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Analysis of Behavior of Embankments on Soft Soils Geotechnical Investigations and Instrumentation Access of Embank ments The Jitituba River Bridge

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ABSTRACT

The present work will be analyzing the access embankments of the Jitituba River Bridge, placed on the Highway AL- 413, km 12,9 – São Luiz do Quintude, a small city next to Maceió, AL, Brazil, where this bridge was built, even before the access embankments. Due to the presence of a soft soil thick layer, that measures up to 12 meters of thickness, and to the construction sequence of the bridge embankments, there was a need to analyze not only its stability but also its general rupture and its vertical displacements, commonly considered in the projects of embankments of soft soils, as well as the stability to reduce the horizontal displacements and the consequent efforts to transverse the piles through the bridge.

The solution adopted consisted in the construction of embankments by stages, allied to the use of prefabricated vertical drains and geotechnical instrumentation (Casagrande piezometers, plates of settlement an inclinometers) to control and follow the project performance. The geotechnical investigations consisted on field tests (standard penetration test SPT with natural water content profile and vane test) and lab tests (characterization, radial and vertical consolidation, compression triaxial UU and CIU).

Starting from the project data given by Gusmão Associated Engineers and the geotechnical investigations and instrumentation, the behavior of the access embankments was analyzed in relation to the measures of pore-pressures, vertical and horizontal displacements, through the application of several methods and models proposed in the bibliography and of the comparison with other cases of embankments on soft soils. Standing out, the analyses of consolidation, compressibility and control of stability.

1. INTRODUCTION

The development of sites underlain by thick deposits of soft soil has turned to be more and more common in the cities placed at the Brazilian Lowlands. That occupation happens in many different applications such as foundations of buildings, embankments of highways, airports, dams, urbanization areas, etc). The study of those embankments behavior has been approached for several national authors (ORTIGÃO, 1980; COUTINHO, 1986; BORGES, 19991; COUTINHO ET AL., 1994; LUCENA, 1997; ALMEIDA, 1996; GUSMÃO Fº. ET AL., 1998; NACCI, 2000; SCHINAID & NACCI, 2000; SPOTTI, 2000; ALMEIDA ET AL., 2001) and international ones (TAVENAS & LEROUIEL, 1980; MAGNAN & DEROY, 1980; BOURGES & MIEUSSENS, 1979; LEROUIEL ET AL., 1990; MESRI ET AL., 1994), accumulating experiences for a better understanding of the soft under solicitation of load increases.

In general, the project of embankments on soft soils should attend the basic requirements of stability against rupture and the settlement for consolidation, during and after the construction, compatible with its objective. In the case of access embankments of a bridge on soft soils, it is recommended to build it before the execution and the foundation of the bridge. Otherwise, it implicates in additional restrictions, especially about its stability, the horizontal displacements of the foundation soil and the bridge's infrastructure interference, once these requirements can be an important external load, and not always they are considered.

In the present work, the analysis of the access embankment behavior of the bridge on Jitituba River will be presented, in relation to the pore-pressures and to the vertical and horizontal displacements, through the application of several methods and models of consolidation and stability analysis proposed in the bibliography.

2. CASE STUDY

The Figure 1 presents the longitudinal section of the basic project, vertical stratigraph, and location of the geotechnical field investigations and instrumentation. The characteristics of each element and the steps of the construction will be following described:

- Bridge: It has 4 empty spaces of 24 m. and two balances of 6 m, and a total extension of 108 meters. Its structure is leaned on in 10 blocks of 4 metallic piles type 4 TR-32, with an average length of 22 meters.
- Embankment: Height initially esteemed of 6,4 meters (to reach the quota of 98,00 m in relation to the average quota in a natural terrain which is 91,60 m) higher width of 11,00m, slopes with inclination of 2V/3H, and unit weight of 18 kN/m^3.
- Soil Foundation: Geology common to the region of valleys with sediment of Holocene Period. Through standard penetration tests SPT was identified a thick layer of soft organic soil (SPT<1) until 12 meters of thickness and a water level approximately equal to the terrain level.
- Constructive Sequence: Due limitations in the budget, the whole upper structure of the bridge was built before the execution of the access embankments.

The main restriction in the project is related to the interference of the access embankments construction in the foundation piles, due its sequence.

In the analysis done by Gusmão Engineers Associates (1998), it was necessary more than the stability analysis and consolidation preview, so it was done also an internal soil stability study in order to hindrance any significant horizontal displacement (MARCHE & LACROIX, 1972), consequently, to minimize the transverse effects in the piles.

The design solution for that problem consisted in a construction of the embankment in 2 stages, allied to the use of prefabricated vertical drains (triangular spacing of 1,00 m), so the time of consolidation was compatible with the construction programming (60 days each stage).

