Effects of an Abutment Construction on Soft Soil on a Neighbouring Structure: Influence of Different Construction Techniques Using Geosynthetics

Íris L. Macêdo, Ennio M. Palmeira, Gregório L.S. Araújo

Abstract. The topographic and geological conditions of several regions in Brazil favour the formation of large deposits of soft soils. Sometimes this requires the use of innovative techniques to allow constructions of geotechnical works in these sites. The presence of deep soft foundation soils has led to an increasing number of reinforced bridge abutments being built in the last years. However, little is known on the behaviour of such works, particularly with respect to displacements and bending moments induced in the neighbouring bridge foundations due to the construction of the embankment. In this paper, hypothetical bridge abutments were modelled, with and without the presence of geosynthetic reinforcement, aiming to evaluate the effects of the embankment construction on the foundations of the neighbouring bridge. The effects of the use of vertical drains and piles with caps underneath the embankments were also investigated. The results obtained clearly indicated the potential of the use of reinforcement, vertical drains or piles to reduce damages to the foundations of neighbouring structures caused by embankment construction. The use of vertical drains in conjunction with geosynthetic reinforcement showed an important effect on the reduction of horizontal displacements and moments in the piles of the bridge.

Key words: geosynthetics, soil reinforcement, soft soil, abutments, foundations.

1. Introduction

The construction of embankments on soft soils requires special design and construction techniques because of the poor stability conditions of the work and large and time-dependent deformations of the soft foundation. Depending on site conditions and project characteristics, the solutions to deal with this type of problems can be partial or total removal of the soft foundation material, embankment basal reinforcement and the use of vertical drains or piles under the embankment (Pilot, 1981; Magnan, 1983; Jewell, 1996; Leroueil *et al.*, 1985; Palmeira, 2002, for instance). Sometimes, some of these solutions can be combined. In this context, the use of the soil reinforcement technique has shown a marked increase during the last decades due to its simplicity and its cost-effectiveness. When a fast dissipation of pore pressures and acceleration of consolidation settlements are required, vertical drains can be used in combination with embankment basal reinforcement. When in-service conditions require limited amount of embankment settlements, geosynthetic reinforcement can still be used in association with piles and caps.

The complexities of designing and constructing embankments on soft soils is significantly increased when there are structures adjacent to the embankments, such as in the cases of bridge abutments. When the abutment is being constructed, not rarely the bridge foundation has already been executed, or the bridge is being constructed simultaneously to the abutment (Fig. 1). Thus, the construction of the latter can cause lateral movements of the soft foundation soil that may yield to serious problems to the existing structure. The use of soil reinforcement and/or soft soil stabilisation techniques in such cases can avoid or significantly reduce the horizontal movements of the soft ground and stresses on foundation elements of the structure. Only a limited number of studies can be found in the literature on this type of problem (Ortigão *et al.*, 2001; Palmeira *et al.*, 2001; Fahel & Palmeira, 2002; Fahel, 2003; Macedo & Palmeira, 2003; Macêdo *et al.*, 2008).

Sometimes soft soil shear strength can be so low that even the construction of rather low embankments may yield to failure. The use of basal geosynthetic reinforcement can then improve the short and long term stability conditions of the embankment (van Leeuwen & Volman, 1976; Volman *et al.*, 1977; Silva, 1996; Rowe & Soderman, 1984; Delmas *et al.*, 1990; Rowe *et al.*, 1995; Rowe, 1997; Palmeira *et al.*, 1998; Fahel, 2003; Oliveira, 2006).

Although short and long term stability of the embankment are of utmost importance, in several cases the limitation of soil deformation is also required to guarantee the project serviceability. However, the estimate of soil deformations is a more complex task than the usual limit equilibrium approaches employed for stability analyses. In these situations, the Finite Element Method (FEM) can be a useful tool to predict soil deformation and several studies involving the use of FEM to analyse the behaviour of

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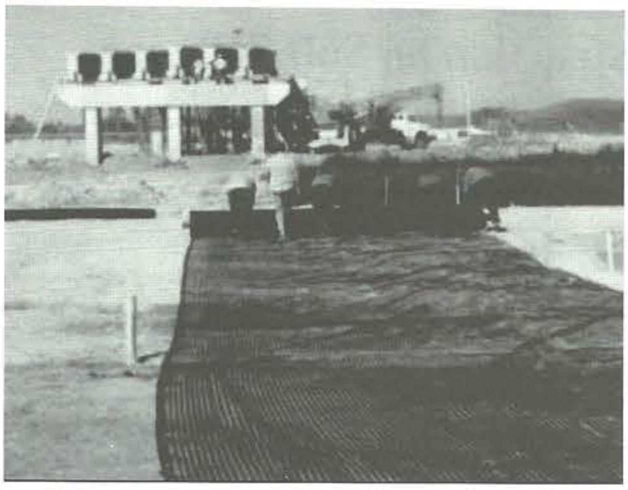

Figure 1 - Example of simultaneous bridge and abutment constructions (Fahel, 2003).

reinforced embankments on soft soils can be found in the literature (Rowe and Soderman, 1987; Schaefer and Duncan, 1988; Loke *et al.*, 1994; Sá, 2000; Borges & Cardoso, 2002; Hinchberger & Rowe, 2003; Araujo, 2004; Li & Rowe, 2008, for instance).

The use of vertical synthetic (geodrains) or granular drains can bring additional benefits to the stability and performance of the embankment, as it accelerates pore pressure dissipation and soil consolidation, with consequent shear strength gains. Besides, after construction, the additional settlements will be smaller and will occur throughout a larger period of time. When there is not enough time to wait for soil consolidation, uncertainties on the performance of the vertical drainage system and the project requires low levels of soil deformation, the use of piled foundation for the embankment can be chosen, though its cost is considerably greater than other solutions. In this case, most of the load of the embankment is transmitted to a stiffer soil layer in depth, minimising the total stress increments in the soft foundation mass.

This paper presents a finite element analysis on the influence of the use of different solutions for abutment reinforcement or soft soil stabilisation on the behaviour of the foundations of a neighbour bridge. Several aspects relevant to the performance of reinforced soil embankments were investigated in these analyses.

2. Characteristics of the Problems Investigated

2.1. Modelling characteristics

In this work the construction of hypothetical abutments were simulated by the Finite Element Method (FEM). The abutments were 5 m high and were built on a 12 m thick soft and saturated foundation soil. The computer code PLAXIS (Brinkgreve & Vermeer, 1998), available at the Graduate Programme of Geotechnics of the University of Brasilia, Brazil, was employed in the analyses, which were carried out under plane strain conditions. Five constitutive models for soil behaviour are available in the program. The mesh generation is automatic and elements can be prescribed with six or fifteen nodes. In the analysis, staged construction of the embankment was simulated and some cases involved the use of vertical drains or piles underneath the embankment. Because of the large number of relevant variables present in this type of analysis, this study focused on the investigation of the performance of abutments on soft soils using typical values for the geometrical characteristics and material properties found in this type of work.

