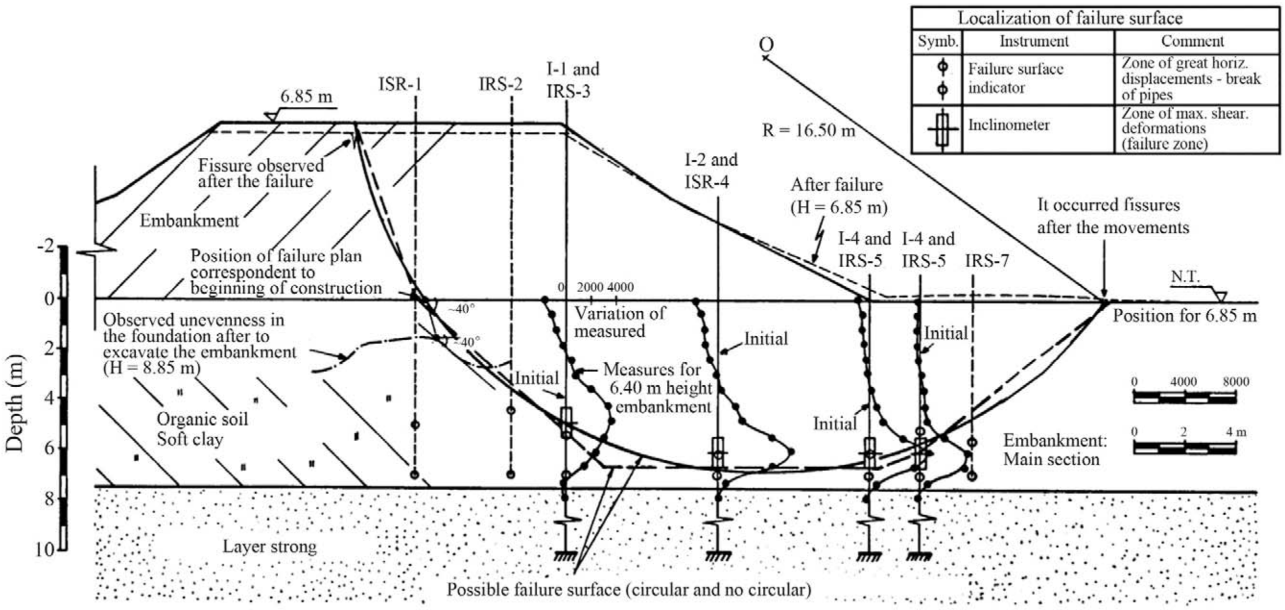


Figure 13 - Localization of the failure surface - Juturnaíba trial embankment (Coutinho 1986).

$$\Delta u = f(\Delta\sigma) \tag{2}$$

As the total stress varies, the pore pressure increase is given by the octaedrical stress increase $\Delta\sigma_{oct}$ (Eq. (3)). This methodology is based on the direct application of the elastic theory, and it is used when the pressure conditions that were imposed are in the limit of elastic behavior of the clayey soil. The principal total stress increases are $\Delta\sigma_1$, $\Delta\sigma_2$, $\Delta\sigma_3$.

$$\Delta u = \Delta\sigma_{oct} = 1/3\,(\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3) \tag{3}$$

In the case of the oedometer test, where an one-dimensional compression occurs, the pore pressure increase due to undrained loading is given by the vertical stress increase $\Delta\sigma_v$ (Eq. (4)).

$$\Delta u = \Delta\sigma_1 = \Delta\sigma_v \tag{4}$$

In many cases, significant partial consolidation during construction has been reported (item 1.1). In these cases pore pressures generated can, in general, be illustrated as in Fig. 2b, with $\bar{B}_1$, during the early stages of loading (Fig. 1) being given by:

$$\bar{B}_1 = 0.6\text{-}2.4\,(Z/D - 0.5)^2 \tag{5}$$

This equation is applicable until the local vertical effective stress reaches the preconsolidation pressure σ'_p, of the clay (O-P', Fig. 2a; OCR < 2.5). After that the clay behaves as normally consolidated and the corresponding height of embankment H_{nc} can be obtained by:

$$\gamma H_{nc} = \sigma'_{vy} \text{ or } \sigma'_{crit} = (\sigma'_p - \sigma'_{vo}) / I(1 - \bar{B}_1) \tag{6}$$

where $\bar{B}_1$ = pore pressure parameter; Z/D = normalized depth; D = thickness of the clay layer; I = influence factor; γ = unit weight of the embankment; γ'_{vy} = in situ vertical yield stress; σ'_{crit} = critical pressure; σ'_p = preconsolidation pressure; σ'_{vo} = initial vertical effective stress.

In the second phase of loading (P'-A', Fig. 2a) the increase in pore pressure is equal to the increase in total stress ($\bar{B}_2 = \Delta u/\Delta\sigma_v = 1.0$). At the end of construction, the excess pore pressure that will dissipate after construction is defined as the horizontal distance between A (total stress) and A' (effective stress) and given by:

$$\Delta u = \Delta\sigma_v - (\sigma'_p - \sigma'_{vo}) = I\gamma H - (\sigma'_p - \sigma'_{vo}) \tag{7}$$

If the third phase of loading occurs, local failure may be reached, and there may be softening of the clay (F' - C', Fig. 2a) associated with an increase in excess pore pressure larger than the increase in total stress ($\bar{B}_3 = \Delta u/\Delta\sigma_v > 1.0$). It should be noted that $\bar{B}_1$, $\bar{B}_2$ and $\bar{B}_3$ are incremental values during different stages of loading.

Coutinho (1986) presents and discusses results of pore pressure generated during the construction of the Juturnaíba trial embankment. Figures 11, 15 and 16 show examples of comparisons between predicted and observed values of Δu for some piezometers. The main conclusions are:

- It is clear that partial drainage occurred during construction. The piezometers located at the depths of 1.0 m and 2.0 m showed significant initial dissipation because they were placed close to the drainage boundary and in organic soils with high OCR values (OCR > 2.5)
- Δu values obtained by the Leroueil *et al.* method generally presented good estimates, particularly for the piezometers in the middle of the clay layer and for the highest height of the embankment. Near the failure height, other methods showed Δu values close to the values measured by

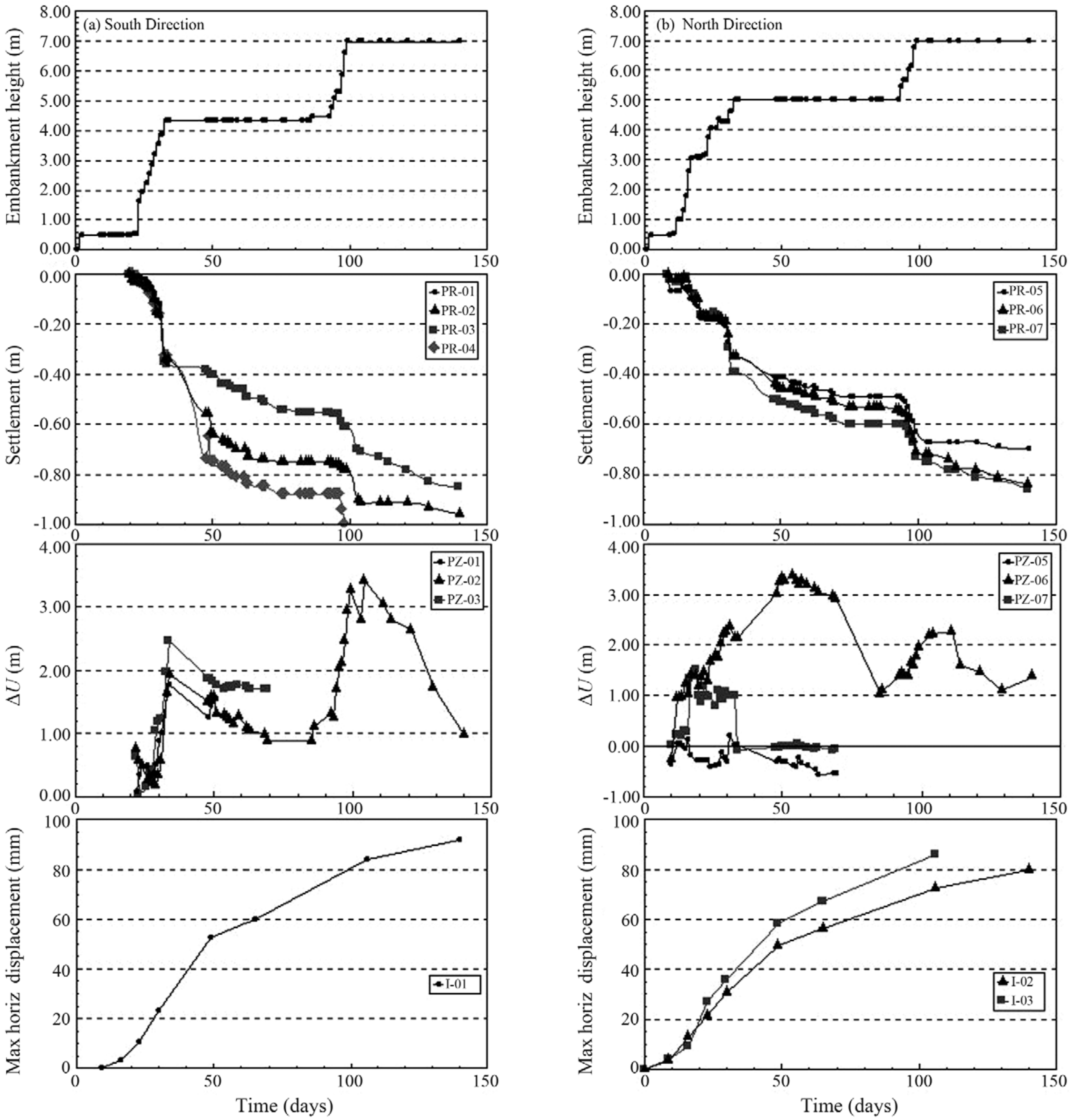


Figure 14 - Results of the geotechnical instrumentation measurements performed in the access embankments of the bridge on the Jitituba River (Cavalcante *et al.* 2004).

the piezometers in the middle of the clay layer (for example $\Delta u = \Delta\sigma_{oct}$).

• It is possible to see that the behavior of the clayey sand layer underneath the clay foundation was not completely drained (partial drainage occurred).

• In the central piezometers P-4 and P-5, a sign of local failure ($B_f > 1.0$) was observed, as indicated by Leroueil *et al.* (1978).

• It is observed that the pore pressure *vs.* depth curves (Figs. 11 and 16) takes the shape of a bell, with the axis of symmetry located below half the height of the soft layer. The behavior was not of the conventional type, because significant drainage in the upper stretch of the foundation occurred up to an embankment height of almost 3.0 m, whereas there was only partial draining in the underlying sand layer.