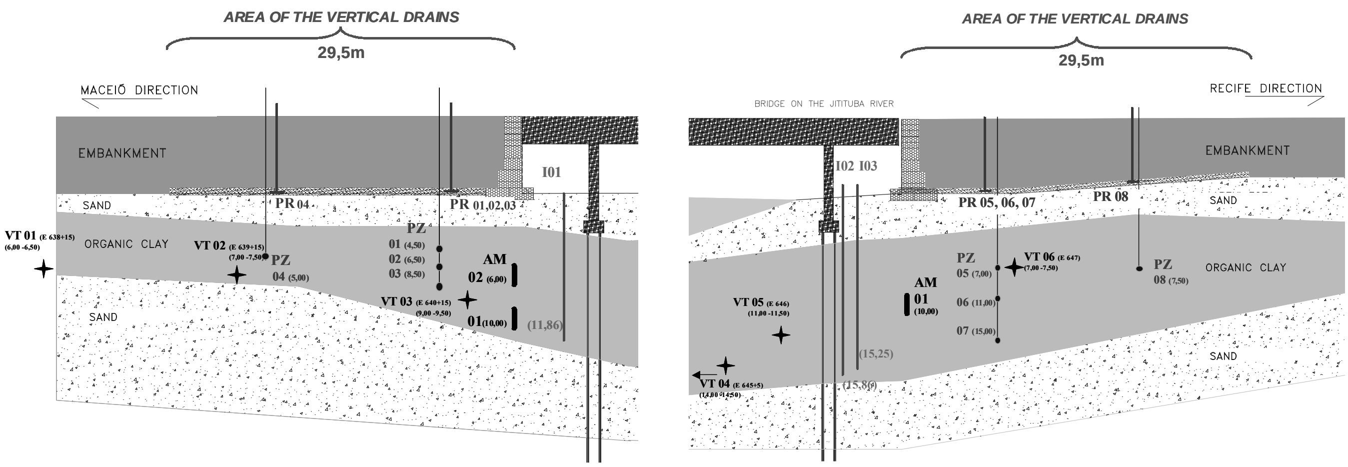
Considering the importance of this construction and the peculiar problems involved, it was part of the design geotechnical instrumentation to follow up pore-pressures, vertical and horizontal displacements.

3. GEOTECHNICAL INVESTIGATIONS

The geotechnical investigations consisted on field tests and on laboratory tests and the controlling investigations of the project consisted on the instrumentation. The field investigations were standard penetration tests SPT (8 preliminary holes in the axis and 18 holes complement, obtaining the natural water content), undisturbed samples shelby´s 4" diameter (03 samples) and vane tests (06 holes). The lab investigations consisted on characterization tests, consolidation (vertical and radial) and resistance (triaxials UU and CIU). The geotechnical instrumentation was Casagrande piezometers, plates of settlement and inclinometers, with the pore-pressures, vertical and horizontal displacements, respectively. The field location, the geotechnical instrumentation and its profiles are presenting on the longitudinal section (Figure 1).

The characterization tests classified the soil as silty organic clay and presented the following medium parameters: liquidity limit W_L~W, plasticity limit W_P–35%, plasticity index I_P–45%, natural water content W-80%, unit weight γ~14 kN/me. A good agreement was observed between the values or W, obtained through the undisturbed samples (Shelby´s of 4") and by SPT soundings, similar the ones observed by COUTINHO ET AL. (2001).

In the oedemeter tests of vertical and radial consolidation were obtained, respectively, compression ratio CR – 22%, swell ratio SR – 7%, initial void index e_0 ~2 and over consolidation stress σ'_{VM}, this last one was presented smaller than effective



(a) Maceió side - South Direction

(b) Recife side – North Direction

Figure 1 - Longitudinal section of the basic project of the access embankments of the bridge on Jitituba River, including the geotechnical profile and the field investigations location.

vertical stress σ'_{V0} indicating a behavior unconsolidated. The quality of the samples was quantified in agreement with the proposal of LUNNE ET AL. (1997), presenting a low quality of the samples, with the values of $\varepsilon_{\sigma'V0}$ varying from 11,7 to 18,8% (from poor to very poor), what justified the low σ'_{VM} and CR values found. The consolidation coefficients of vertical and radial consolidation where presented the average values of c_v= 2,5x10^{-8}m^2/s and of c_h=6,0x10^{-8}m^2/s, in the interval normally consolidated, and of c_v=10,0 x10^{-8}m^2/s and of c_h= 35,0x10^{-8}m^2/s, in the interval pre-consolidated.