The behaviour of the soft soil was modelled using the elastic-plastic "soft soil" model present in the Plaxis code, which is based on the Cam-Clay model, and uses Biot's theory of consolidation (Biot, 1941). An elastic-plastic model ("Mohr-Coulomb Model" in the Plaxis code) was used for the abutment material and for the soil underneath the soft soil layer. The reinforcement layers were simulated using the "geotextile element" present in the Plaxis code, for which a linear elastic response is assumed.

Two different situations were considered in the simulation of the foundations of the neighbouring bridge. In the first situation, only the piles of the adjacent structure were already constructed at the start of the abutment lifting. Thus, under these circumstances the top of the piles were not subjected to vertical loads or movement restriction yet, except for that from the passive resistance of the foundation soil. Hereafter, these analyses will be referred to as Free Pile Top analysis (FPT analysis). In the second situation, it was assumed that the bridge was already constructed at the start of the embankment lifting. In this case, the line of piles closest to the embankment toe are vertically loaded by a fraction of the bridge weight and there is further restriction to the movement of the piles because of the influence of the presence of the other lines of piles of the bridge foundation system. This type of analyses will be referred to in this paper as Restricted Pile Top analysis (RPT analysis). In both types of analyses, the investigations were concentrated on the study of the effects of abutment construction on the piles of the bridge at varying distances from the embankment toe. In all cases analysed (either FPT or RPT analyses) the piles penetrated 0.5 m in the stronger soil layer underneath the soft soil.

Additional comments and information on the cases studied and methodology used can be found in Macêdo (2002).

2.2. Geometric characteristics and properties of the materials

The height of the abutment was assumed equal to 5 m. Analyses of unreinforced abutments with this height, constructed on the soft soil with the properties used in this study, showed that reinforcement layers would be required in the abutment for that height to be reached. Even one reinforcement layer was not enough in some cases for the abutment to reach its final height, depending on the reinforcement tensile properties. The abutments were constructed in five stages of 1 m. The results presented in this paper are those obtained at the end of embankment construction (28 days total construction time) and 180 days after embankment construction.

Different values of tensile stiffness and number of reinforcement layers were used in the analyses. When more than one layer of reinforcement was used, the vertical spacing between layers was equal to 0.5 m. The bottom reinforcement layer always coincided with the abutment-soft soil interface, except in the case of the presence of piles and caps underneath the embankment.

Figure 2(a) shows a general view of the geometrical characteristics of the problem analysed. Figure 2(b) presents the finite element mesh utilized in the calculations, based on initial trial runs of the programme to evaluate the size of the mesh to be used to minimise boundary effects. The slope of the abutment facing the bridge was equal to 2:1 (horizontal:vertical).

As mentioned above, some analyses considered the presence of vertical drains to accelerate pore pressure dissipation. In these cases the spacing between vertical drains was assumed equal to 1.5 m, with the drains distributed in a square pattern in plan. To model the vertical drains, interface elements were employed and the equivalent wall concept (Indraratna & Redana, 1997) was used to satisfy plane strain conditions.

An additional study involved the investigation of solutions incorporating piles and caps under the abutment with varying spacing between piles. The piles and caps

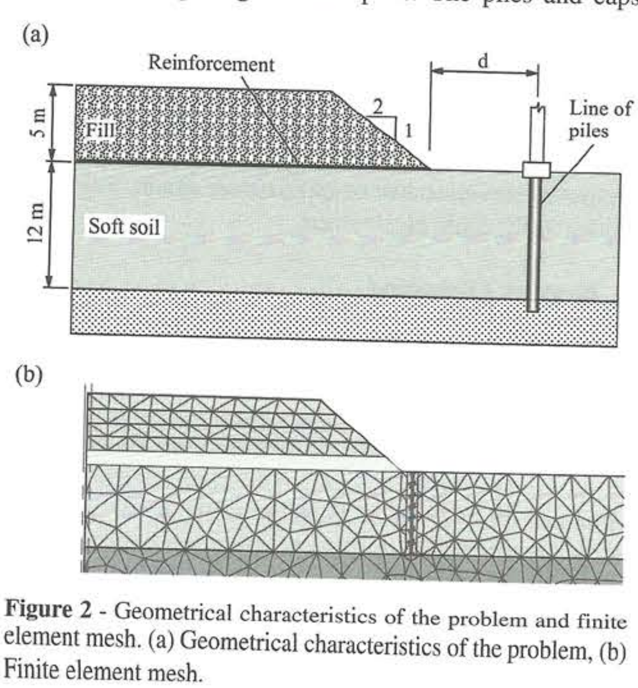

Figure 2 - Geometrical characteristics of the problem and finite element mesh. (a) Geometrical characteristics of the problem, (b) Finite element mesh.

were distributed along the entire base of the abutment, were assumed as made of concrete and were modelled as beam elements. The piles had a diameter of 0.25 m and were 12.5 m long, whereas the caps were 1 m x 1 m x 0.5 m (height). The piles were distributed in plan in a square pattern with varying spacing between them, depending on the objectives of the study. Interface elements were used along the shaft of the piles under the embankment as well as in the piles of the neighbouring bridge to simulate more accurately the interaction between pile and foundation soil. The piles under the abutment were simulated under two-dimensional conditions as equivalent walls. The thickness of the wall was calculated as a function of the pile diameter, spacing and material properties in order to attend stiffness equivalency with the actual array of discrete piles. When present, the reinforcement layer was assumed to be installed on the top of the caps. It is important to point out that in this analysis the reinforcement layer was assumed directly on the cap for the sake of simplicity in the modelling process. In a real situation, the direct contact between cap and reinforcement may cause mechanical damage to the reinforcement (Almeida *et al.*, 2008).

Typical soil properties were assumed for the soils involved in the analysis. Table 1 summarises the properties used in the analyses for the embankment soil, soft soil layer and for the stronger soil layer (3 m thick) underneath the soft soil. These properties are required by the computer code used (Plaxis) when the "Soft Soil" and "Mohr-Coulomb" models are employed.

Some hypotheses had to be made for the analyses of the cases where the entire bridge was already built by the time of the abutment construction (RPT). In this case, a 40 m long and 12 m wide concrete bridge was assumed for this study, as shown in Fig. 3. The bridge is supported by 6 columns. Each couple of columns transfers the vertical load to a group of 10 piles in line, 0.3 m in diameter and with a spacing of 1.5 m. The central columns have diameter equal to 1.5 m and those at the extremities of the bridge have diameter of 1 m. Traditional structural design procedures were used for the hypothetical reinforced concrete bridge (Macêdo, 2002) and the vertical loads acting on each set of two columns were equal to 5000 kN, which yields to an average vertical load per pile equal to 500 kN. This load is consistent with the allowable axial load for a 0.3 m diameter pile reinforced with four 12.5 mm diameter steel bars.