Figure 17 and Table 3 show results of the preconsolidation pressure (σ'_p) and vertical effective stress σ'_{crit} (σ'_{vy}) obtained in the Juturnaíba trial embankment by two procedures: oedometric tests and field pore pressure using Le-

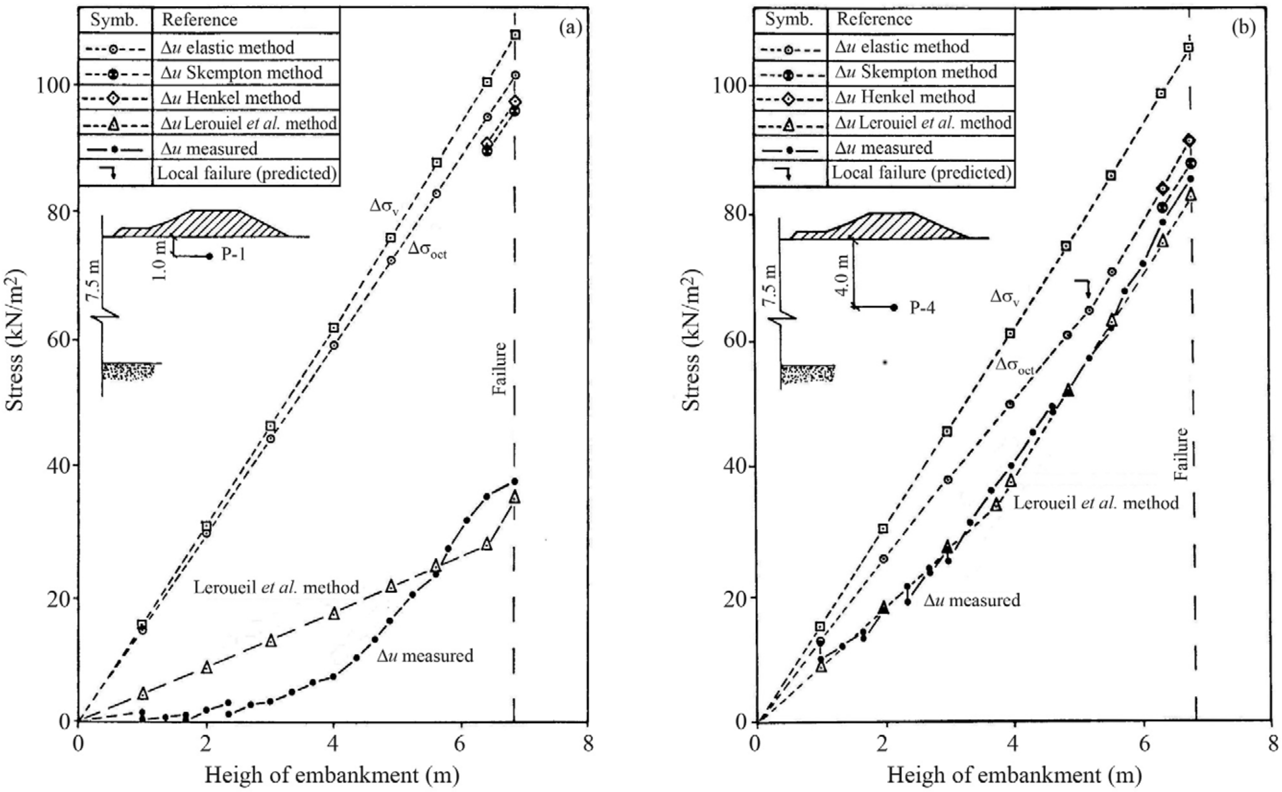


Figure 15 - Comparison between predicted and observed Δu value in the Juturnaíba trial embankment - Piezometer: (a) P1; (b) P4 (Coutinho 1986).

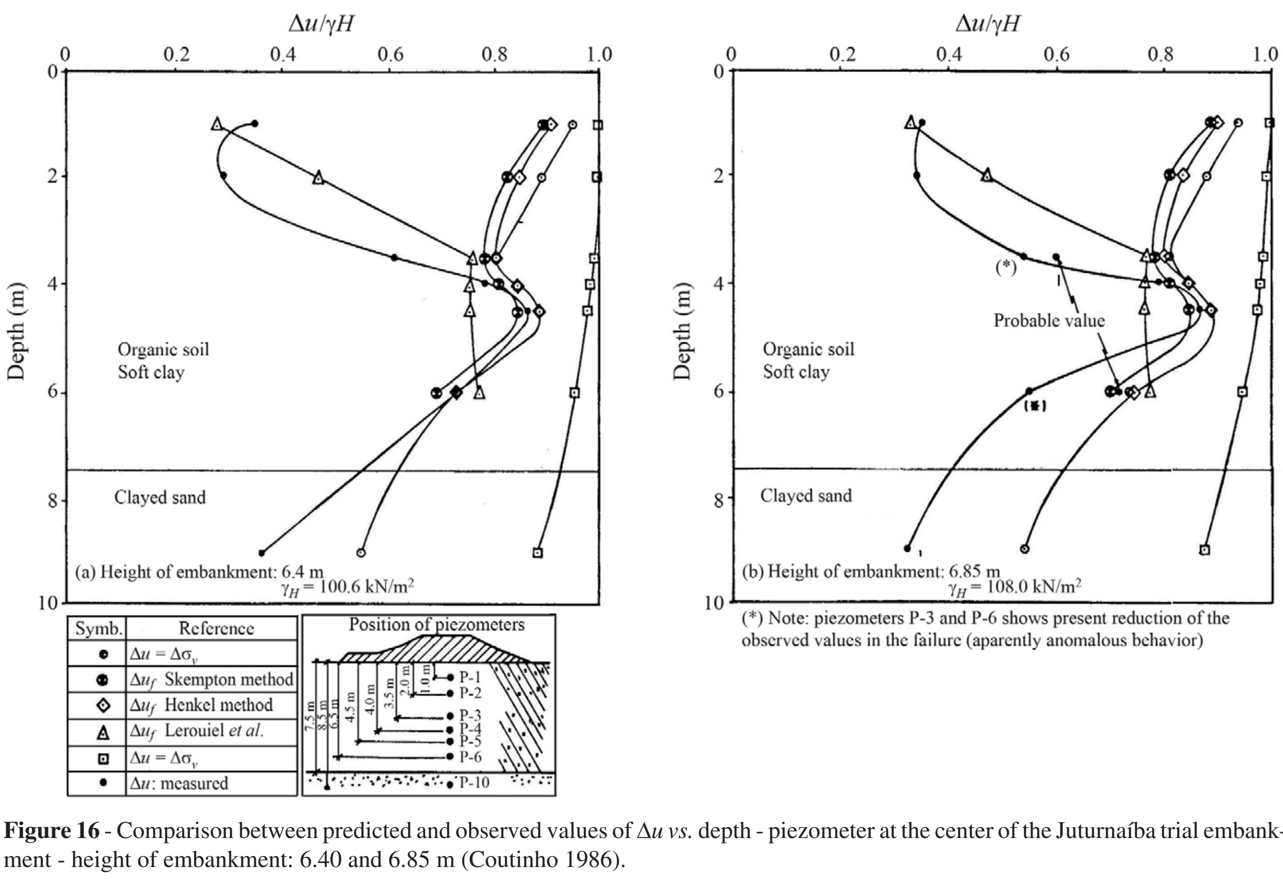


Figure 16 - Comparison between predicted and observed values of Δu *vs.* depth - piezometer at the center of the Juturnaíba trial embankment - height of embankment: 6.40 and 6.85 m (Coutinho 1986).

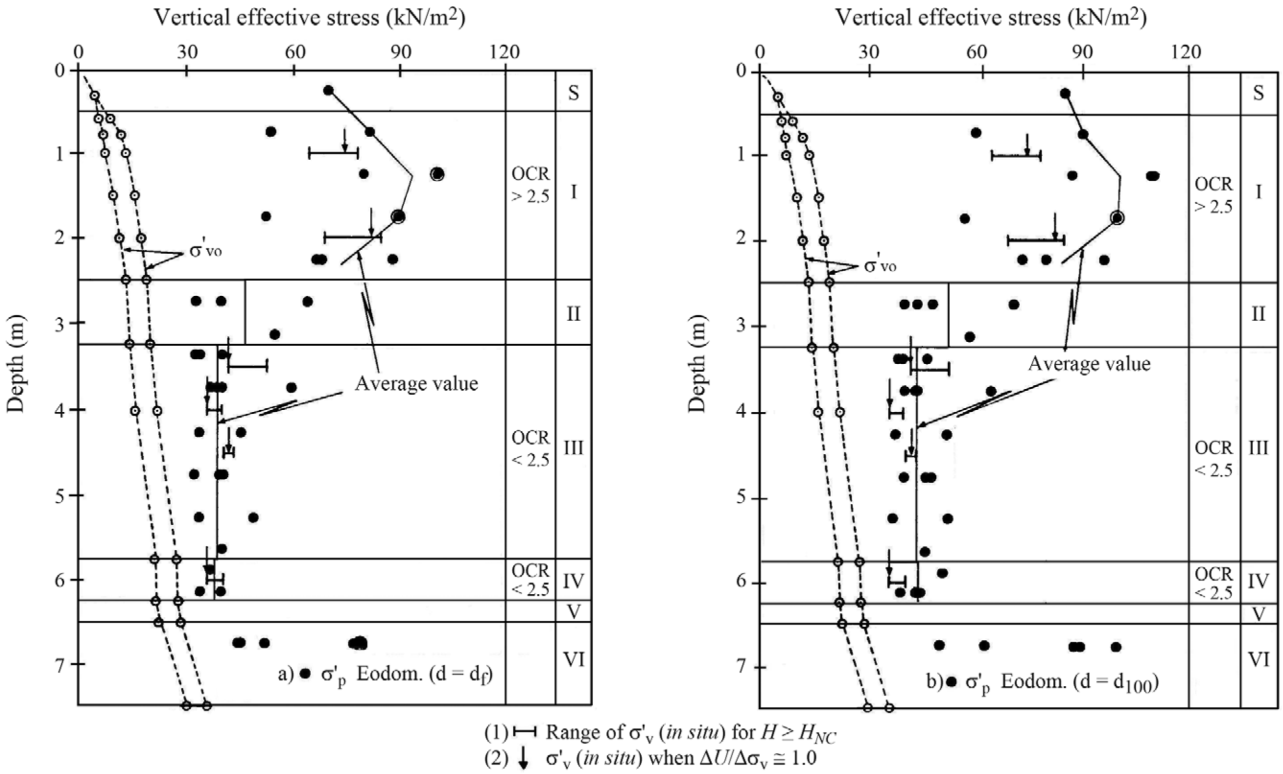


Figure 17 - σ'_{vo}, $\sigma'_{v\,crit}$, σ'_v and σ'_p values (laboratory and *in situ*) *vs.* depth - center of the Juturnaíba trial embankment (Coutinho 1986).

roueil *et al.* (1978) proposal. Field results showed reasonable agreement with described in item 1.1. For OCR > 2.5, the σ'_{crit} values were smaller than σ'_p showing an increase as the construction continued and reaching values close to σ'_p for an embankment height of 5.6 m. For OCR < 2.5, the σ'_{crit} values were in the same order as σ'_p values, remaining constant in the same range while the construction continued. The condition $\Delta u \cong \Delta\sigma_v$ was obtained for an embankment height of 3.0 m for the foundation zone with OCR < 2.5, and for an embankment height of 5.6 m in the foundation zone with OCR > 2.5.

Table 3 also shows a summary of the observed values of pore pressure coefficient ($\overline{B}$) and effective stresses in comparison with those predicted by Leroueil *et al.* (1978). The relation observed between Δu and $\Delta\sigma_v$ for the OCR > 2.5 presents a further stage in relation to soils with OCR < 2.5, due to the difference between the effective stress paths in each case (Fig. 2a), showing the "specificity" and complexity of the behavior of the clay foundation of the Juturnaíba trial embankment.

Coutinho *et al.* (1994) presented an analysis of the Juturnaíba Dam behavior with primary emphasis on pore pressure and settlement data. One procedure consisted of comparing numerical finite difference (CONMULT PROGRAM) predictions with instrumentation data. Another approach made use of the classical method (undrained condition) and the Tavenas & Leroueil method (partial drained condition) to estimate the pore pressure increases during construction.

The correction in the measurements of pore pressures from the Casagrande piezometers proposed by Coutinho (1986) was applied in the analysis of the response to the stress increase, where Δu corrected ($\Delta u_{pneumatic}$) = $\Delta u_{Casagrande}$ / 0.75 (see item 2, Fig. 12). The predicted and measured Δu values at piezometers C1 and C3 are shown in Fig. 18a. Reasonable agreement is reached at loading stages 1 and 2, but for stages 3 and 4 the agreement is poor. The discrepancy seems to be related to the generated Δu calculated by the CONMULT PROGRAM (undrained condition) and the condition in the field.

Generally Δu generated during construction is related to the increasing vertical load from the embankment. Predicted and measured Δu values at piezometers C3 (typical example) are showing in Fig. 18b, where the measured Δu is much lower than the predicted values from conventional methods, considering undrained condition throughout construction. Good agreement is observed between measured Δu and predicted values by Leroueil *et al.* method, which considers partial drainage during construction, until the embankment rises to a height of 5.6 m. As the embankment height increases, the clay layer is supposed to reach a normally consolidated condition. Under this condition, the measured values are lower than the predicted ones, show-

Table 3 - Comparison between observed and predicted excess pore pressure values by Leroueil *et al.* (1978) Method - Juturnaíba trial embankment (Coutinho 1986).