The undrained strength for the tests triaxials UU was presented lower (average of 9 kPa) to obtained it in the vane test in the natural soil (average of 35 kPa), and very close to the vane tests with the disturbed soil (average of 6 kPa), evidencing the disturbance effects in the lab samples. The internal friction angle (triaxial test CIU) presented an average value of 23°. Due to the peculiarity and importance of the construction as well as the need of a good operation and a security construction of adjacent structures, it was recommended the geotechnical instrumentation through the measures of pore-pressures, vertical and horizontal displacements. The geothecnical instrumentation was composed of 8 plates of settlement, 8 Casagrande piezometers and 3 inclinometers. The Figure 1 shows the location in the longitudinal section of the geotechnical instrumentation accomplished.

The Figure 2 presents the results of the geotechnical instrumentation obtained in the first 140 days. After this, the construction period was paralyzed because we were out of money. Later on, the pavement was executed and the operation of the bridge was liberated, and only the measures of the horizontal displacements could be accomplished.

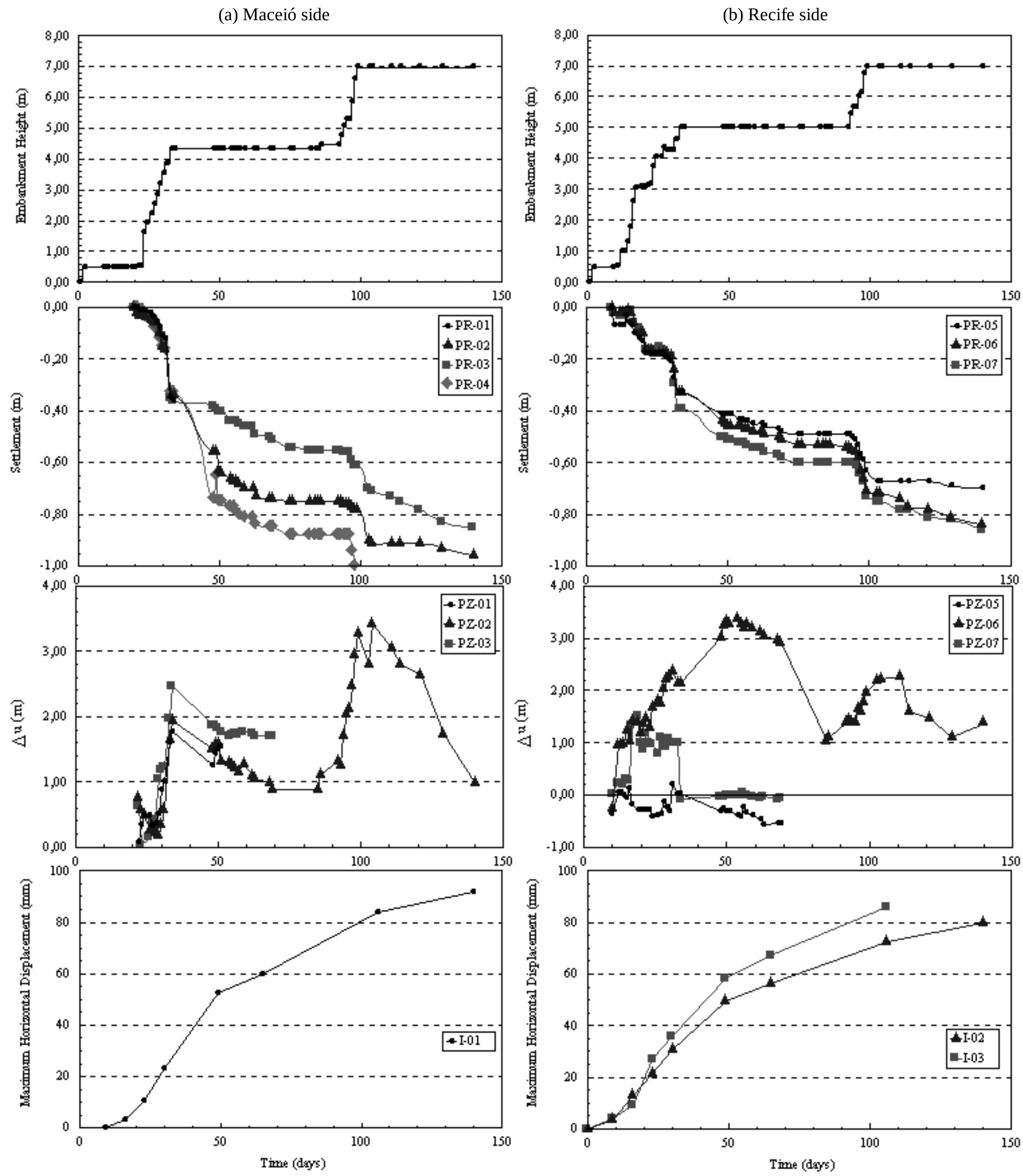


Figure 2 - Results of the geotechnical instrumentation mensurations - embankments of access of the Bridge on Jitituba River.