It should be pointed out that, in fact, the problem examined in this paper is a three dimensional one. So, with this regard, the results obtained should be considered as approximations to the actual conditions in the field. To simulate the line of piles of the bridge foundation under plane strain conditions, the equivalent wall concept was used. As in the case of piles underneath the abutment, the thickness of the equivalent wall was calculated as a function of the pile diameter, spacing and material properties

Table 1 - Soil parameters.

Parameter	Symbol	Unit	Material		
			Soft soil (undrained)	Fill (drained)	Bottom soil layer (drained)
Unsaturated unit weight	γ_d	kN/m^3	13.0	19.0	20.0
Saturated unit weight	γ_w	kN/m^3	16.0	20.0	22.0
Horizontal permeability	k_x	m/day	0.001	1,000	1,000
Vertical permeability	k_y	m/day	0.001	1,000	1,000
Young Modulus	E'	kPa	-	15000	60000
Poisson's ratio	υ'	-	-	0.30	0.30
Shear modulus	G'	kPa	-	5769.2	23076.9
Oedometer modulus	E_{oed}	kPa	-	20192.3	80769.2
Cohesion	c'	kPa	5.0	1.0	5.0
Friction's Angle	ϕ'	(°)	25.0	30.0	36.0
At-rest earth pressure coefficient	k_o	-	0.64	0.50	0.41
Modified compression ratio	λ^*		0.08	-	-
Modified expansion ratio	κ^*		0.011	-	-
Over-consolidation ratio	OCR		1.3	-	-
Poisson's ratio (unloading/ reloading)	υ_{ur}		0.15	-	-

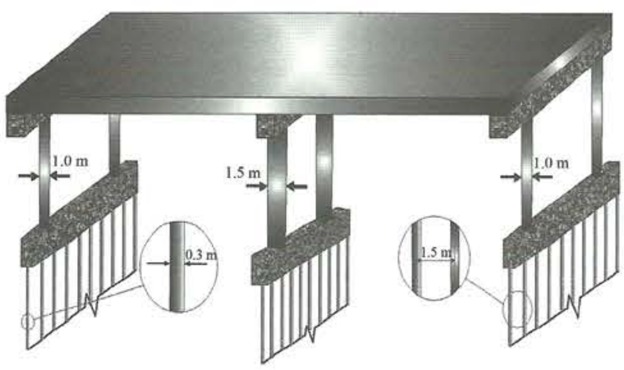

Figure 3 - Bridge geometrical characteristics.

Table 2 - Pile and equivalent wall characteristics.

Parameter	Symbol	Unit	Value
Diameter	D	m	0.30
Young modulus	E	kPa	3.5 x 10^7
Cross section area	A	m^2/m	0.15
Moment of inertia	I	m^4/m	2.65 x 10^{-4}
Axial stiffness	EA	kN/m	5.15 x 10^6
Flexural rigidity	EI	kNm^2/m	9.28
Equivalent wall thickness	d_{eq}	m/m	0.15
Poisson's ratio	υ	-	0.15

in order to attend stiffness equivalency to the line of discrete piles (Fig. 3). Table 2 presents the mechanical and geometrical characteristics of the piles and of the equivalent wall.

A beam element working in a similar way as a spring was used to simulate the restriction to the movement of the pile top due to the presence of the bridge (Fig. 3, RPT analysis). The stiffness (K) of this spring like element was obtained simulating the bridge subjected to different values of an horizontal load (Q) applied to one of its ends, as schematically shown in Fig. 4, and obtaining the corresponding horizontal displacements (δ). Thus, the stiffness (K) of the spring element to be used at the top of the pile, that would restrict its movements in an approximate similar way as the bridge itself, could be obtained.

3. Results Obtained

This section presents and discusses the results obtained for the maximum horizontal displacements (δ_{hmax}), bending moments on the bridge piles, maximum abutment vertical displacements (δ_{vmax}) and maximum tensile forces in the reinforcement (T_{max}) caused by the construction of the abutment for FTP and RTP analyses. The tensile stiffness (J) and number of reinforcement layers was varied, as well as the distance between the piles and the abutment toe and the distance between pile caps (s - a) in the case of piled abutments, where s is the spacing between piles and a is the cap width.

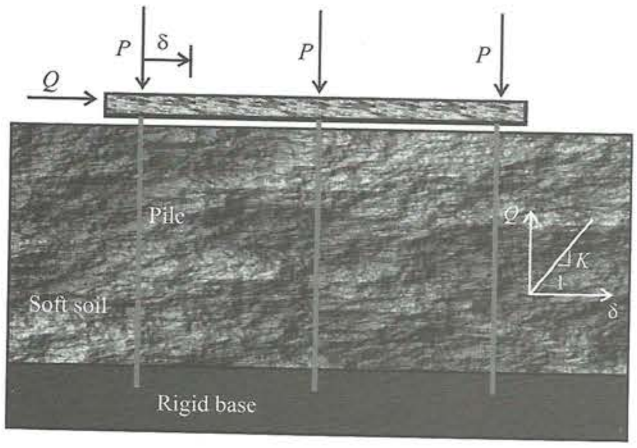

Figure 4 - Estimate of the equivalent bridge stiffness (K) of the spring like element to be used at the top of the piles in RPT analyses.

3.1. Maximum horizontal displacements (δ_{hmax}) of the piles

3.1.1. Influence of the tensile stiffness and number of reinforcement layers

Figure 5 shows the variation of maximum horizontal displacements (δ_{hmax}) of the bridge piles as a function of the geosynthetic stiffness for different numbers of reinforcement layers and for a distance (d) between the line of piles and the abutment toe equal to 1 m for the FPT (free pile top) case. In these cases the maximum displacements occurred at the pile top. These results are those obtained for a time (t) equal to 180 days after the end of construction of the abutment. It can be observed that the reinforcement stiffness has an important effect on the reduction of horizontal displacements. For the conditions analysed, the stiffer the reinforcement and the greater the number of reinforcement layers the smaller the horizontal displacements of the foundation soil and of the piles. However, the influence of the number of reinforcement layers is reduced as the reinforcement tensile stiffness increases. For this and other figures in this paper, missing points in the graph (particularly for low values of

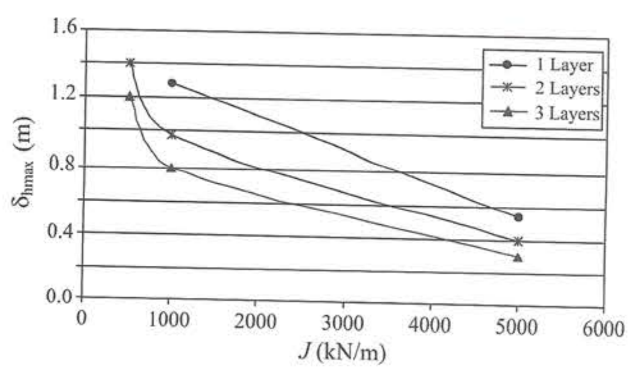

Figure 5 - Maximum pile horizontal displacement *vs.* reinforcement tensile stiffness - FTP analysis, d = 1 m and t = 180 days.

reinforcement stiffness - 1 reinforcement layer and J = 500 kN/m in Fig. 5, for instance) indicate situations for which the embankment failed (excessive number of plastic points) before the end of construction.