Piezometer			$\bar{B}_{measur.} = \Delta U / \Delta\sigma_V$				$\bar{B}_{measur.} = \Delta U / \Delta\sigma_V$				H_{nc} (m)	Effective stress kN/m^2			Velocity of construction
Nº	Depth (m)	Layer	$\sigma'_v \le \sigma'_{crit}$ $\bar{B}_1$	$\sigma'_v > \sigma'_{crit}$			$\sigma'_v \le \sigma'_{crit}$ $\bar{B}_1$	$\sigma'_v > \sigma'_{crit}$			Observed	σ'_{vo}	σ'_p	σ'_{crit}	
				1° stage $\bar{B}_2$	2° stage $\bar{B}_3$	Final stage $\bar{B}_f$		1° stage $\bar{B}_2$	2° stage $\bar{B}_3$	Final stage $\bar{B}_f$					
P-1	1.0	Ia	0.117	0.63	0.90	-	(2)	(3)	1.0	1.0	4.0	10.0	86.0	64.6	square below
P-2	2.0	Ib	0.100	0.48	0.85	-			1.0	1.0	4.0	10.9	82.5	68.7	

(a) Piezometers installed in clayey soils with OCR > 2.5.
(1) σ'_p value adopted corresponding to representative value in the position of piezometer in Fig. 5.
(2) The average $\bar{B}_1$ value throughout the layer was 0.22.
(3) $\bar{B}_2$ value varied between 0.42 and 1.01, with average of 0.75.

Piezometer			$\bar{B}_{measur.} = \Delta U / \Delta\sigma_V$			$\bar{B}_{measur.} = \Delta U / \Delta\sigma_V$			H_{nc} (m)		Effective stress (kN/m^2)			Velocity of construction	
Nº	Depth (m)	Layer	(4) $\sigma'_v \le \sigma'_{crit}$ $\bar{B}_1$	$\sigma'_v > \sigma'_{crit}$		$\sigma'_v > \sigma'_{crit}$	$\sigma'_v \le \sigma'_{crit}$ $\bar{B}_1$		Obs	Pred	σ'_{vo}	σ'_p	σ'_{crit}	Height of embankment (m)	Velocity average (kN/m^2 / day)
				1° stage $\bar{B}_2$	(5) Final stage $\bar{B}_f$		1° stage $\bar{B}_2$	(2) Final stage $\bar{B}_f$							
P-3	3.0	III	0.278	0.85	~0.85	0.597	1.0	> 1.0	2.35	3.8	14.6	38.75	42.0	0-3.0	2.20
P-4	4.0	III	0.556	0.91	1.25	0.597	1.0	> 1.0	3.0	3.76	15.6	38.75	36.0		
P-5	4.5	III	~0.547	~1.0	1.45	0.576	1.0	> 1.0	~3.0	3.45	16.5	38.75	42.4	3.0-6.85	4.30
P-6	4.5	IV	0.640	0.92	~0.92	0.384	1.0	> 1.0	3.0	2.0	19.9	38.3	36.1		

(b) Piezometers installed in clayey soils with OCR < 2.5.
(4) $\bar{B}_1$ value corresponding to the last measurement of pore pressure for $\sigma'_v = \sigma'_{crit}$.
(5) $\bar{B}_f$ value corresponding to the representative value for an embankment height of 5.6 m to 6.4 m.
Obs: Observed; Pred: Predict.

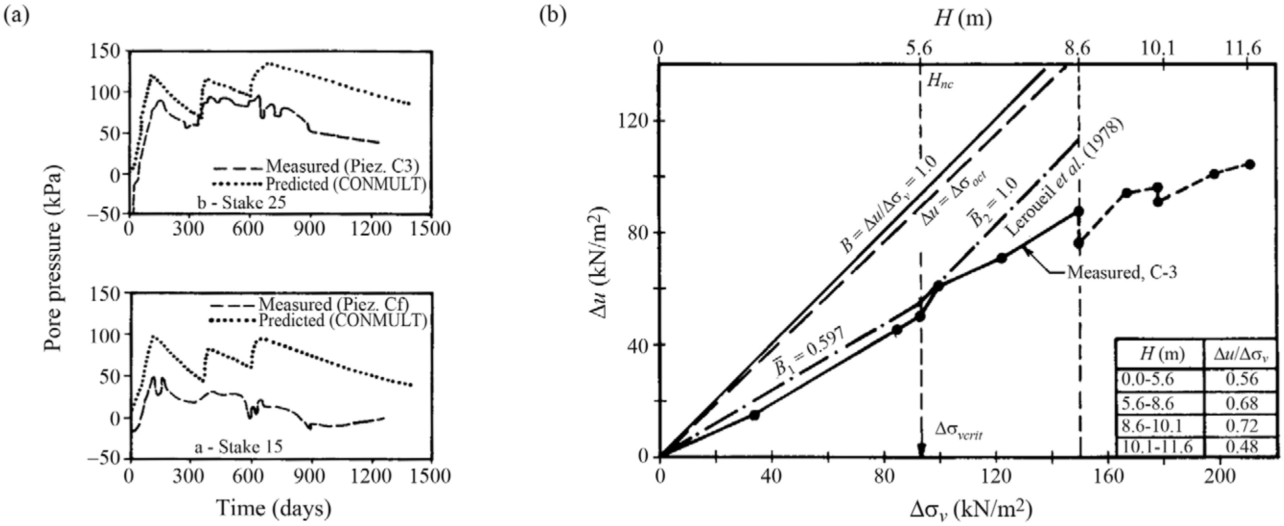


Figure 18 - Predicted and measured Δu in the Juturnaíba Dam: (a) against time; (b) during construction (Coutinho *et al.* 1994).

ing a ratio $\Delta u/\Delta\sigma_v$ of about 0.7, instead of 1.0, as predicted by the method. It seems that partial drainage continues to occur during construction.

The main objective to the follow up of pore pressures in the Jitituba embankments was to evaluate the efficiency of the adopted solution to accelerate the settlements and dissipate the pore-pressures, that is to say, prefabricated vertical drains (Cavalcante *et al.* 2003; 2004). The large time-lag of the Casagrande piezometer allied to the variations of the water level makes the analysis of data from piezometers more difficult. The results obtained from piezometers PZ-02 (South Direction) and PZ-06 (North Direction) were evaluated in relation to the response to the increase in stress and the dissipation of pore pressures with time, including the coefficient of horizontal consolidation by Orleach method (1983). This method is based in the Terzaghi Theory (1943) to obtain the coefficient of vertical consolidation and in the Barron Theory (1948) to obtain the coefficient of radial consolidation.

In order to minimize the effect of the Casagrande piezometer limitations, in this case the corrections on the measurements of pore pressures from Casagrande piezometers were also applied as proposed by Coutinho (1986) (see item 2).

Figure 19 presents the graph Δu *vs.* $\Delta\sigma_v$ (including stages construction) for the piezometer PZ-02 (South Direction), which relates the pore-pressure increase (measured and corrected) to the stress increase. Table 4 presents the values of Δu, $\Delta\sigma_v$, $\Delta u/\Delta\sigma_v$ and $\Delta u_{corrected}/\Delta\sigma_v$ for two piezometers and for both stages of construction. It is observed that the ratios of both $\Delta u/\Delta\sigma_v$ and $\Delta u_{corrected}/\Delta\sigma_v$ presented a compatible maximum value of 0.58 with a partially drained behavior during construction ($\Delta u/\Delta\sigma_v < 0.6$) (see item 1.1). This behavior is due to the possible overconsolidated behavior (high c_v or c_h) at the start of the construction,

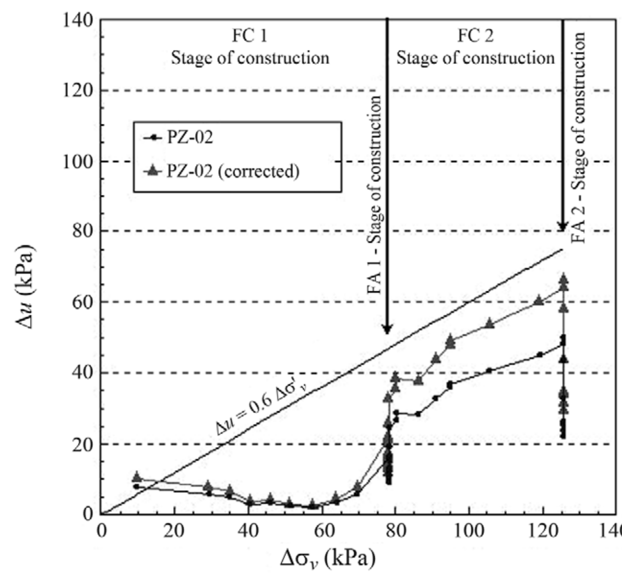


Figure 19 - Pore-pressures responses - South direction of Jitituba embankment (Cavalcante 2001).

Table 4 - Pore Pressure increase (Δu) *vs.* stress increase ($\Delta\sigma_v$) - Jitituba embankment (Cavalcante 2001).

Direction	South PZ-02	North PZ-06
Δu (kPa)	12	34
$\Delta\sigma_v$ (kPa)	69	82
$\Delta u/\Delta\sigma_v$	0.17	0.41
$\Delta u_{corrected}/\Delta\sigma_v$	0.23	0.55
Δu (kPa)	21	13
$\Delta\sigma_v$ (kPa)	48	35
$\Delta u/\Delta\sigma_v$	0.43	0.36
$\Delta u_{corrected}/\Delta\sigma_v$	0.58	0.48

and/or to the use of vertical drains in order to accelerate settlement, where both cause a faster dissipation of the pore-pressures generated.

The Orleach (1983) method was applied to obtain the coefficient of radial consolidation for piezometers PZ-02 South Direction (Fig. 20) and PZ-06 North Direction. It is observed in Table 5 that the PZ-06 (North) presents smaller consolidation coefficients than the PZ-02 (South), in both construction stages. The second stage of the construction presents a higher consolidation coefficient than the first one, on both sides. When compared with the laboratory average (normally consolidated interval) they become 1.04 and 1.50 times higher for the South and North Direction, respectively. When compared to the value used in the project, they are lower. The samples used in the laboratory were of low quality, which influenced the results of the consolidation parameters, reducing the values of c_v and σ'_P (Coutinho *et al.* 1998). In spite of the mechanical limitations (time-lag) of the Casagrande piezometers, the application of the Orleach (1983) method yielded reasonable results in comparison with laboratory results with low quality samples.

4. Vertical Displacements

The analysis of vertical displacements of an embankment consists of one or more stages, and usually requires the prediction of the initial and long-term settlements and their variation with time. The total settlement will be the sum of the settlements during construction and settlements in the long term. As described in item 1.1, the behavior of an embankment on clay foundations can occur in some cases when it is essentially undrained and in many cases with partial drainage during construction.

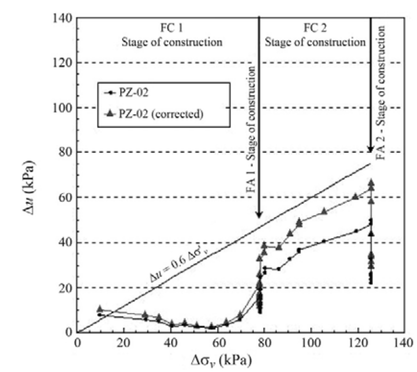


Figure 20 - Log (Δu) *vs.* time - Application of the Orleach (1983) method, Jitituba embankment (Cavalcante 2001).

Table 5 - Consolidation coefficient obtained by Orleach (1983) Method in the laboratory and adopted in the project - Jitituba embankment (Cavalcante 2001).