4. ANALYSIS OF THE VERTICAL DISPLACEMENTS AND PORE-PRESSURES

4.1. Pore-Pressure

The main objective to the follow up of pore-pressures in the embankments of the Jitituba river bridge was to evaluate the efficiency of the adopted solution to accelerate the settlements and dissipate the pore-pressures, that is to say the pre-fabricated vertical drains.

The long time delay of the Casagrande piezometer allied the variations of the water level makes the analysis of the piezometry more difficult. Despite of this, the results obtained in piezometers were evaluated in relation to the reply to stress increase and the dissipation of pore-pressures with time, for the piezometers PZ-02 (Maceió side) and PZ-06 (Recife side) (Figure 5).

The measure analysis of pore-pressures consisted on the reply to the stress increase, according to the proposal of TAVENAS &LEROUEIL (1980), and on the coefficient of horizontal consolidation, according to the proposal of ORLEACH (1983). In order to minimize, the effect of the Casagrande limitations, it was applied in the analysis of reply to the stress increase, the correction of the measures of pore-pressures of the Casagrande piezometer proposal for COUTINHO (1986), regarding to the difference between the Casagrande piezometer and the Pneumatic $\Delta u_{corrected}=\Delta u_{pneumatic}=\Delta u_{casagrande}/0,75$.

4.1.1. Reply to the Stress Increase

The Figure 3 presents the graph $\Delta\sigma'_V$ versus Δu and $\Delta u_{corrected}$ for the piezometer PZ-02 (Maceió side), relating the stress increase to the pore-pressure increases measured and

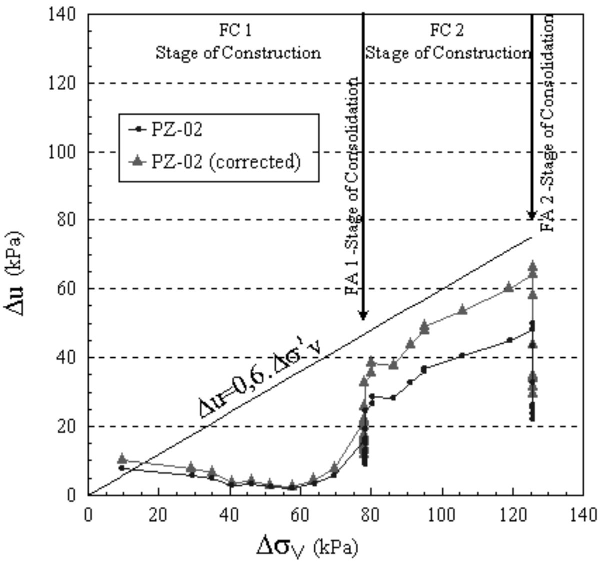


Figure 3 - Pore-pressure increase (Δu) versus stress increase ($\Delta\sigma'_V$), Maceió side (CAVALCANTE, 2001).

Table 1 – Reply of the Pore-pressures

	Side	Maceió PZ-02	Recife PZ-06
1ª Stage	Δu (kPa)	12 kPa	34 kPa
	$\Delta\sigma_V$ (kPa)	69 kPa	82 kPa
	$\Delta u/\Delta\sigma_V$	**0,17**	**0,41**
	$\Delta u_{corrig}/\Delta\sigma_V$	**0,23**	**0,55**
2ª Stage	Δu (kPa)	21 kPa	13 kPa
	$\Delta\sigma_V$ (kPa)	48 kPa	35 kPa
	$\Delta u/\Delta\sigma_V$	**0,43**	**0,36**
	$\Delta u_{corrig}/\Delta\sigma_V$	**0,58**	**0,48**

corrected. The Table 1 presents the values of Δu, $\Delta\sigma'_V$ and of $\Delta u/\Delta\sigma'_V$ for both stages of the construction, including the relation of $\Delta u_{corrected}/\Delta\sigma'_V$, according to the proposal for Δu of COUTINHO (1986).

It is observed that both relations of $\Delta u/\Delta\sigma'_V$ and of $\Delta u_{corrected}/\Delta\sigma'_V$ had presented a compatible maximum value of 0,58 partially with a drained behavior during the construction ($\Delta u/\Delta\sigma'_V < 0,6$) (TAVENAS & LEROUEIL, 1980) that is due to the possible overconsolidating behavior (high c_v or c_h) in the beginning of the construction, an/or to the use of the vertical drains to accelerate the settlements, therefore both cause a fast dissipation of the generated pore-pressures.