It should be noted that the lines of piles (or equivalent wall) is also a stabilizing element for the abutment, avoiding or minimising the possibility of failure. The line of piles works as a retaining structure, as far as the horizontal movement of the soft soil is concerned, increasing the stability of the system as a whole. For values of d greater than 1 m reinforcements with high values of tensile stiffness are required to make possible the construction of the abutment to its final height. Because of the stabilising effect of the line of piles, less stiff reinforcements were required to construct the abutment to its final height for the cases where the piles were installed closer to the embankment toe (d = 1 m).

The restriction of the pile top (RPT case) influences the displacements of the piles and changes their horizontal displacement profiles, as shown in Fig. 6 for the unreinforced case. For FPT cases, the maximum horizontal displacements occur at top of the pile, whereas for RPT cases the maximum horizontal displacements occur at a depth approximately equal to 0.3 times the soft soil thickness. As expected, the displacements obtained in RPT cases were significantly smaller than those obtained in FPT cases. The displacements in the former case can be up to 75% smaller and are a consequence of the influence of the stiffness of the bridge and of the other foundation elements present.

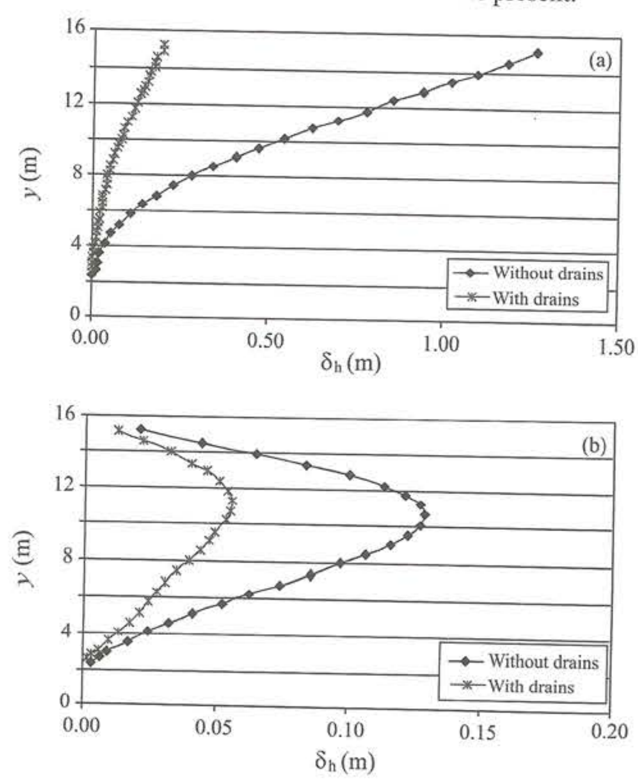

Figure 6 - Horizontal displacement profiles of the piles under unreinforced conditions - d = 1 m and t = 180 days. (a) FPT case, (b) RPT case.

Figs. 7(a) and (b) show the influence of the reinforcement tensile stiffness and number of reinforcement layers on the maximum horizontal displacements of the piles, for cases where the movement of the pile top was restricted (RPT case). The results indicate that the reinforcement stiffness is more important than the number of reinforcement layers for the reduction of the maximum horizontal displacement (Fig. 7a). When the distance (d) between the embankment toe and the piles is increased to 3 m, the pile displacements decrease as well as the influence of the reinforcement tensile stiffness on the pile movements (Fig. 7b).

3.1.2. Influence of the presence of vertical drains

The presence of vertical drains in the soft foundation soil causes a significant reduction on the horizontal displacements of the soft soil associated with the rapid dissipation of pore pressures generated by the abutment construction. Figures 8(a) and (b) show the variation of maximum horizontal displacements of the piles *vs.* reinforcement stiffness for the cases where the presence of vertical drains in the foundation soil were considered. Comparing the results in Fig. 8(a) and in Fig. 5, it can be observed a reduction up to 86% in the pile horizontal displacements close to the abutment toe (d = 1 m) due to the presence of the vertical drains, in comparison with the same case without the drains (Fig. 5). This reduction was observed for the abutment with two layers of reinforcement at its base and with a reinforcement tensile stiffness of 500 kN/m. The presence of the vertical drains allowed the construction of the abutment in all the cases analysed (no failures during construction), including those where reinforcements were not used. This can be explained by the fast increase in effective stresses due to pore pressure dissipation and consequent increase in soft soil strength when the drains were present.

Figure 9 shows that for RTP cases the presence of the vertical drains caused a reduction of approximately 60% on the pile horizontal displacements in comparison with the same case without the drains, for d = 1 m and t = 180 days (Fig. 7a). In RTP cases, the maximum reduction on pile displacement due to the presence of the reinforcement varied between 4.5% and 14% in comparison to the unreinforced case (J = 0 in Fig. 9), depending on the stiffness and number of reinforcement layers considered. The presence of the reinforcement layers was more significant for a number of layers equal to 3 (Fig. 9). However, the presence of the reinforcement layers may be fundamental during abutment construction to guarantee short-term stability conditions.