		1° Stage		2° Stage	
		α1	c_h 10^{-8}. m^2/s	α1	c_h 10^{-8}. m^2/s
Orleach (1983)	South	0.024	7.65	0.032	10.29
	North	0.008	2.51	0.031	9.98
	Average	-	5.08	-	10.14
Laboratory	-	4.0-8.0	-	4.0-8.0	
Project	-	15.0	-	15.0	

4.1 Construction settlements

4.1.1. Undrained condition
("Conventional Design Approach")

When a load is applied quickly to a limited area on a clay deposit, the strain induced in the clay causes lateral deformation of the soil, resulting in settlement. This settlement is generally considered as an instantaneous response to the load applied, occurring under undrained conditions and known as immediate or initial settlement, S_c. The prediction of the initial settlement uses a model derived from elasticity theory and has the form:

$$S_c = \rho_i = [q\, B\, (1 - \upsilon^2)\, I\rho] / E_u \qquad (8)$$

where ρ_i = immediate or initial settlement; q = stress applied to soil foundation; B = width or diameter of the loaded area; υ = Poisson coefficient, in this case 0.5; $I\rho$ = influence factor, which depends on the geometry of the problem; E_u = undrained Young modulus of the soil.

The initial settlement tends to be small in comparison with the settlement due to consolidation. This occurs when the base of the loaded area "B" is much bigger than the thickness of the clay layer "H", for which "$I\rho$" values become very small. Foot & Ladd (1981) presented situations where the "ρ_i" value was very significant. Soils with high plasticity and/or high organic content are susceptible to these movements especially when loaded with a low factor of safety. Using the calculation procedure considered by D'Appolonia *et al.* (1971), the authors proposed a method to predict "ρ_i" values for use in this project.

Figure 21a presents results of construction settlements (predicted and measured by plate PL-2) under the center of Juturnaíba trial embankment (Coutinho 1986). The agreement between the theoretical results and the values observed was not satisfactory. Only the results corresponding to circular loading and for a height of 6.40 m presented values close to the results observed. The low value of the relation H/B corresponds to very small values of $I\rho$ and, consequently, to small settlement values. The circular load presented higher values for $I\rho$. Coutinho *et al.* (1994) and Lucena (1994) used the same procedure to calculate the initial settlements in two sections of the Juturnaíba Dam and observed similar behavior.

4.1.2. Construction settlement ("Partial Drainage")

Leroueil *et al.* (1978) and Tavenas & Leroueil (1980) presented an empirical method for evaluating construction settlements of overconsolidated clay, with OCR < 2.5, where partial drainage of the foundation during construction was considered to have occurred. Figure 2a presents the effective stress path and considers the settlement takes place over two stages: initially a rapid consolidation in the Ko condition (preconsolidation compression) and after that, an essentially undrained shear deformation under the limit state curve.

The settlement of the first stage can be obtained from the results of oedometer tests on the overconsolidation clays, and can be calculated by the conventional expression:

$$S_r = \sum_{i=1}^{n} H \times RR \times \log \frac{\sigma'_p}{\sigma'_{vo}} \tag{9}$$

where RR = recompression index; H = clay layer thickness; σ'_p = preconsolidation pressure; σ'_{vo} = initial vertical effective stress.

When one significant part of the foundation becomes normally consolidated (second stage - $H_{emb} = H_{nc}$), the velocity of the occurrence of the settlement increases. The clay foundation, now with reduced rigidity and low permeability (Fig. 2a - P'A'), is deformed because of undrained shear distortion. After reviewing historical cases, Eq. 10) was proposed by the authors to obtain the undrained settlement S_u, corresponding to the second stage.

$$S_u = (0.07 \pm 0.03)\,(H - H_{nc}) \tag{10}$$

where: $\gamma H_{nc} = (\sigma'_p - \sigma'_{vo}) / I\,(1 - \overline{B}_1)$; I = influence factor; γ = unit weight of the embankment; $\overline{B}_1$ = pore pressure parameter.

The settlement S_c at the end of construction results from the sum of the recompression settlement, S_r (O' towards P' in Fig. 2a), and the undrained settlement, S_u (P'-A' in Fig. 2a):

$$S_c = S_r + S_u \tag{11}$$

Figure 21b presents results of calculated and observed (settlement plate PL-2) construction settlements in the center of the Juturnaíba trial embankment as a function of the height of the embankment. For this method (partial drainage) the agreement between predicted and measured values was satisfactory, and showed slightly higher values for lower embankment heights and slightly smaller values for greater embankment heights.

4.2. Long term settlement

The equation usually used in conventional designs to calculate the primary settlement of a deposit considers overconsolidated and normally consolidated compressions:

$$S_p = \sum_{i=1}^{n} \left(H \times RR \times \log \frac{\sigma'_p}{\sigma'_{vo}} + H \times CR \times \log \frac{\sigma'_{vf}}{\sigma'_p} \right) \tag{12}$$

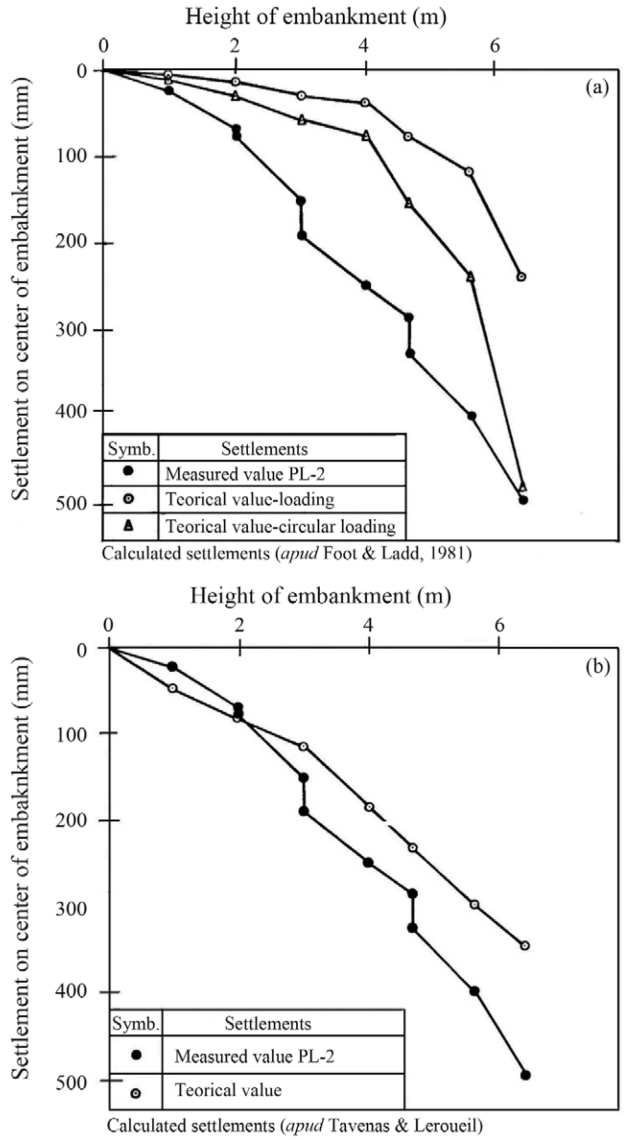


Figure 21 - Measured and calculated settlements - Juturnaíba trial embankments (Coutinho 1986).

where i = layer number; n = number of sub-layers; $RR = C_r / (1 + e_o)$, recompression coefficient; $CR = C_c / (1 + e_o)$, virgin compression coefficient; σ'_{vf} = final vertical effective stress.

In the case of the partial drainage method, the long term settlement, S_d, is the second part of the consolidation settlement (normally consolidated compression) associated with effective stress increase from σ'_p to σ'_{vf} ($\sigma'_{vo} + \Delta\sigma'$). Thus:

$$S_d = \frac{H}{1 + e_{oi}} C_{ci} \log\left(\frac{\sigma'_{voi} + \Delta\sigma'}{\sigma'_{pi}} \right) \tag{13}$$

The total settlement, S, for both cases (Leroueil *et al.* and conventional methods) is given by the sum of the construction and long term settlements, namely:

• Leroueil *et al.* method: $S = S_c + S_d = S_r + S_u + S_d$ (14)

• Conventional design: $S = S_c + S_p = \rho\, i + S_r + S_d$ (15)

The differences between the two approaches are basically: during construction, the Leroueil *et al.* method presents higher settlement; in the total settlement, the difference is between the values of ρi (the conventional method) and S_u (the Leroueil *et al.* method).

After the end of primary consolidation, settlement continues with time due to secondary consolidation, S_s, and in conventional design has been estimated by:

$$S_s = DC_{\alpha\varepsilon} \log\left(\frac{t}{t_p}\right) = \frac{DC_{\alpha e}}{1+e_p} \log\left(\frac{t}{t_p}\right) \quad (16)$$

Where $C_{\alpha\varepsilon}$ and $C_{\alpha e}$ are measured in oedometer tests or estimated from C_c; t_p = time at the end-of-primary consolidation; t = time to estimate the settlement; e_p = the void ratio at time t_p.

In fact, the influence of the viscous behavior of the clays (and also peats) in the settlement during primary consolidation is more complex. There are two extreme possibilities: (a) the creep occurs only after the end-of-primary consolidation, and consequently, the strain at the end-of-primary consolidation would be the same in situ and in laboratory; (b) the viscous strains develop during primary consolidation and, consequently, the strain at the end-of-primary consolidation is larger in situ than in the laboratory. In addition, there is also a discussion about a finite final value for the secondary compression (see Leroueil & Rowe 2000; Rémy *et al.* 2010a).

Coutinho *et al.* (1994) and Lucena (1994) present an analysis of the Juturnaíba Dam settlement behavior considering undrained condition during construction using two approaches: (a) classical conventional method (Terzaghi's Theory) and (b) CONMULT PROGRAM. Settlements were measured by settlement plates installed at the embankment-clay interface. The final primary settlement and the in situ coefficient of consolidation were obtained from settlement data using Asaoka's method (Asaoka 1978; Magnan & Deroy 1980). Computed and measured total settlements are show in Table 6 for the three sections and the eleven stakes analyzed. Good agreement between the results of settlements computed and the Asaoka's method is generally observed. Relative differences [(Asaoka - Terzaghi) / Asaoka] varied from -7.7 to +13% for the eleven points. Another good example of results and discussion about primary and secondary consolidation can be seen in Rémy *et al.* (2010a,b).

Computed and measured settlement curves are shown in Figs. 22a and 22b for two of the four stakes (section 2) analyzed by CONMULT PROGRAM. Good agreement between numerical and measured settlements is observed at stakes 15 and 30 (5 to 10%). Settlements are slightly overpredicted at stakes 20 and 25 (25 to 30%). Computed initial settlement rates are greater than the ones measured. In the classical method, consolidation analysis was conducted by using Terzaghi's theory, considering the loading stages and the load increase with time. The final settlement and the average coefficient of consolidation to be used in the analysis were also computed on the basis of the surface settlement using Asaoka's method. A typical example of classical consolidation analysis is shown in Fig. 22c. Very close agreement is observed in this case provided that a proper c_v and total settlement values are used for each construction stage and the load increase with time is considered. These results show the applicability of Terzaghi's Theory for this particular case. When the use of c_v of laboratory and settlements obtained from Terzaghi, the corrected predicted curves show some difference from the result measured by plate R-3.

Values of average *in situ* c_v were back-calculated from the settlement data for different loading stages using

Table 6 - Comparison between measured and predicted settlements Juturnaíba Dam (Borges 1991; Coutinho *et al.* 1994; Lucena 1997).