4.1.2. Evaluation of Consolidation with the Time – ORLEACH (1983) Method

The ORLEACH (1983) method was applied to obtained the coefficient of radial consolidation for the piezometers PZ –02 (Maceió side) and PZ –06 (Recife side), by plotting of the graphics Log (Δu) x t and the obtained equations in the radial drainage:

$$c_h = \frac{F(n) \cdot d_e^2 \cdot \alpha_1}{8} \qquad \text{(I)}$$

Where:

- $\alpha_i =$ inclination interval consolidation obtained in graphic Log (Δu) x t (Figure 4)
- $d_e = 1{,}05 \cdot l = 105cm$ for triangular grid
- $l = 100cm$ spacing between the drains
- $F(n) = \ln(n) - 0{,}75 = 2{,}04 \qquad n = d_e/d_w = 16$

The Figure 4 presents the graphic construction of the ORLEACH (1983) method for the piezometer PZ-02 (Maceió side). The Table 2 presents the results obtained. It is observed that the Recife side presents smaller consolidation coefficients than the Maceió side, 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 lab average (interval normally consolidated.) they came 1,04 and 1,50 times higher to Maceió and Recife sides, respectively. When compared to the value obtained in the project, they came lower.

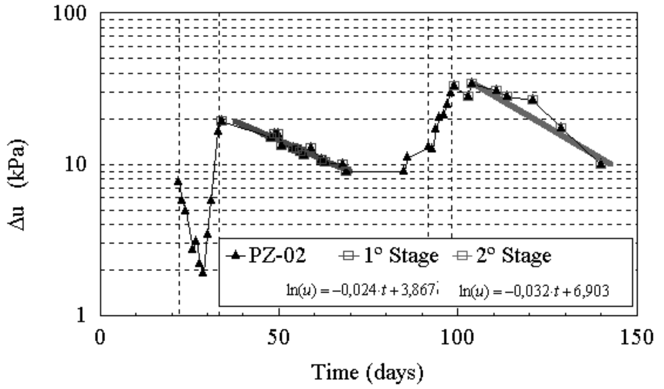


Figure 4 - Graphics Log (Δu) x time - Application of the ORLEACH (1983) method, Maceió side (CAVALCANTE, 2001).

Table 2 - Consolidation coefficients obtained in the ORLEACH (1983) method, in laboratory and adopted in the project.

		1ª Stage		2ª Stage	
	Side	α_1	c_h $10^{-8}.m^2/s$	α_1	c_h $10^{-8}.m^2/s$
ORLEACH (1983)	*Maceió*	0,024	7,65	0,032	10,29
	Recife	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**

It is important to mention that the samples used in the lab presented low quality, what interfered in the results of the consolidation parameters, decreasing c_v and σ'_{VM} (COUTINHO ET AL., 1998). In spite of the mechanical limitations (time of reply) of the Casagrande piezometers, the application of the ORLEACH (1983) method presented reasonable results faced with the lab results with poor quality samples.

4.2. Vertical Displacements

4.2.1. Evaluation of the Consolidation with the Time - ASAOKA (1978) Method

The interpretation of the measure soil in the vertical displacements in field (Figure 2) was done through the ASAOKA (1978) method, modified by MAGNAN & DEROY (1980).

They were esteemed through the ASAOKA (1978) method, the maximum settlement and the consolidation coefficient in the field, considering the period of the curve settlement versus time, after the end of the construction of each stage (consolidation phase). The total time considered in the evaluation of settlements of the first and the second stage was equal to 56 and 40 days, respectively. Due to the use of vertical drains to accelerate the settlements and the deformation high-speed, in which an interval of 5 days was used.

The Figure 5 presents the application of the graphic construction of ASAOKA (1978), for one of the settlement plates, PR-06 (Recife side). For the period of observed measures, the consolidation degree for all the plates was higher than 94%, indicating the stabilization of the settlements.

The formulation used for obtain it went to proposed the drainage purely radial (MAGNAN & DEROY, 1980)

$$c_h = -\frac{d_e^2 \cdot F(n)}{8} \cdot \frac{\ln \beta_1}{\Delta t} \qquad \text{(II)}$$

Where, $\beta_i =$ inclination of the straight line, was obtained. (In the graphic construction by ASAOKA (1978) (Figure 5).

The Table 3 presents the average results of coefficient radial consolidation obtained through the application of the ASAOKA (1978) method modified by MAGNAN & LEROY (1980), together with the consolidation coefficients obtained through the ORLEACH (1983) method, in the lab tests and considered in the project.