3.1.3. Influence of the presence of piles and caps underneath the abutment

The movements of the piles of the bridge foundation were also investigated for the case of a piled abutment with and without the presence of reinforcement layers. Only FPT cases were analysed under these conditions. In general, as expected, the shorter the distance between caps the smaller the movements of the bridge piles. As the distance

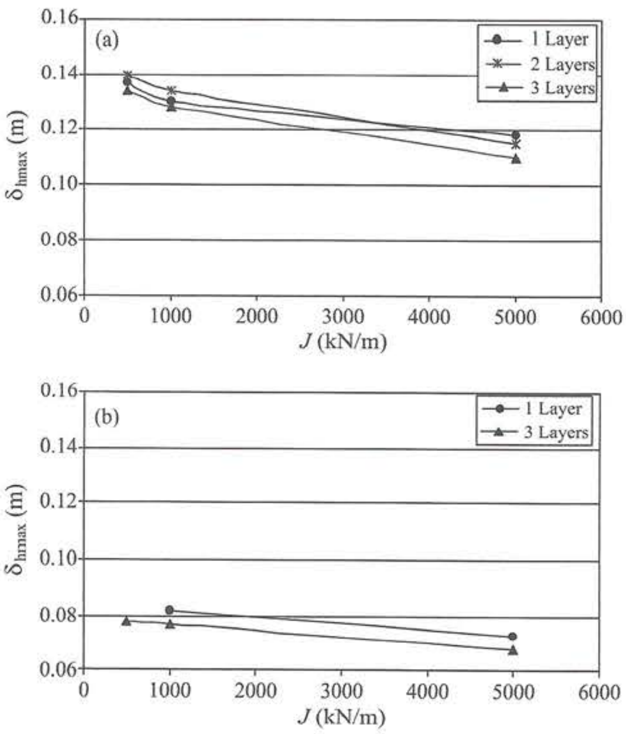

Figure 7 - Maximum pile horizontal displacement *vs.* reinforcement tensile stiffness - RPT cases and t = 180 days. (a) d = 1 m, (b) d = 3 m.

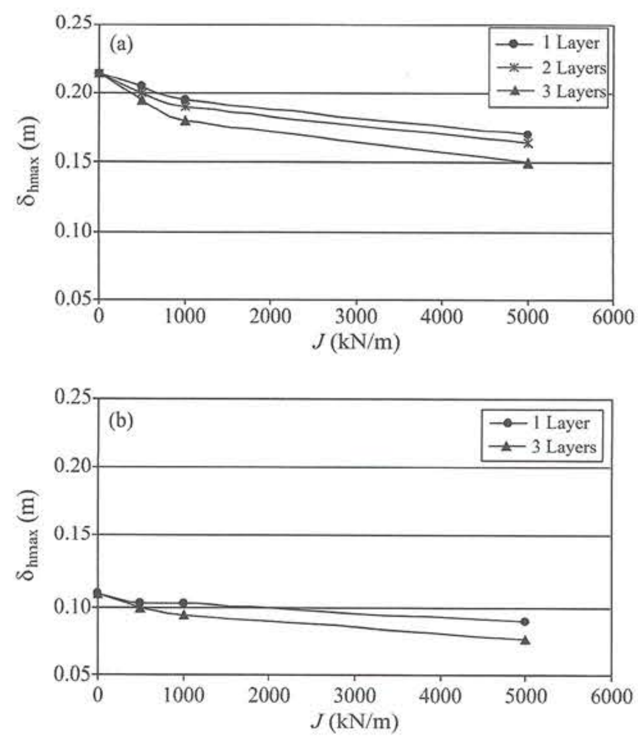

Figure 8 - Maximum pile horizontal displacements *vs.* reinforcement stiffness for the cases with vertical drains - FPT cases and t = 180 days. (a) d = 1 m, (b) d = 3 m.

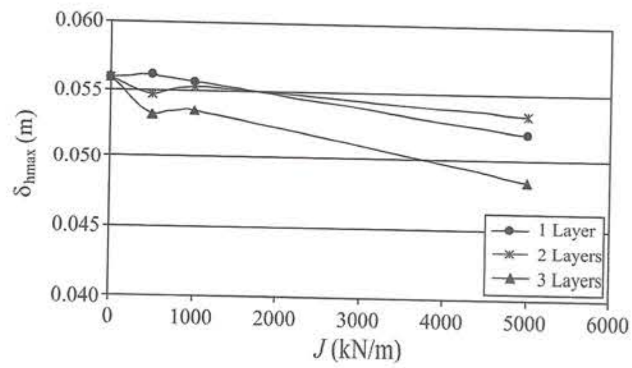

Figure 9 - Maximum pile horizontal displacement *vs.* reinforcement tensile stiffness in cases with vertical drains - RPT cases, $d = 1$ m and $t = 180$ days.

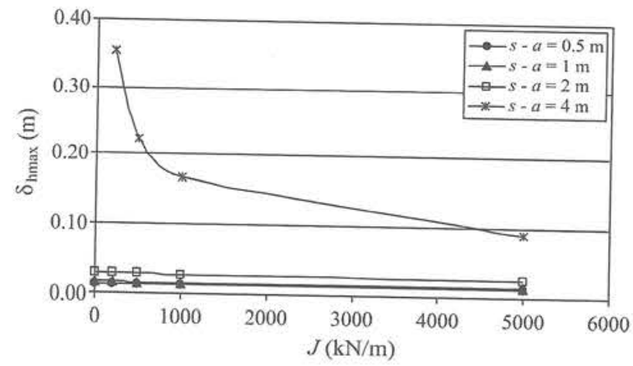

Figure 11 - Variation of pile horizontal displacements with reinforcement stiffness for different values of (s - a), $d = 1$ m and $t = 180$ days - FPT cases.

between caps increases, the efficiency of the piles is reduced because of the greater transference of abutment loads to the foundation soil caused by less arching of abutment soil in between the pile caps. The presence of the geosynthetic layer in the abutment improves the distribution of loads to the piles and the performance of the piled abutment.

Figure 10 depicts the variation of horizontal displacements of the piles of the bridge as a function of the distance between caps, given by (s - a), where s is the spacing between piles under the embankment and a is the cap width. It can be noted the small values of bridge piles displacements (less than 0.03 m) for values of (s - a) smaller than 3 m. It can also be observed that, in numerical terms, the construction of the embankment without a geosynthetic layer on the caps was possible only for values of (s - a) up to 2 m. For larger values, excessive horizontal displacements of the piles occurred. However, for the cases where a reinforcement layer with $J = 5000$ kN/m was used the construction of the abutment was possible up to values of s - a equal to 4 m, with a maximum pile horizontal displacement of 0.09 m.

Figure 11 shows the variation of maximum horizontal displacement of the bridge piles *vs.* reinforcement stiffness for piles distant 1 m from the abutment toe and for different values of the distance (s - a) between the caps of the piled abutment. The results in this figure show that there is little influence of the value of reinforcement stiffness on horizontal displacements for values of (s - a) smaller than 2 m. However, for (s - a) equal to 4 m there is a marked effect of the increase in reinforcement tensile stiffness in reducing horizontal displacements of the piles of the bridge. A similar behaviour was observed by Macêdo (2002) for the case of a 6 m thick soft foundation soil layer.