Section	Stake	Asaoka (mm) (1)	Plate R-3 (mm) (2)	Terzaghi (mm) (3)	Difference (%) (1)-(3)/(1)	Difference (%) (1)-(2)/(1)
III-2	46	1436	1400	1547	-7,73	2.51
	50	1346	1200	1371	-1.86	10.85
	55	318	312	297	6.60	1.89
	60	310	310	298	3.87	0
V	35	1196	1130	1040	13.04	5.52
	37+10	1349	1070	1436	-6.45	20.68
	40	1170	1150	1185	-1.28	1.71
II	15	1153	1155	1005	12.8	-0.17
	20	1270	1210	1350	-6.3	4.72
	25	1457	1430	1320	9.4	1.85
	30	-	1093	975	10.8	-

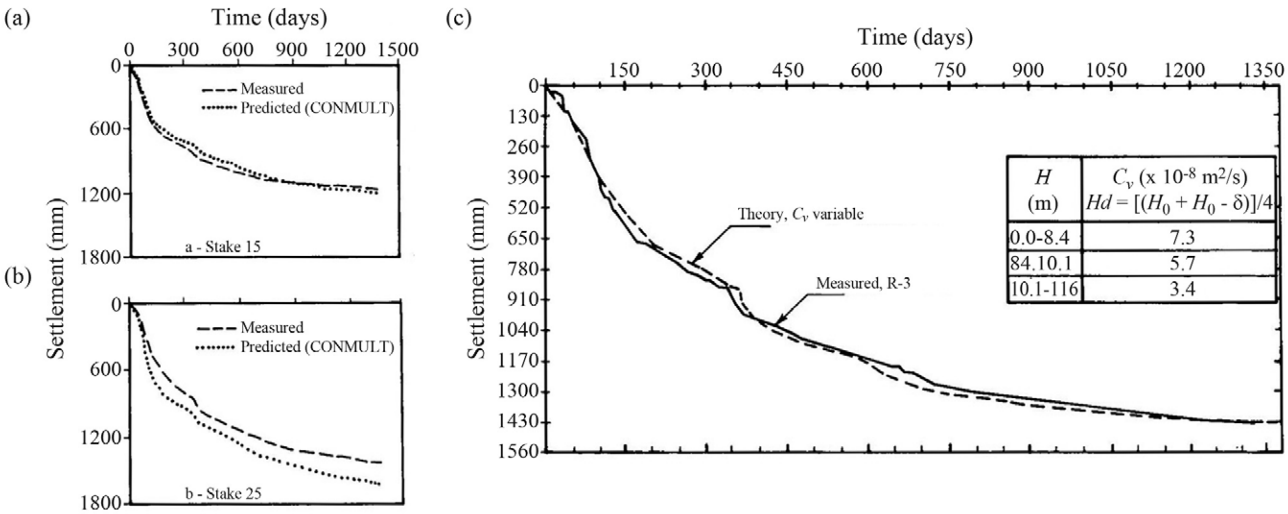


Figure 22 - Predicted and measured settlements against time - Juturnaíba Dam: (a) and (b) Conmult analysis; (c) Terzaghi analysis (Coutinho *et al.* 1994).

Asaoka's method. Table 7 compares such results with average values obtained in the laboratory for the range of the total stress applied. It can be seen that *in situ* values are higher than average laboratory values, but with the ratio c_v *in situ* / c_v lab in the order of 2 or less. The analysis performed using CONMULT showed that c_v values generally increase slightly up to the overconsolidation (yield) pressure, decrease abruptly afterwards and then show a slow decrease (Coutinho *et al.* 1994).

Cavalcante (2001) and Cavalcante *et al.* (2003) measured and analyzed vertical displacements in Jitituba embankment (Fig. 9). The maximum settlement and consolidation coefficient in the field were obtained, considering the period of the curve settlement *vs.* time, after the end of the construction of each stage (consolidation phase) (Fig. 23).

In the analysis of stress-deformation behavior, it is observed that estimated settlements using laboratory parameters (in normally consolidated soil) presented values about 40% to 60% higher than the maximum values measured in the field (Table 8). A possible reason for this difference is disturbance effects on the laboratory results. The stress-deformation behavior obtained in the laboratory was compared graphically to that in the field (settlement plates located in the center of embankments) with the final vertical effective stress (σ'_{vf}) values in the field being obtained through the difference between the estimated total stresses and the pore pressure measured in the field. The disturbance effects in the laboratory samples can be observed through the deformation corresponding to the initial effective stress σ'_{vo} (~14%) and the overconsolidation stress results to be lower than σ'_{vo} values (Fig. 24).

Table 8 - Settlement estimated using laboratory and maximum values measured in the field - Jitituba embankment (Cavalcante *et al.* 2003).

Direction	South	North
$\Delta\sigma_{v1}/\Delta\sigma v_{o1}$	3.0	2.4
CR	22%	22%
Layer thickness	9.70 m	11.20 m
Settlement laboratory	1.30 m 13.2%	1.30 m 11.6%
Settlement field	0,91 m 9.38%	0,80 m 7.14%
Difference lab-field	42.86	62.5

The coefficient of horizontal consolidation values obtained by the Asaoka (1978) method was higher in the North Direction than in the South Direction (around 26%). With regard to those obtained in laboratory, the field values

Table 7 - Coefficient of consolidation values: Section II - Juturnaíba Dam (Coutinho *et al.* 1994).

H (cm)	Laboratory (m²/s)		*In situ* (x 10^{-8} m²/s)			
	c_v		c_v Asaoka's method			
	Range	Mean stress	STAKE 15	STAKE 20	STAKE 25	MEAN
0.0-8.5	40.0-3.5	7.0	8.1	11.2	7.3	8.9
8.5-10.0	3.5-2.1	2.7	5.9	4.8	5.7	5.5
10.0-11.5	2.1-2.1	2.1	3.5	2.8	3.4	3.2

were from about 2.6 to 7.7 times higher (Table 9). The possible causes for this difference are the disturbance effects in the compressibility and consolidation parameters, that is, induces σ'_p and c_h values that are underestimated in the laboratory (Coutinho *et al.* 1998).

Results of c_h obtained from the Asaoka method were 2.3 to 3.1 times the values from the Orleach method (interpretation of Δu measured). In both methods, c_h was found to be higher in the second stage (1.18 and 2.65 times). One reason may be the low stress increment ratio, which results in a larger participation of the secondary settlement in the total settlement (Martins & Lacerda 1985) and/or it is not sufficient to exceed the overconsolidation caused by the secondary consolidation from previous stress stage (Leonards & Altschaefl 1964).

Cavalcante (2001) and Cavalcante *et al.* (2003), based on Massad (1988; 1999), proposed the estimation of an overconsolidation parameter in the field, OCR_{GLOBAL}, given the settlement expressed and the collected data of maximum settlements estimated / measured in the field and the estimated stresses σ'_{vo} and $\Delta\sigma'_v$. For all cases studied CR = 40% and SR = 5% were considered due to the similarity between the compressibility parameters of Brazilian soft clays. Figure 25 presents OCR_{GLOBAL} values obtained for several embankments. The influence of overconsolidation stress on the magnitude of deformations can be observed. In the Jitituba embankment, the OCR_{GLOBAL} presented average values of 2.1 and 2.6 for each stage of loading. In other embankments, this parameter presented values in the range of 1.3 and 6.7.

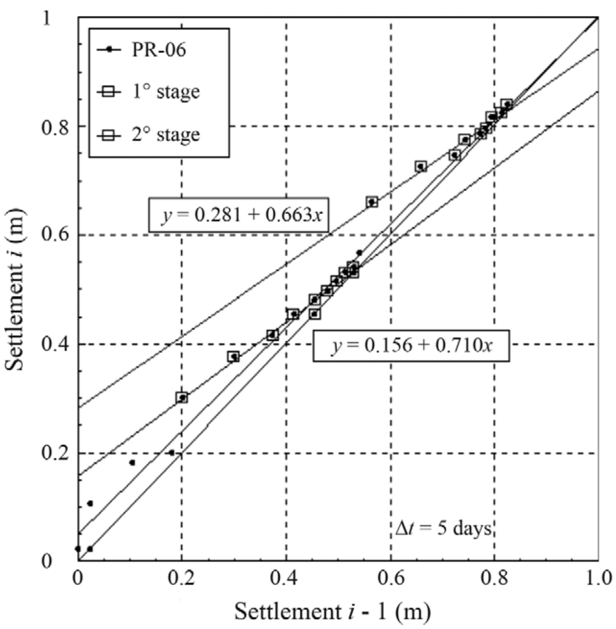


Figure 23 - Application of the Asaoka graphic construction, plate PR-06, North Direction - Jitituba embankment (Cavalcante *et al.* 2003).

5. Horizontal Displacement

Using empirical correlations and the Ylight model, Tavenas *et al.* (1979) present a method that allows the hori-

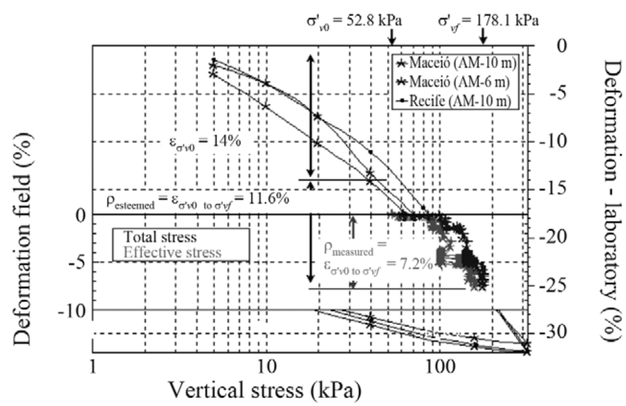


Figure 24 - Comparison between the behavior stress deformations observed in the field and in the laboratory, North Direction - Jitituba embankment (Cavalcante *et al.* 2003).

Table 9 - Comparison between c_h obtained through field measurements, from laboratory tests and used in the project - Jitituba embankment (Cavalcante *et al.* 2003).

Direction			South PZ-02		North PZ-06	
		Stage	1°	2°	1°	2°
Stress increase		$H_{emb\ end}$ (m)	4.34	6.98	5.03	6.96
		σ'_{V0i} (kPa)	41.80	119.92	52.80	143.3
		$\Delta\sigma_{Vi}$ (kPa)	78.12	47.52	90.54	34.74
		$\Delta\sigma_{Vi}/\Delta\sigma'_{Vi}$	1.87	0.40	1.71	0.24
c_h (10^{-8}.m^2/s)	Field	Asaoka (1978)	20.55	24.34	26.32	30.8
		Orleach (1983)	7.65	10.29	2.51	9.98
		Laboratory	4.00-8.00 normally consolidated range			
		Project	15.00			

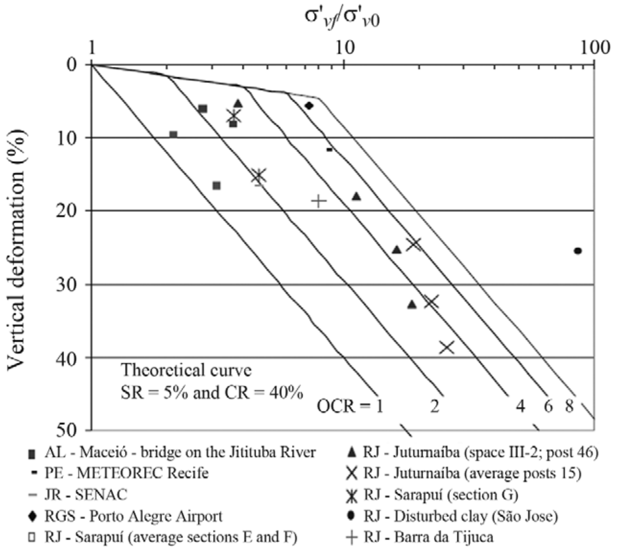


Figure 25 - Influence of the overconsolidation ratio on the magnitude of the settlements of several Brazilian embankments on soft soils (Cavalcante *et al.* 2003).

zontal deformation to be expressed as a ratio between maximum vertical displacement, S, measured in the center of embankment and maximum horizontal displacement y_m, measured in a vertical under the foot of the embankment.