The coefficient of horizontal consolidation obtained by the ASAOKA (1978) method was presented higher on the Recife side in relation to the Maceió side around 26%. With relation to the to the obtained in laboratory, these were presented highers about 2,6 to 7,7 times. The possible causes for this difference are: (a) the interference of the secondary consolidation during the primary, what does with that results of ASAOKA (1978) method are overestimated; (b) the disturbance effects in the compressibility parameters and consolidation, that it induces to σ'_{VM} and c_h

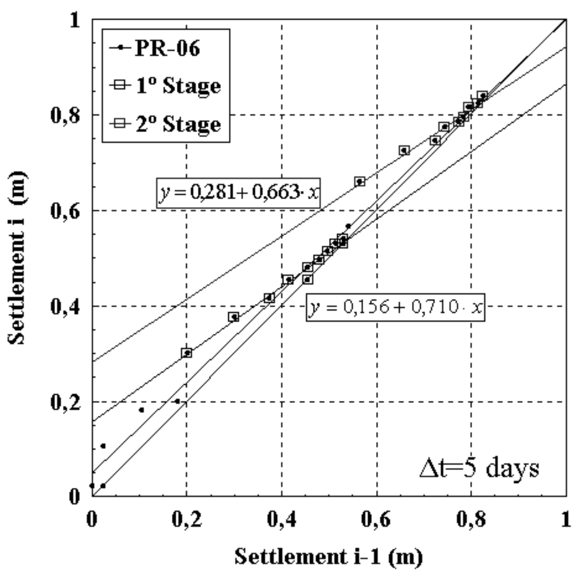


Figure 5 - Application of the Asaoka graphic construction, plate PR-06 - Recife side.

Table 3 - Comparison between the coefficients of horizontal consolidation obtained through field mensurations, of laboratory tests and adopted in project.

		Side	Maceió		Recife	
Stress Increase		Stage	1º	2º	1º	2º
		Hemb end (m)	4,34	6,98	5,03	6,96
		σ'_{V0i} (kPa)	41,80	119,92	52,80	143,34
		$\Delta\sigma_{Vi}$ (kPa)	78,12	47,52	90,54	34,74
		$\Delta\sigma_{Vi}$ /σ'_{V0i}	1,87	0,40	1,71	0,24
c_h ($10^{-8}.m^2/s$)	FIELD	**ASAOKA (1978)**	**20,55**	**24,34**	**26,32**	**30,80**
		ORLEACH (1983)	**7,65**	**10,29**	**2,51**	**9,98**
		LABORATORY	**4,00 - 8,00 (Normally consolidated.)**			
		PROJECT	**15,00**			

underestimated in laboratory, as having observed by COUTINHO ET AL (1998).

The difference between the results obtained in the ASAOKA (1978) method and in the lab (2,7 to 7,7, higher) is higher to the difference of the results obtained on the field between ASAOKA (1978) and ORLEACH (1983) (2,3, to 3,1 times higher), what is probably due to the disturbance effects of the lab samples.

In both methods, the consolidation coefficient was presented higher in the second stage, about 1,18 and 2,65 times, respectively. Some possible reasons is the low stress increment ratio, which brings a larger participation of the secondary settlement in the total one. (MARTINS & LACERDA, 1985) and /or this is not sufficient to exceed a certain overconsolidation, caused for the secondary consolidation previous stress stage. (LEONARDS & ALTSCHAEFL, 1964; BJERRUM, 1967).

4.2.2. The Analysis of the Behavior Stress – Deformation (Compressibility)

In the analysis of the behavior stress-deformation, it is observed that the estimate settlements through the lab parameters (considering the soil normally consolidated) were presented about 30% to 60% highers to the maximum values measured in the field (Table 4). A possible and main reason for this differences it is the disturbance effects in the lab parameters.

The behavior stress-deformation obtained in the lab was compared graphically and in the field (plates of settlement located in the embankments center) being obtained the σ'_{VF} values in field through the difference between the estimate total stresses the measure of the pore-pressure in the field. The Figure 6 presents the comparison to the Recife side (PR-06).

At the presented curve can be observed the disturbance effects in the lab samples, through the deformation corresponding to the initial effective stress σ'_{V0} (value around 14%) and the comparison of the overconsolidation stress that was presented lower to the initial effective stress, as if deposit was unconsolidated.

A similar analysis to the proposed by MASSAD (1988 and 1999), for the embankments on soft soils of the Santos depression in São Paulo, for the results of instrumentation of settlements divulged by several Brazilian authors (COUTINHO ET AL. 2001; ALMEIDA ET AL., 2001, SCHINAID & NACCI, 2000; ALMEIDA & TERRA, 1988; LUCENA, 1997; COUTINHO ET AL., 1994; SCHMIDT, 1992; ALMEIDA ET AL., 1993; SANDRONI, 2001), were accomplished. Seeking to evaluate the overconsolidation of the other deposits of Brazilian soft clays, including the case in study. CAVALCANTE (2001) proposed the estimate of an overconsolidation parameter in field OCR_{GLOBAL}, through it settlement expresses and the collected data of maximum settlements estimated / measured in field and estimated stresses σ'_{V0} and $\Delta\sigma'_{V}$.

$$OCR_{global} = 10^{\left(\frac{CR \cdot Log\left(\frac{\sigma'_{VF}}{\sigma'_{V0}}\right) - \frac{\rho_{global}}{D}}{CR - SR}\right)} \qquad \text{(III)}$$

For all the cases were considered CR=40% and SR=5% due to the similarity between the compressibility parameters of Brazilian soft clays. The Figure 7 presents the values of OCR_{GLOBAL} obtained for the several embankments and for the study case. Where, can be to observe graphically the influence of overconsolidation stress in the deformations magnitude.