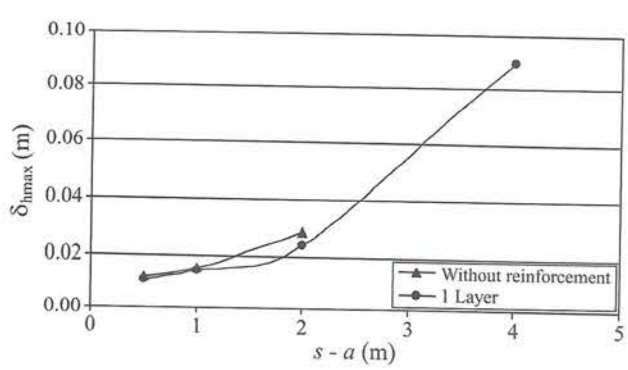

Figure 10 - Maximum pile horizontal displacement *vs.* distance between caps for $J = 5000$ kN/m, $d = 1$ m and $t = 180$ days - FPT cases.

3.2. Maximum bending moments in the piles

3.2.1. Influence of the reinforcement tensile stiffness and number of reinforcement layers

An accurate prediction of the bending moments in the bridge piles is fundamental for the stability of the bridge. Figures 12(a) and (b) present maximum values of predicted pile bending moments *vs.* reinforcement stiffness for FPT and RPT cases, respectively, at the end of the abutment construction ($t = 28$ days). A marked influence of the presence and tensile stiffness of the reinforcement can be noticed in FPT cases (Fig. 12a). A reduction of maximum bending moments up to 60% was obtained with an increase of reinforcement tensile stiffness from 1000 kN/m to 5000 kN/m in those cases. There is less influence of the stiffness and number of reinforcement layers on the maximum bending moments in the pile in RPT cases than in FPT cases (Fig. 12b).

3.2.2. Influence of the presence of vertical drains

The influence of the presence of vertical drains underneath the abutment is shown in Figs. 13(a) and (b) for values of d equal to 1 m and 3 m (FPT cases and $t = 180$ days), where it can be seen that the maximum pile bending moment decreases with the increase of the reinforcement tensile stiffness, but there is little influence of the number of reinforcement layers. For piles distant 1 m from the abutment toe the use of a reinforcement with a tensile stiffness of 5000 kN/m caused a reduction of approximately 34% on the maximum bending moment obtained for the situation without reinforcement (Fig. 13a).

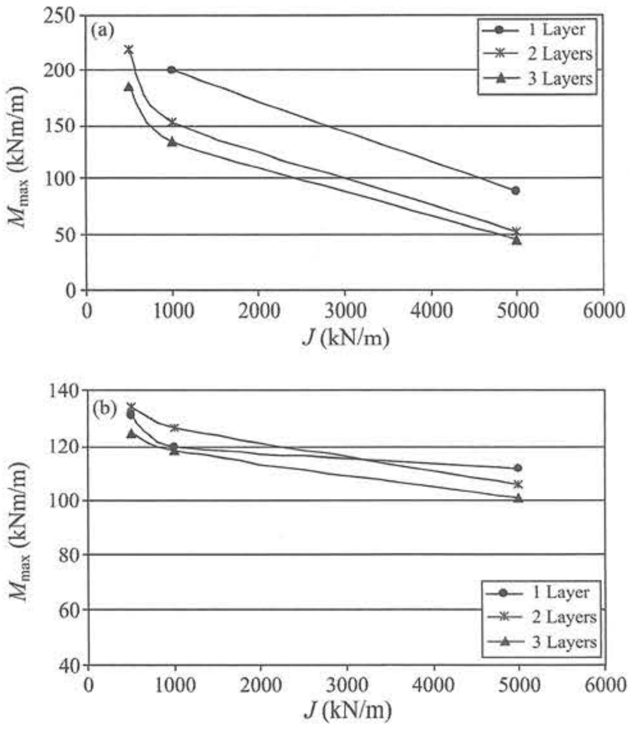

Figure 12 - Maximum pile bending moment *vs.* reinforcement tensile stiffness for $d = 1$ m and $t = 28$ days. (a) FPT case, (b) RPT case.

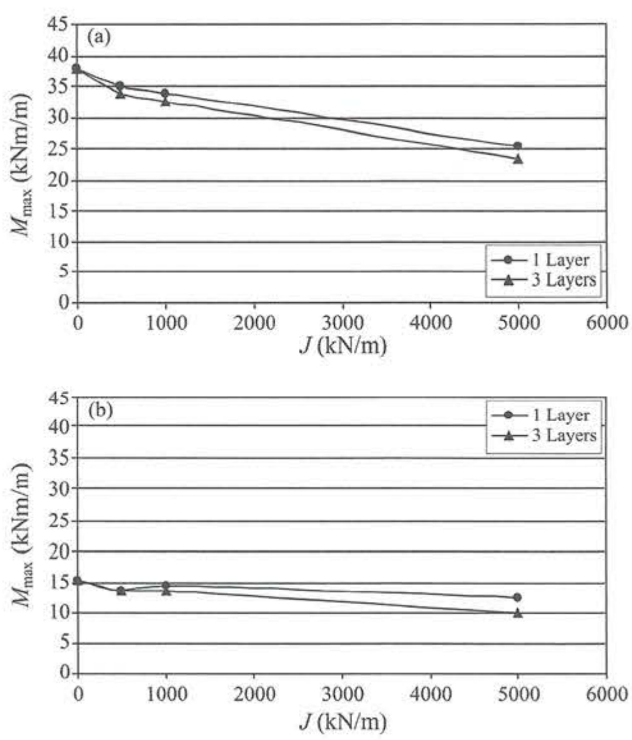

Figure 13 - Maximum pile bending moments *vs.* reinforcement tensile stiffness for the cases with vertical drains - FPT case and $t = 180$ days. (a) $d = 1$ m, (b) $d = 3$ m.

3.2.3. *Influence of the presence of piles and caps underneath the abutment*

Figure 14 presents the influence of the presence of piles and caps underneath the abutment on the maximum bending moments in the bridge piles at the end of construction for RPT cases. It can be noted that there is a substantial reduction on the pile moments due to abutment piling and that this reduction is a function of the distance between caps (s - a). The tensile stiffness of the reinforcement seems to be relevant only for large values of s - a.

3.2.4. *Bending moment distribution along the pile length*

Figures 15(a) and (b) show the distributions of bending moments along the pile length for $d = 1$ m, at the end of abutment construction ($t = 28$ days), for FPT cases, with and without the presence of vertical drains underneath the abutment. It can be noted that for the case without vertical drains there is a greater influence of the presence of the reinforcement layers and of the magnitude of the reinforcement tensile stiffness on the bending moment distribution along the pile length (Fig. 15a). The use of vertical drains reduces significantly the values of the bending moments and for the case where a reinforcement layer with $J = 5000$ kN/m was employed, the maximum bending moment was approximately 40% smaller that that observed for the case without reinforcement (Fig. 15b).