According to the Ylight model (Tavenas & Leroueil 1980), during the construction of an embankment on overconsolidated soft soil foundation (OCR < 2.5), the effective stress path for a point at or near the centerline is O'P'A' (Fig. 2a). What is observed initially in the overconsolidated clay is drained consolidation near the Ko condition. In this condition, the horizontal displacements are much smaller than the settlements (preconsolidation compression - S_r) and in fact, all the deformation components are small because of the rigidity of the clay - O'P'. Based on field observations, Bourges & Mieussens (1979) and Tavenas *et al.* (1979) (see also Leroueil & Rowe 2000) proposed an empirical method to calculate horizontal displacements. It was observed that in cases of embankments with slopes of the order of 1.5 to 2.5 (H): 1 (V), this initial maximum horizontal displacement would be correlated with the maximum settlement (Fig. 26) using the equation:

$$y_{mr} = (0.18 \pm 0.09)\, S_r \tag{17}$$

When reaching the P' point in the path (Fig. 2a), the clay foundation becomes normally consolidated and is subjected to an approximately undrained state of plastic shear during which the horizontal displacements increase quickly, at the same rate as the settlements (P'A') until construction ends.

The statistical expressions proposed that relates the increase of horizontal displacements to the settlement (Fig. 27) is:

$$\Delta y_{mu} = (0.91 \pm 0.2)\, S_u \tag{18}$$

The total maximum horizontal displacement, y_m, at the end of construction will be represented by the addition of two parcels of displacement:

$$y_m = y_{mr} + y_{mu} \tag{19}$$

where y_{mr} is the lateral displacements during the preconsolidation compression, and y_{mu} is the lateral displacements during the following phase, when the soil is normally consolidated.

With the time, after construction ends, the effective stress increases following a path such as A'B' (Fig. 2a), with the settlements due to consolidation of the normally consolidated clay. Tavenas *et al.* (1979) concluded that the maximum horizontal displacement continues to increase linearly with the settlement yielding, for definitive conditions of geometry and stability, in:

$$\Delta y_m = (0.16 \pm 0.02)\, \Delta S \tag{20}$$

where ΔS corresponds to consolidation settlement "S_c".

The value of the ratio $\Delta y_m/\Delta S$ during consolidation can be a function of the width L or angle β of the embankment slope, of the thickness of clay (D or H) and of the fac-

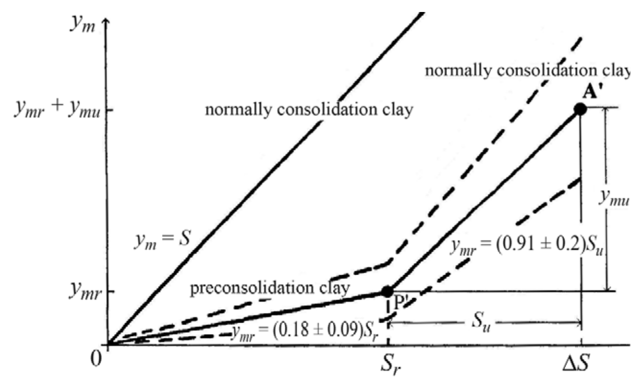


Figure 26 - Calculation of horizontal displacement in function of the settlement during construction (Tavenas *et al.* 1979).

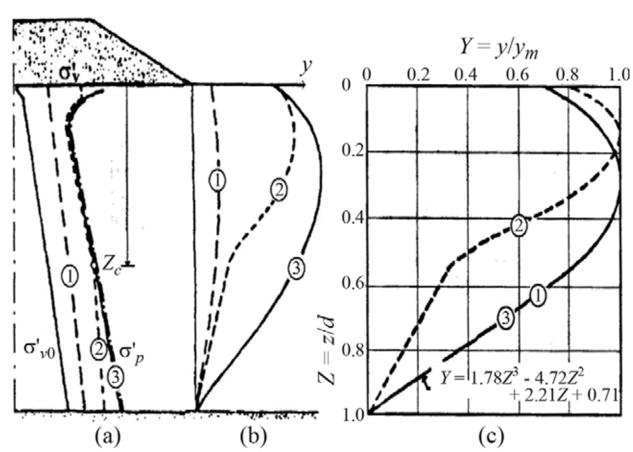


Figure 27 - Method of estimating the distribution of the horizontal deformation with the depth under the base of the embankment (Tavenas *et al.* 1979).

tor of safety, which limits the level of the shear stress at point A' (Fig. 2a).

The similarity between Eq. (20) and the corresponding one at the beginning of the construction (Eq. (17)) provides additional evidence of the drained nature of the response of the clayey foundation at the initial stage, according to the literature. The observed relationship shows as a general occurrence in the initial period of consolidation, that is, about 5 years later for investigated soils. For long periods of consolidation, the $\Delta y_m/\Delta S$ ratio can decrease by to about 1/3 of the observed value at the start of consolidation, that is, the $\Delta y_m/\Delta S$ ratio could also be a function of time.

The distribution of the horizontal displacement with the depth can be estimated using the relation between the normalized deformation, $Y = y/y_m$, and the relative depth $Z = z/D$, where "D" is the thickness of the clayey layer (Fig. 27). The empirical relation between Y and Z depends directly on the consolidation state of the foundation clay. During the initial period of construction, when all the clay is in an overconsolidated state, the deformation is of type 1, which corresponds to the classic solution of elasticity theory. If, during the final phase of construction, all clay layer moves to the normally consolidated state the deformation, when construction ends, reflects this final homogeneity final of the foundation soil (situation type 3, Fig. 27b). Bourges & Mieussens (1979) showed empirically that, in these cases, the normalized deformations are identical (Fig. 27c). The distribution of horizontal displacements when construction ends can be obtained using Eq. (21):

$$Y = 1.78\,Z^3 - 4.72\,Z^2 + 2.21\,Z + 0.71 \qquad (21)$$

where $Y = y/y_m$ and $y_m = y_{mr}$, and $Z = z/D$.

If only part of the soil foundation reaches the normal consolidated state during the construction, then the final deformations will reflect this heterogeneity with a form of type 2 (Fig. 26c - see Tavenas *et al.* 1979 for the equation).

The results of horizontal displacements observed in the Juturnaíba trial embankment were represented as indicated by Tavenas *et al.* (1979) and Bourges & Mieussens (1979) (Figs. 4, 28, 29 and 30). Analyzing the results, it can be observed that (Coutinho 1986):

• Figure 29 presents the horizontal displacements measured using the four inclinometers set up in one cross section. It is possible to see that the behavior observed is similar to I-1 and I-2 and shows some influence from the strength of the embankment. The I-3 and I-4 show the foundation deformation in a vertical under or near the foot of the embankment.

• The observed values (Fig. 29a) of y_m (foot of embankment) *vs.* S (center of embankment) showed three rectilinear intervals, instead of only two as indicated by Tavenas *et al.* (1979). The first and the third intervals showed values of $\Delta y_m/\Delta S$ in very good agreement with the Tavenas *et al.* proposal. The soft clay foundation under the trial embankment presents a shallow part (depth: 0-2.5 m) with OCR > 2.5 (Fig. 16) and the proposed method was developed for clays with OCR < 2.5. This characteristic of the soft deposit may be one reason for the different behavior.

• The inclinometers (under the embankment, I-1 and I-2) showed similar behavior for the relation of y_m *vs.* S, with the ratio $\Delta y_m/\Delta S$ presenting, in general, higher values than the reference inclinometer at the toe of embankment (Fig. 27b).

Analyzing the pore pressure and the increases in stress developed because of the construction of the em-

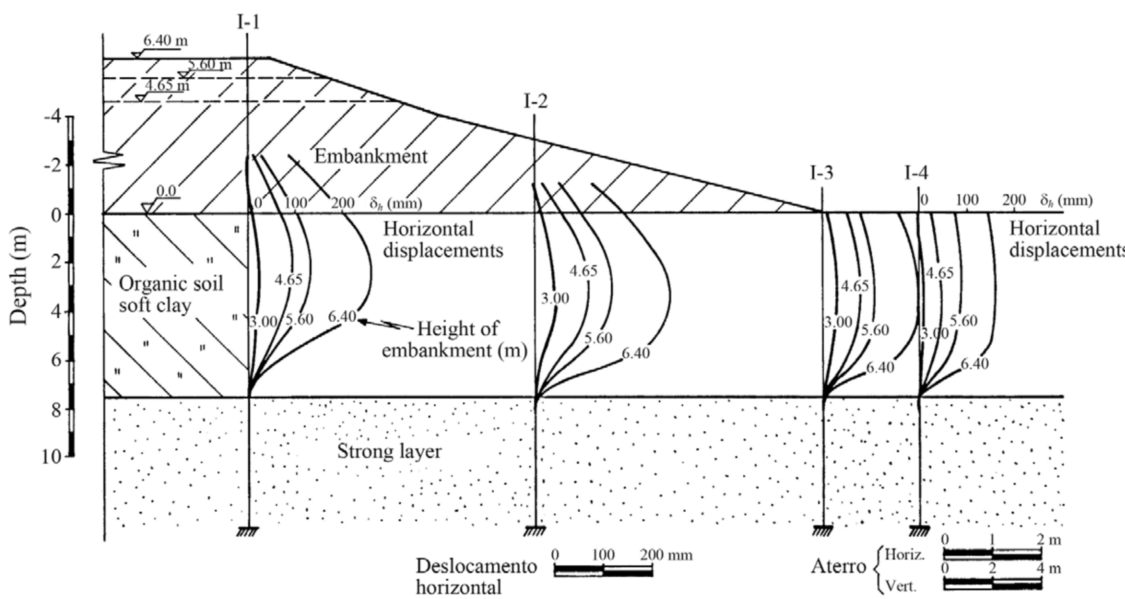


Figure 28 - Horizontal displacements measured for inclinometers - embankment with 3.00; 4.65; 5.60 and 6.40 heights, Juturnaíba trial embankment (Coutinho 1986).

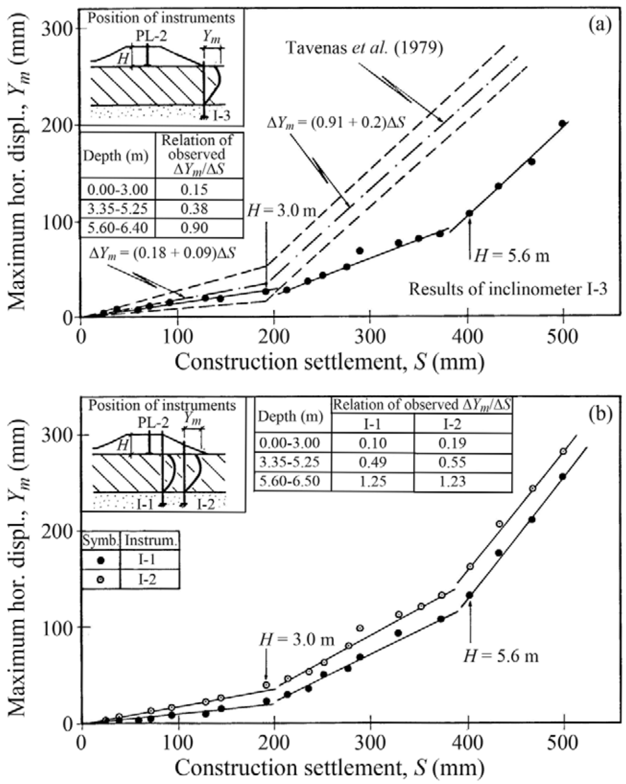


Figure 29 - Maximum horizontal displacement vs. settlement of the construction in the center of the Juturnaíba trial embankment: (a) inclinometer at the foot of the embankment and (b) inside the emban kment (Coutinho 1986).