Table 4 - Settlement esteemed through laboratory and maximum measured in field

Side	*Maceió*	*Recife*
$\Delta\sigma_{Vi}$ /σ'_{V0i}	***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%

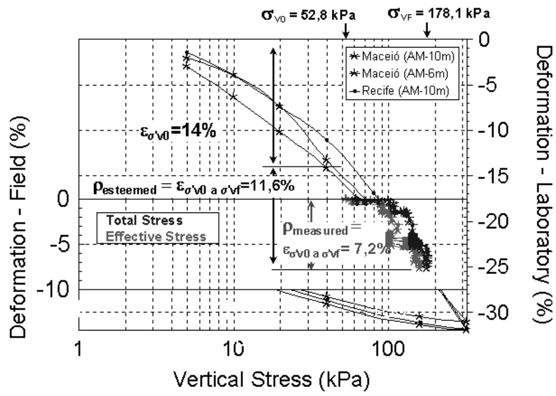


Figure 6 - Comparison between the behavior stress-deformation observed in field and in laboratory, Recife side.

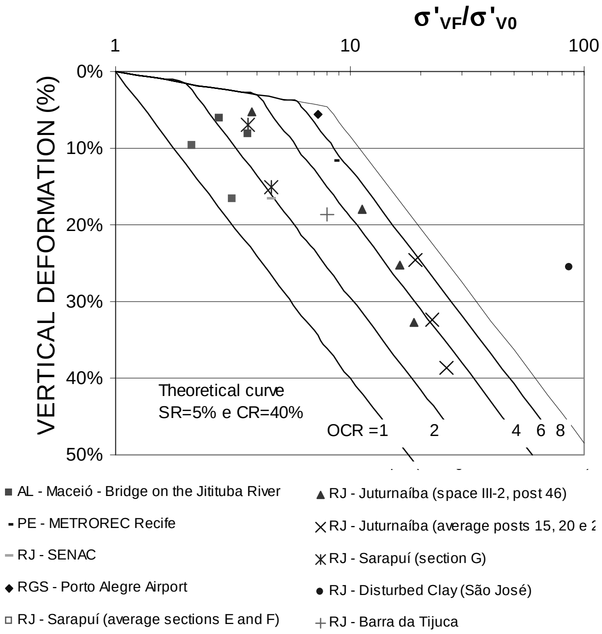


Figure 7 - Influence of the overconsolidation ratio in the magnitude of the settlements of several Brazilians embankments on soft soils.

In this present case, the OCR_{GLOBAL} had medium values for each stage of 2,1 and 2,6. In the analysis the other embankments it is observed that this parameter was presented between 1,3 and 6,7, excepting the case of a disturbed clay in Rio de Janeiro, that had presented OCR_{GLOBAL}=30,7.

4.3. Horizontal Displacements

The main purpose of the follow up horizontal displacements was to check the stability of the foundation soil during and after the construction of the embankments access of the Jitituba River Bridge in Alagoas – Brazil, in order to assure these displacements were maintained in stable conditions and presented the less minimal values. The inclinometers location can be observed in the Figure 1.

The analysis was divided in: analysis behavior and stability control. The Table 5 presents the main results obtained through the several methods announced on the bibliography and the comparison to other cases of embankments on soft soils in horizontal displacements. More details about it can be seen at CAVALCANTE (2001) and CAVALCANTE ET AL. (2003). The Figures 8 and 9 present the normalized profile of the horizontal displacements and the evolution of the maximum horizontal displacements with the time, respectively.

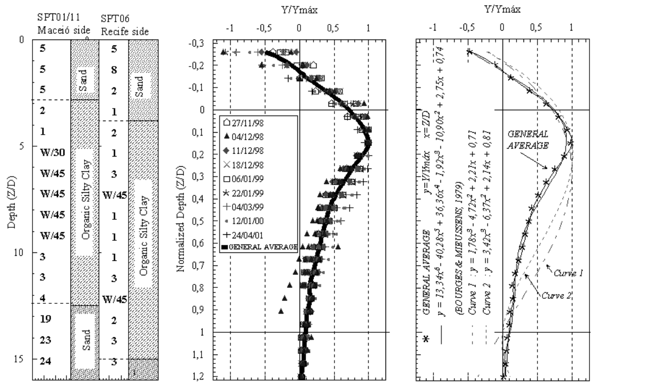


Figure 8 - Comparison of the average of the profiles normalized with the curves proposed by BOURGES & MIEUSSENS (1979).