3.3. Vertical displacements of the abutment

3.3.1. *Influence of the tensile stiffness and number of reinforcement layers*

Figures 16(a) and (b) present the results of maximum abutment settlement *vs.* reinforcement stiffness at the end of construction ($t = 28$ days) and for $t = 180$ days, respectively, for FPT cases. The greatest variations of maximum settlement with reinforcement tensile stiffness occurred for values of the latter up to 1000 kN/m. The greater the reinforcement tensile stiffness the less the influence of the number of reinforcement layers. In these analyses it was also observed that increases on reinforce-

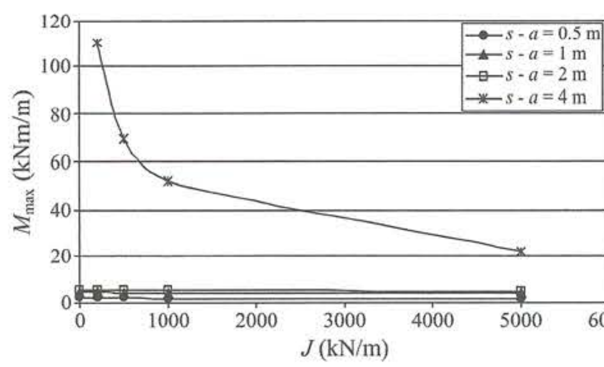

Figure 14 - Maximum pile bending moment *vs.* reinforcement tensile stiffness for cases of piled abutments - RPT cases and $t = 28$ days).

ment tensile stiffness and number of reinforcement layers tend to result in more uniform settlement profiles, reducing the differential settlements of the abutment along its longitudinal direction.

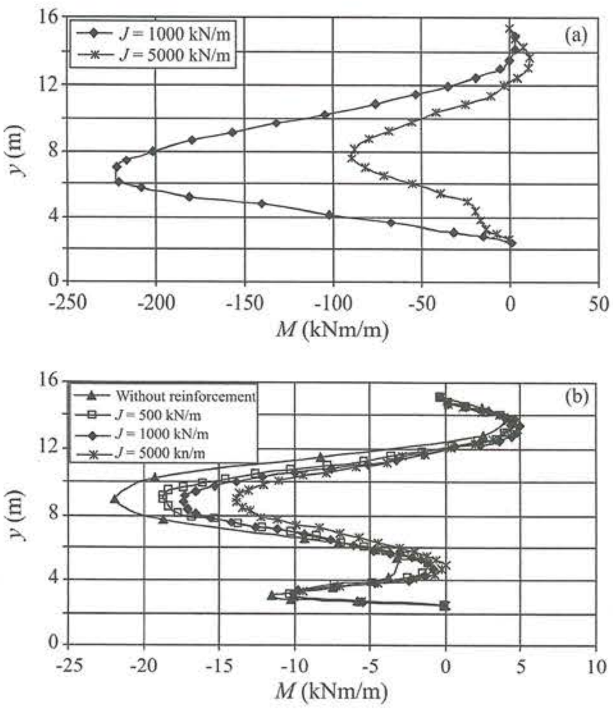

Figure 15 - Distribution of bending moments along the pile length for d = 1 m and t = 28 days (FPT case). (a) Without drains, (b) With drains.

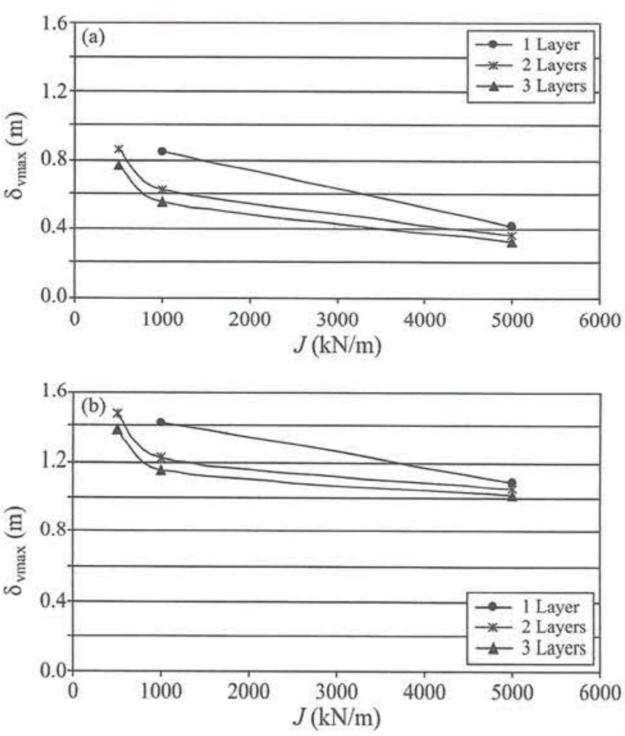

Figure 16 - Maximum abutment settlement *vs.* reinforcement tensile stiffness - FPT cases and d = 1 m. (a) t = 28 days, (b) t = 180 days.

The results of maximum abutment settlements *vs.* reinforcement tensile stiffness for RPT cases are depicted in Figs. 17(a) and (b) for t = 28 days and t = 180 days. With respect to reduction of abutment settlements, the use of three reinforcement layers provides only marginal benefit to the use of 2 reinforcement layers. However, the third layer of reinforcement may be necessary to preserve the stability conditions of the abutment, depending on the tensile strength of the reinforcement and on the project characteristics.

3.3.2. Influence of the presence of vertical drains

Figures 18(a) and (b) shows the settlement profiles along the base of the abutment with and without vertical drains at the end of construction. It can be noted that the presence of the vertical drains causes a more uniform abutment settlement profile independent on the presence or not of reinforcement. For the case of vertical drains, and for the conditions analysed, the influence of the reinforcement tensile stiffness on the settlement profile was negligible (Fig. 18b).

3.4. Forces mobilised in the reinforcement

Figure 19 shows the variation of maximum tensile force (T_{max}) mobilised in the bottom reinforcement layer with tensile stiffness for an abutment with two reinforce-

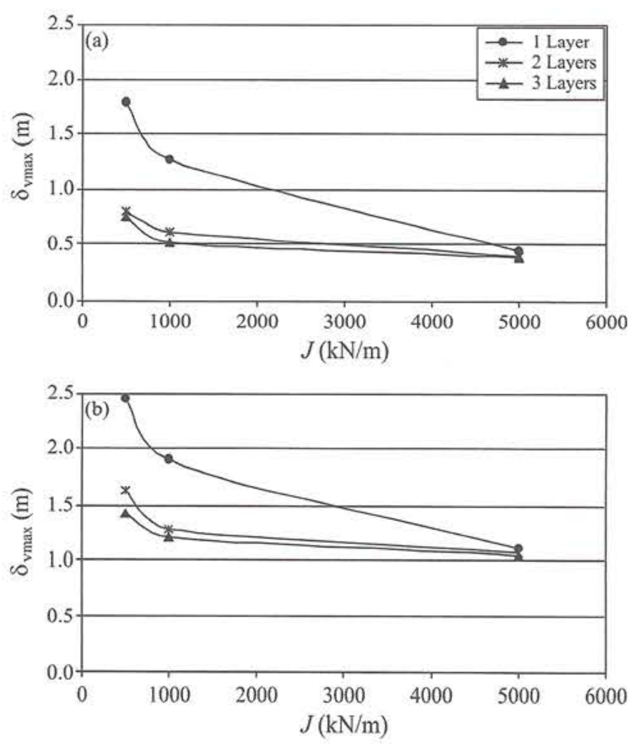

Figure 17 - Maximum abutment settlement *vs.* reinforcement tensile stiffness - RPT case and d = 1 m. (a) t = 28 days, (b) t = 180 days.