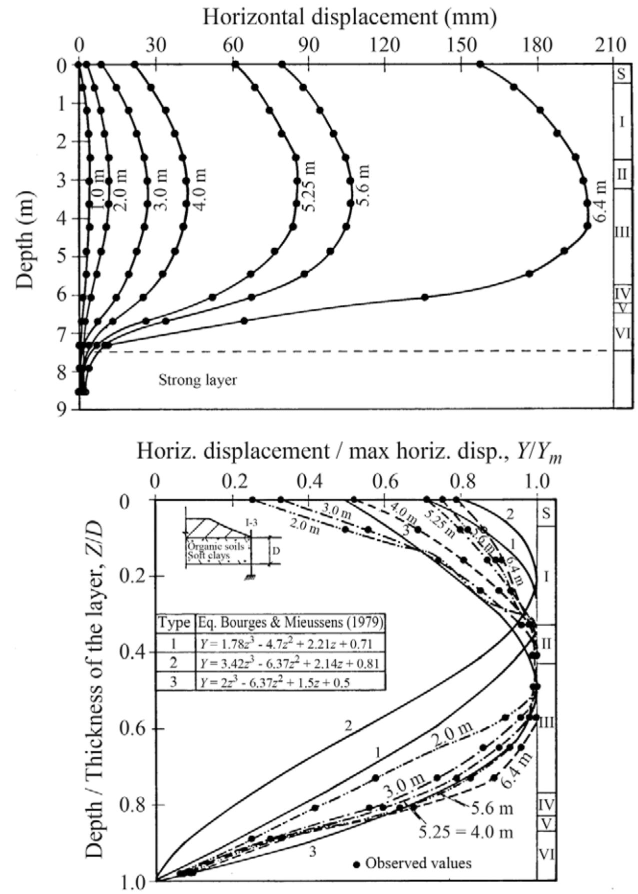


Figure 30 - Distribution of horizontal displacements with depth, foot of Juturnaíba embankment:(a) horizontal displacement; (b) horizontal displacement/maximum horizontal displacement (Coutinho 1986).

bankment, it was observed that for the part of the foundation with OCR > 2.5 (Fig. 16; 0-2.5 m depth), the σ'_{crit} values were smaller than σ'_p from the oedometer laboratory tests and tended to increase as the construction continued, reaching values similar to σ'_p for an embankment height of 5.6 m. For the part of the foundation with OCR < 2.5 (Fig. 16; 2.5-7.5 m depth), the σ'_{crit} values were in the same range as the σ'_p values, and remained constant in the same range while the embankment continued to be constructed. The condition $\Delta u \cong \Delta\sigma_v$ was obtained for an embankment height of 3.0 m for the foundation zone with OCR < 2.5 and for a height of 5.6 m in the foundation zone with OCR > 2.5. This condition has a strong influence on the horizontal displacements observed:

• The normalized distribution of the horizontal displacement with the corresponding depth along the vertical line under the foot of embankment, $y/y_m = f(z/D)$ seems to be in agreement with the Bourges & Mieussens' proposal (depending on the strength condition of the deposit), being stationary in the zones of the foundation that is normally consolidated. The good agreement with curve type 3 for the entire foundation occurred when the height of the embankment was between 3.0 and 4.0 m (Fig. 30).

• The existence of a relatively thick layer with OCR > 2.5 seems to be responsible for the differences observed between the predicted and the observed values, and for the variation of $y/y_m = f(Z/D)$ behavior during the construction.

In the Juturnaíba trial embankment it was possible to have different instruments in the same location to measure vertical and horizontal displacements (Fig. 4). Figure 31 shows the resultant displacements (vectors) at points of the foundation for different embankment heights. It can be seen that the tendency is for vectors to be displaced during construction basically until failure, showing an expected behavior.

Coutinho *et al.* (1994) and Lucena (1994) present an analysis of the horizontal displacement of the Juturnaíba Dam in which they show measured and predicted maximum horizontal displacement *vs.* settlement under the center of the embankment are presented (Fig. 32). Horizontal displacements were measured at the inclinometer set up at the slope-berm interface. During the construction stages, the values measured are in agreement with those proposed only up to an embankment height of 5.6 m, when the soft clay appeared to become normally consolidated. Above this height, the measured values are much lower than the

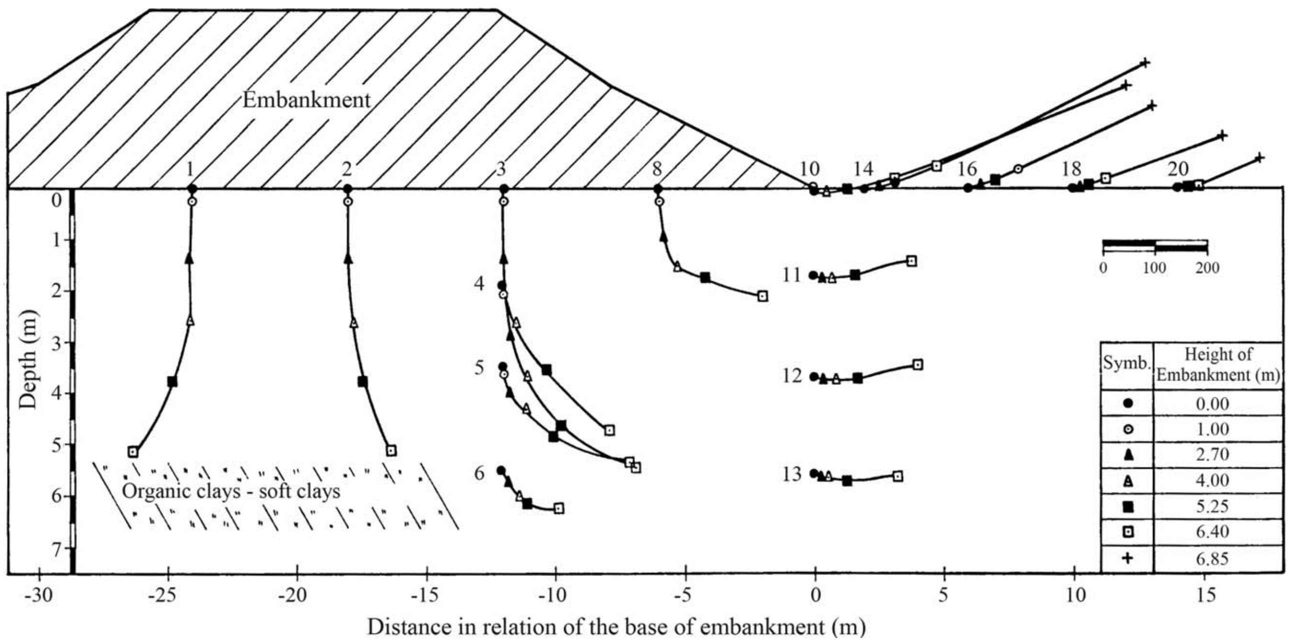


Figure 31 - Resultants (vertical + horizontal) displacements in points of foundation - Juturnaíba trial embankment (Coutinho 1986).

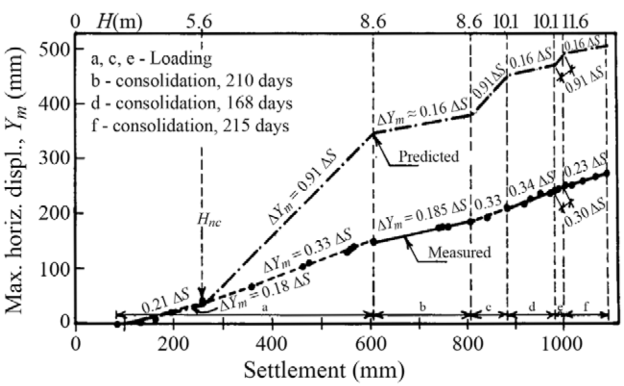


Figure 32 - Maximum Horizontal Displacements *vs.* Settlements - Juturnaíba Dam (Coutinho *et al.* 1994).

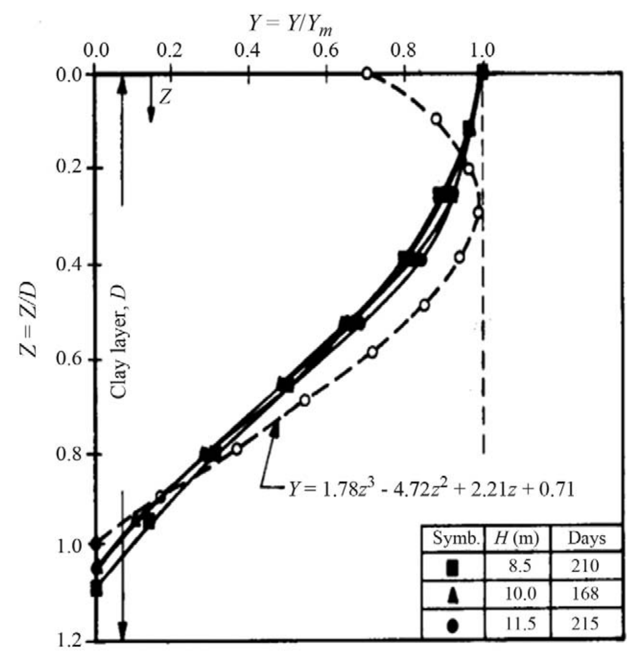


Figure 33 - Distribution of Horizontal Deformation with depth - Juturnaíba Dam (Coutinho *et al.* 1994).

predicted ones. The $\Delta Y_m/\Delta S$ measured values were in the range of 0.30-0.33 instead of 0.91 as predicted by the method. During the consolidation periods, the agreement is generally good, where the ratio $\Delta Y_m/\Delta S$ values are generally in the range of 0.18-0.23, close to the predicted values which average about 0.16. The results appear to show that partial drainage occurred during all the construction stages. One possible explanation for this behavior is the combination of the following factors: the large width of the embankment, the relatively small thickness of the compressible layer, and perhaps the location of the inclinometer.

The distribution of the lateral displacement with respect to depth depends directly on the consolidation state of the clay beneath the embankment. Figure 33 shows for Juturnaíba Dam the predicted (Tavenas *et al.* 1979) and measured $Y = f(Z)$ for the consolidation period under different loading stages, after the clay layer has become normally consolidated. As proposed by Tavenas *et al.* (1979), the distribution of lateral displacement regarding depth remained essentially unchanged with respect to time and construction stages. In addition, reasonable agreement between predicted and measured $Y = (Z)$ curves is also observed. The main discrepancy is the displacement at the top of the clay foundation.

Cavalcante (2001) and Cavalcante *et al.* (2003 and 2004) performed horizontal displacement analysis and stability control in the Jitituba embankment. The main purpose of following up horizontal displacements was to check the stability of the foundation soil during and after the construction of the access embankments of the Jitituba River Bridge, in order to ensure these displacements were maintained within safe limits and that they presented the least possible minimal values, because of proximity of piles of the bridge foundation. Figures 34 and 35 present measured and predicted maximum horizontal displacement *vs.* settlement under the center of the embankment and the normalized profile of the horizontal displacements with the depth, respectively.

It is observed in Fig. 34 that the ratio $\Delta Y_{max}/\Delta S$ presented in general values in all the construction and consolidation phases (except in the phase of the consolidation for inclinometer I-01) that were lower than the values proposed by Tavenas *et al.* (1979). During the construction phases, the $\Delta Y_{max}/\Delta S$ values were between 0.08 and 0.46, indicating a range of values correspondent to predominantly partially drained condition during construction. In the consolidation phases, it is observed that the values presented a reasonable range within 0.04 and 0.22, basically in the range proposed by Tavenas *et al.* (1979). This behavior can be due to the use of vertical drains, which accelerates consolidation as well as the increase of soil strength and/or to a possible overconsolidated state during the first construction phase.

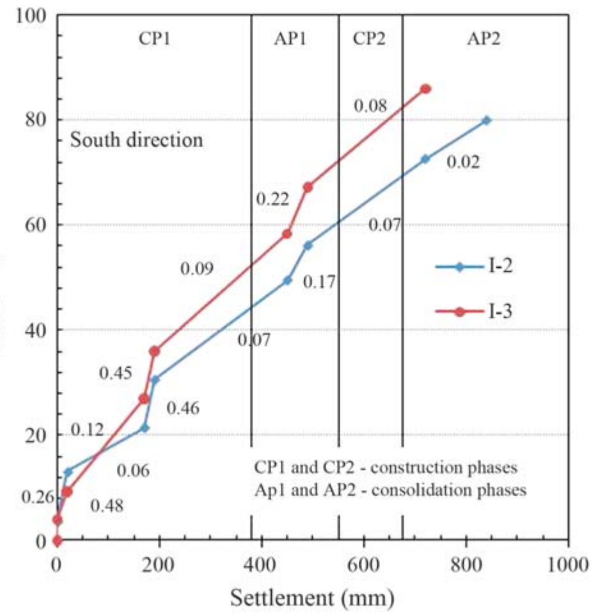


Figure 34 - Graphics Y_{max} (mm) *vs.* S (m) - Check of the ratio $\Delta Y_{max}/\Delta S$ proposed by Tavenas *et al.* (1979) Jitituba embankment (Cavalcante e*t al.* 2004).