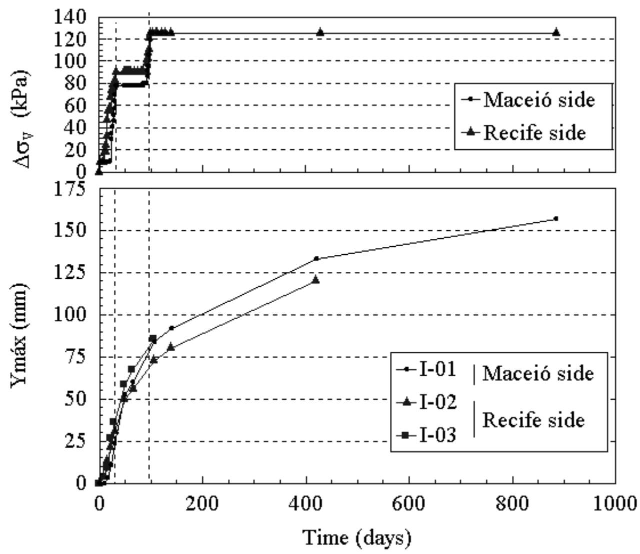


Figure 9 - Maximum horizontal displacements with time - up to period 885 days.

5. CONCLUSIONS

The main conclusions are the following:

- The stabilization of the settlements measures in each phase of the construction
- The reasonable coefficient of consolidation field obtained, through the pore-pressures and settlements methods, faced with those find in the lab (low quality samples)
- The verification of the disturbance effects on the lab samples, through the parameters evaluation obtained, and the comparison between the maximum settlements measured in the field, and estimated through the parameters in the lab.
- The overconsolidating behavior of the soil researched, through the evaluation and quantification of OCR_{GLOBAL} =2,1 to 2,6 (SR=5% and CR=-40%)
- In general, the behavior of the access embankments of the Jitituba River Bridge was stable. Evaluate through the several methods proposed in the bibliography, starting from the measures of pore-pressures and horizontal displacements.

Table 5 - Summary of the results obtained in the analysis of the horizontal displacements

ANALYSIS OF THE HORIZONTAL DISPLACEMENTS - EMBANKMENTS OF THE BRIDGE ON THE JITITUBA RIVER				
		ANALYSIS METHODS	**RESULTS**	**CLASSIFICATION**
(a) BEHAVIOR		PROFILE FORMS WITH THE DEPTH (BOURGES & MIEUSSENS, 1979)	CURVE TYPE 2	DEPOSIT WITH UNIFORM DEFORMABILITY
		ΔYmáx/ΔS (TAVENAS ET AL., 1979)	0,08 a 0,46 (FC) 0,04 a 0,22 (FA)	PARTIALLY DRAINED DRAINED
(b) STABILITY CONTROL	Ymáx/D X TIME	TENDENCY WITH THE TIME (KAWAMURA, 1985) MAXIMUM VALUE	CONVERGENT 1,62% (157 mm)	STABLE
		RATE OF VARIATION Δ(Ymáx/D)/Δt (CAVALCANTE, 2001 and CAVALCANTE ET AL., 2003)	0,024%/day	<<0,2%/day STABLE
	ANGULAR DISTORTION x TIME	TENDENCY WITH THE TIME (CAVALCANTE, 2001 and CAVALCANTE ET AL., 2003)	CONVERGENT	STABLE
		MAXIMUM VALUE — CONSTRUCTION END (ORTIGÃO, 1980 e COUTINHO,1986)	2,23%	<3% STABLE
		MAXIMUM VALUE — CONSOLIDATION END	3,55%	
		RATE OF VARIATION (ALMEIDA ET AL., 2001)	0,07%/day	<< 1,5%/day STABLE
	DISPLACED VERTICAL VOLUME X DISPLACED HORIZONTAL VOLUME	$\Delta V_V/\Delta V_H$ (SANDRONI & LACERDA, 2001)	24,6 a 4,2 (FC)	STABLE TO MEDIUM / ALERT (ΔVV/ΔVH >6) (<3ΔVV/ΔVH >6)
			46,5 a 9,2 (FA)	STABLE (ΔVV/ΔVH >6)
		(JOHNSTON, 1973)		PARTIALLY DRAINED
	$\delta h_{máx}/D \times f$	(BOURGES & MIEUSSENS,1979; CAVALCANTE, 2001 and CAVALCANTE ET AL., 2003)	0,54%	< 0,8% (f>1,71) STABLE MINIMUM HORIZONTAL DISPLACEMENTS

* FC - Stage of Construction / FA - Stage of Consolidation

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