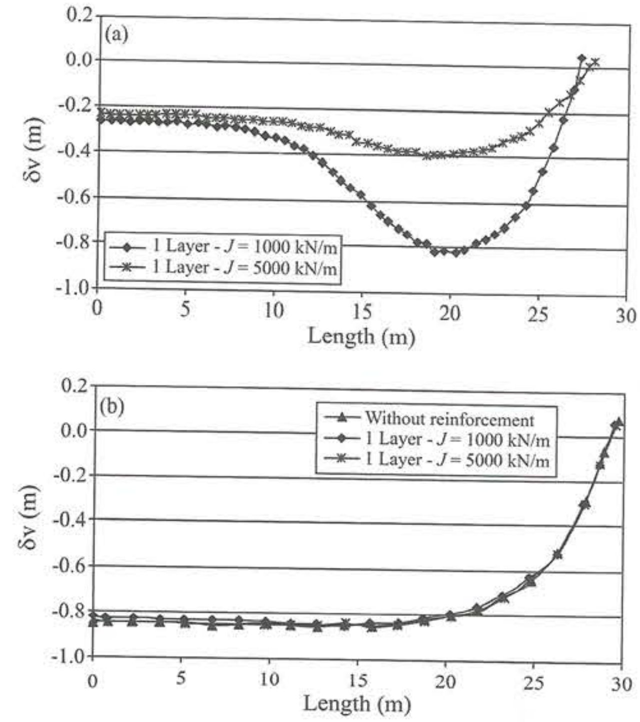

Figure 18 - Abutment settlement profiles with and without vertical drains for d = 1 m at t = 28 days - FPT cases. (a) Without drains, (b) With drains.

ment layers, d = 1 m and t = 180 days in FPT cases. In cases with more than one reinforcement layer in the abutment, the bottom reinforcement layer was the one that presented the highest tensile force. It can be noted in Fig. 19 that the presence of the vertical drains reduces significantly the maximum tensile force mobilised.

The influence of the restriction of the pile movement (RPT cases) on the tensile force mobilised in the lowest reinforcement for the case of an abutment with two reinforcement layers, d = 1 m and t = 180 days is shown in Fig. 20. Comparing Figs. 19 and 20 it can be observed that the restriction of the pile movement alone reduces the maximum

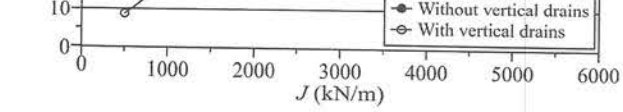

Figure 19 - Maximum mobilised tensile force in the bottom reinforcement *vs.* reinforcement tensile stiffness for cases with vertical drains - FTP cases, d = 1 and t = 180 days.

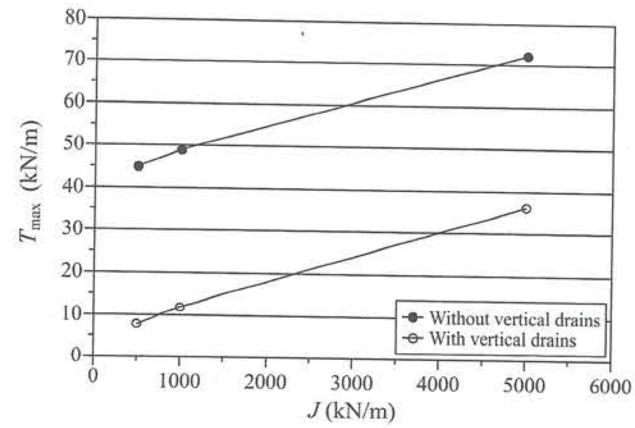

Figure 20 - Maximum mobilised tensile force in the bottom reinforcement *vs.* reinforcement tensile stiffness - RTP cases, d = 1 and t = 180 days.

force mobilised in the reinforcement. However, the greatest reduction on mobilised tensile forces is caused by the presence of the vertical drains. The vertical drains cause smaller horizontal displacements of the soft foundation soil, which yields to less mobilisation of tensile forces in the reinforcement layers.

4. Conclusions

This paper presented a numerical study on the influence of the construction of abutments on soft soils on neighbouring structures. The influence of several techniques to stabilise the abutment and to reduce the effects of its construction on the foundations of a neighbouring bridge were examined, such as the use of reinforcement, vertical drains and piles underneath the abutment. The results obtained showed that for the cases analysed the reinforcement tensile stiffness had an important effect in reducing horizontal displacements of the piles of the bridge. The increase in the number of reinforcement layers in the embankment reduced even further the horizontal movement of the foundation soil and consequently the displacements of the piles.

The use of vertical drains had a marked effect on the reduction of soft soil lateral movement with beneficial effects to the behaviour of the piles of the neighbouring bridge. When vertical drains were combined with basal reinforcement of the abutment the reduction of horizontal displacements of the bridge piles was increased even further.

As expected, the use of piles with caps underneath the abutment was the solution that presented the best results in terms of minimising horizontal movements of the soft foundation soil. The beneficial effect of the presence of geosynthetic reinforcement in this case was more evident for large distances between pile caps.

Either the use of high tensile stiffness reinforcements or vertical drains yielded more uniform settlement profiles of the abutment, with more relevance for the use of vertical drains in that regard.

The maximum tensile forces mobilised in the reinforcement were influenced by the presence of vertical drains or restrictions to the movement of the bridge piles. For cases of abutments reinforced with more than one reinforcement layer, the bottom layer was the one presenting the highest tensile force.

It is important to point out the limitations of the analyses carried out, such as the treatment of a three dimensional problem as an equivalent two dimensional one, limitations inherent to finite element analyses and the geometrical conditions assumed. Nevertheless, the results obtained show the potential of the use of basal reinforcement and vertical drains as an effective solution to reduce the effects of the construction of abutments on soft soils on the foundations of bridges or other neighbouring structures. However, one should bear in mind that there will certainly be cases where the presence of the reinforcement (and vertical drains) alone may not be sufficient to guarantee an appropriate margin of safety against damage to the foundations of the structure. Further studies are required for a better understanding and quantification on the use of geosynthetic reinforcement in such applications.

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