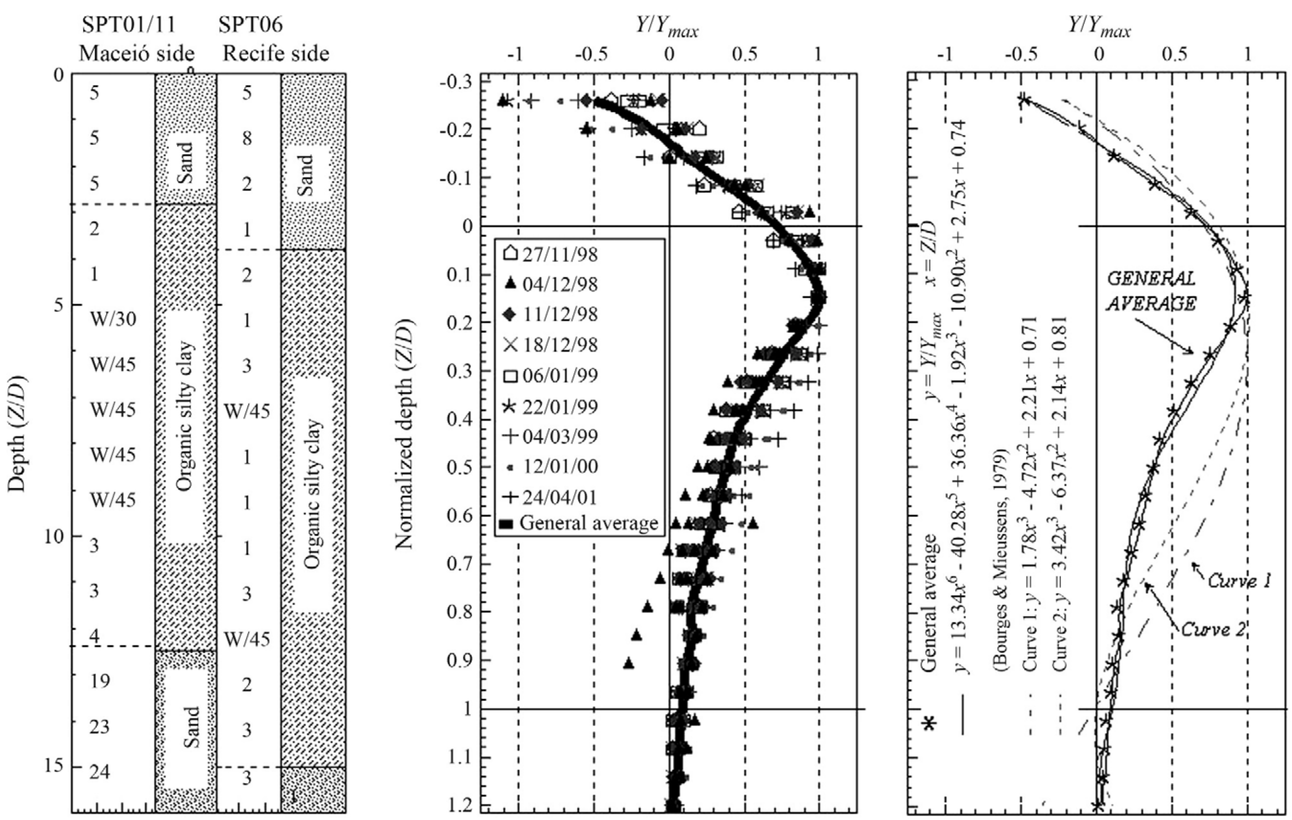


Figure 35 - Comparison between the average of the profiles normalized with the curves proposed by Bourges & Mieussens (1979) - Jitituba embankment (Cavalcante *et al.* 2004).

6. Final Comments and Conclusions

This paper presented and discussed concepts and results of monitoring and the performance of an embankment on soft soil deposits. Research and practical cases were used in the paper with emphasis on a well-instrumented and extensive study performed on the Juturnaíba trial embankment. The conventional ("undrained condition during construction") and the Tavenas & Leroueil models ("partial drainage during construction") were used in the analysis of the behavior of the embankments. In general, all the embankments studied presented partial drainage during construction, thus showing that actual behavior can be more complicated and that the conventional undrained analysis overpredicts pore pressure and horizontal displacements.

For the well-instrumented Juturnaíba trial embankment, in principle, the Tavenas & Leroueil model presented good results. Due to the specific soil foundation conditions of the area with the presence of organic soil layers and with part (~30%) of the deposit with OCR > 2.5, the effective stress path and the behavior observed showed more partial drainage and an "intermediate" interval in the behavior of pore pressure and lateral displacements during construction / failure. This case can be as an "extension of the Tavenas & Leroueil model proposed for the cases where the foundation deposit has a significant part with OCR > 2.5 but most of the soft deposit has OCR < 2.5.

Above an embankment height of 5.60 m (FS = 1.31), the foundation behavior (pore pressure, vertical and horizontal displacements) changes significantly showing the beginning of a possible process of failure, which occurred shortly after with H_{emb} = 6.85 m. This result confirms the importance of having a factor of safety in a project higher than 1.3-1.4, as indicated in the literature.

The cases studied in this paper show how complicated actual embankment behavior can be but they also point out to the possibility of predicting embankment behavior, depending on the condition of the foundation deposit, embankment geometry, location and type of instrument, etc. The importance of monitoring and evaluating each case based on an appropriate model is fundamental for the success of a project.

Acknowledgments

The authors are indebted to CNPq and CAPES for their financial support. Special thanks to Sarita Cavalcante and Maria Helena Lucena who contributed with part of the data used in this paper, and to Gusmão Associated Engineers for the partnership of in this project.

References

Almeida, M.S.S. (1996) Aterros Sobre Solos Moles: Da Concepção à Avaliação do Desempenho. Editora UFRJ, Rio de Janeiro, 216 pp.

Asaoka, A. (1978) Observational procedure of settlement prediction. Japanese Society of Soil Mechanics and Foundation Engineering, v. 18:4, p.87-101.

Barron, R.A. (1948) Consolidation of Fine-Grained Soils by Drain Well. Journal of the Geotechnical Engineering Division, ASCE, Transactions, paper 2346, v. 113, pp. 718-742.

Bjerrum L. (1973) Problems of soil mechanics and construction of soft clays and structurally unstable soils. Proc. of the 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.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. (In Portuguese).

Cavalcante, S.P.P.; Coutinho, R.Q. & Gusmão, A.D. (2003) Evaluation and control of stability of embankments 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. Proc. 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 da Barragem de Juturnaíba. D.Sc. Thesis, COPPE/Universidade Federal do Rio de Janeiro, Rio de Janeiro, 634 pp. (In Portuguese).

Coutinho, R.Q. & Lacerda, W.A. (1987) Characterization-consolidation of Juturnaíba organic clays. Proc. ISGESS, Mexico City, 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.; Almeida, M.S.S. & Borges, J.B. (1994) Analysis of the Juturnaíba embankment dam built on soft clay. Vertical and horizontal deformations of foundations and embankments. Proc. of Settlement 94, Texas, v. 1, pp. 348-363.

Coutinho, R.Q.; Oliveira, J.T.R. & Oliveira, A.T.J. (1998) Estudo quantitativo da qualidade de amostras de argilas moles brasileiras - Recife e Rio de Janeiro. Proc. XI COBRAMSEG - Congresso Brasileiro de Mecânica dos Solos e Engenharia Geotécnica, Brasília, v. 2. pp. 927-936.

Coutinho, R.Q. & Bello, M.I.M. (2005) Geotecnia do Nordeste. ABMS / Núcleo Nordeste, Recife, Editora Universitária / UFPE, v. 3, p. 111-153.

Coutinho, R.Q. & Bello, M.I.M. (2010) Analysis and control of the stability of embankments on soft soil: Juturnaíba and other experiences in Brazil. Proc. Symp. of New Techniques for Design and Construction in Soft Clays, Guarujá, pp. 247-268.

D'Apolonia, D.J.; Poulos, H.G. & Ladd, C.C. (1971) Initial Settlement of Structures on Clay. ASCE, JSMFE, No. 10 0, pp. 1359-1377.

Folkes, D.J. & Crooks, J.H.A. (1985) Effective stress paths and yielding in soft clays below embankments. Canadian Geotechnical Journal, v. 22: 3, p. 357-374.

Foot, R. & Ladd, C.C. (1981) Undrained settlement of plastic and organic clays. ASCE, JTGED, v. 107:GT8, p. 1079-1094.

Leonards, G.A. & Altschaeffl, H.G. (1964) Compressibility of Clay. Journal of the Soil Mechanics and Foundation Division, Transactions of ASCE, v. 90:5, p. 133-155.

Leroueil, S.; Tavenas, F.; Mieussens, C. & Peignaud, M. (1978) Construction pore pressures in clay foundation under embankments. Part II: Generalized behaviour. Canadian Geot. Journal, v. 16:1, p. 66-82.

Leroueil, S. & Tavenas, F. (1986) Discussion on "Effective stress paths and yielding in soft clays below embankments". Canadian Geotechnical Journal, v. 23:3, p. 410-413.

Leroueil, S. (1996) Compressibility of clays: Fundamental and practical aspects. Journal of Geot. Eng. ASCE, v. 122:7, p. 534-543.

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, pp. 463-498.

Lucena, M.H.L. (1994) Análise do Comportamento da Fundação da Barragem de Juturnaíba - Techos III-2 e V. M.Sc. Thesis, Universidade Federal de Pernambuco, Recife, 219 pp (In Portuguese).

Massad, F. (1988) Baixada Santista: Aterros sobre solos moles interpretados à luz dos novos conhecimentos sobre a gênese das argilas quaternárias. Proc. Simp. Depósitos Quaternários das Baixadas Litorâneas Brasileiras, São Paulo, v. 1. pp. 5.1-5.10.

Massad, F. (1999) Baixada Santista: Implicações da história geológica no projeto de fundações. Solos e Rochas, v. 22:1, p. 3-49.

Magnan, J.P. & Deroy, J.M. (1980) Analyse graphique des tassement observés sons les ouvrages. Bull. Liaison Laboratoire des Ponts e Chaussés, 109, p. 9-21.

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).

Martins, I.S.M. & Lacerda, W.A. (1985) A theory of consolidation with secondary compression. Proc. 11th ICSMFE, San Francisco, p. 567-570.

Morin, P.; Leroueil, S. & Sampson, L. (1983) Preconsolidation pressure of champlain clays. Part. I. In-situ determination. Canadian Geotechnical Journal, v. 20:4, p. 782-802.

Orleach, P. (1983) Techniques to Evaluate the Field Performance of Vertical Drains. Msc. Thesis, MIT, Cambridge.

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).

Pinto, C.S. (1992) Primeira conferência Pacheco Silva: Tópicos de contribuição de Pacheco Silva e considerações sobre a resistência não drenada as argilas. Solos e Rochas, São Paulo, v.15:2, p. 49-87.

Rémy, J.P.P.; Martins, I.S.M.; Santa Maria, P.E.L.; Aguiar, V.N. & Andrade, M.E.S. (2010a) The embraport pilot embankment - Primary and secondary consolidations of Santos soft clay with and without wick drains - Part 1. Proc. Symp. of New Techniques for Design and Construction in Soft Clays, Guarujá, pp. 289-310.

Rémy, J.P.P.; Martins, I.S.M.; Santa Maria, P.E.L.; Aguiar, V.N. & Andrade, M.E.S. (2010b) The Embraport pilot embankment - Primary and secondary consolidations of Santos soft clay with and without wick drains - Part 2. Proc. Symp. of New Techniques for Design and Construction in Soft Clays, Guarujá, pp. 311-330.

Tavenas, F.; Mieussens, C. & Bougens, F. (1979) Lateral displacements in clay foundations under embankments. Canadian Geotechnical Journal, v. 16:3, p. 532-550.

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

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