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Behaviour of Geogrid Reinforced Abutments on Soft Soil

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ABSTRACT: The behaviour of embankments on soft soils is a complex problem, particularly when the substitution of the soft material is not cost effective. In this case, the use of geosynthetics may be a feasible and economical solution for the stabilization of the embankment and reduction of the effects of differential settlements. This paper shows an investigation on the use of geogrid reinforcement in combination with pre-fabricated vertical drains (PVD) to accelerate soft soil consolidation in abutments for the duplication of the BR-101 highway, in Brazil. The instrumentation of bridge abutments included inclinometers, settlement and horizontal displacement plates, full-profile settlement gauges and piezometers, as well as strain gauges in the reinforcement layer. The results obtained showed that one of the abutments almost collapsed due to wrong construction practice and reinforcement specification. Insufficient reinforcement tensile force mobilization along the embankment transverse direction combined with wrong reinforcement orientation yielded to large embankment displacements and the initiation of failure. This failure mechanism could have been predicted with the use of current slope stability analysis for this type of problem. The results also showed the beneficial effect of the reinforcement for the stability of the embankment and reduction of lateral displacements of the abutments.

KEYWORDS: Reinforced abutments, soft soil, geogrid, stability, strains

1. INTRODUCTION

The construction of embankments on unstable soils is common in geotechnical engineering due to the need to continuously improve and expand the transportation infra-structure of countries. Poor construction practice in this type of work can lead to failures or excessive deformation of the embankment, which may compromise its function. Because of these characteristics several traditional embankment and soft soil stabilizing solutions have been available and employed for decades. Among some of the more recent engineering solutions, the use of geosynthetics for embankment reinforcement and pre-fabricated vertical drains to accelerate soft soil consolidation are nowadays some of the most popular ones. The combined use of these solutions can provide short and long term stability for the embankment, accelerate consolidation settlements of the foundation soil (vertical drains), reduce horizontal displacements of the embankment and of the foundation soil, reduce differential settlements and construction time.

Different types of geosynthetic products are available to function as reinforcement. The most commonly used in embankments on soft soils are geogrids and woven geotextiles. Geocells and some geocomposites can also be employed but are still less used than geogrids and geotextiles. Geosynthetic reinforcement can be specified to the required tensile strength and stiffness to reinforce the embankment. Durability of the reinforcement has not been an issue under normal conditions, although due care must be taken when working with aggressive soils (high or low pH values), as well as with the possibility of mechanical damage during reinforcement installation and embankment construction.

There are many studies on the benefits of reinforcing embankments on soft soils with geosynthetics (e.g. Volman *et al.* 1977; Rowe and Soderman 1984; 1985; Schaefer and Duncan 1988; Delmas *et al.* 1990, Bergado *et al.* 1994, Loke *et al.* 1994, Rowe *et al.* 1995; Macedo and Palmeira 2003; Bergado and Teerawattanasuk 2008; Rankine *et al.* 2008; Macedo *et al.* 2009; Magnani *et al.* 2009; Oliveira and Lemos 2011; Palmeira 2012). Geosynthetics have also been employed as casing for granular columns to reduce embankment settlements (e.g. Ayadat and Hanna 2005; Raithel *et al.* 2005; Murugesan and Rajagopal 2006; Araujo 2009; Araujo *et al.* 2009).

Rowe and Soderman (1984; 1985) report the construction of trial embankments on a 3.3 m thick soft subgrade with an average undrained strength of 8 kPa. The tensile strength of the geotextile reinforcement used was 215 kN/m and its tensile stiffness 2000 kN/m. It was observed that the unreinforced and the reinforced embankments failed with heights equal to 1.75 m and 2.75 m, respectively (Rowe and Soderman, 1985). Fabric pull-out from the soil at fabric strains between 4% and 5% governed the failure of the reinforced embankment with a mobilization of a force in the reinforcement at this stage of the order of 60 kN/m. Predictions of embankment heights at failure using limit equilibrium method compared well with those of the experimental embankments.

Loke *et al.* (1994) presented the behaviour of two geosynthetic reinforced embankments and one unreinforced embankment with similar geometrical characteristics that were led to failure on an 11 m deep soft subgrade. One of the reinforced embankments was reinforced with four layers of non-woven geotextiles. The other reinforced embankment was reinforced with a single geotextile layer. The unreinforced embankment failed with a height of 4 m whereas the reinforced embankments failed with heights of 4.2 m and 6 m, respectively.

Rowe and Li (2005) reported results from field observations and finite element analyses on the behaviour of reinforced embankments on soft soils. The study concluded that the maximum reinforcement strains observed in the field under working conditions are usually lower than the design values. This can be attributed to a combination of the low shear strength adopted in design, partial consolidation of foundation soils during construction and working stress conditions. The use of reinforcement can reduce the number of construction stages and consequently shorten the construction time. Embankments can be safely constructed over peat soils using reinforcement in combination with appropriate construction rates. According to those authors the major effect of reinforcement is to reduce lateral spreading and increase stability.

Magnani *et al.* (2009) presented the behaviour of two reinforced test embankments built on a normally consolidated soft clay deposit underlying a top sand layer. The embankments were constructed close to undrained conditions in about 60 days. The mobilized tension forces (T) in reinforcements increased with the embankment height.

The works discussed above are some examples showing the benefits of geosynthetic reinforcement for the stabilization of embankments on soft soils. However, in some instances, inexperienced designers and contractors may take the stability of the work for granted because of the increasing success of the use of geosynthetics as reinforcement, overestimating its contribution.

Crude stability analyses during design and poor construction practices can also lead to failures and large deformations. This paper describes the performance of two reinforced bridge abutments, one of which almost failed during construction. A consistent and rather simple analysis of the problem could have predicted and avoided the instability mechanism observed in the field.

2. CHARACTERISTICS OF THE PROJECT

The reinforced abutments investigated in this paper were built during the duplication of the BR-101 highway, in the state of Santa Catarina, south of the Brazilian territory. This highway is very important for the transportation of agricultural and industrial goods, being located close to the sea shore in a region of very soft soil deposits. Figure 1 shows the region where the abutments were constructed, 2 km from the city of Tijucas, in the state of Santa Catarina.

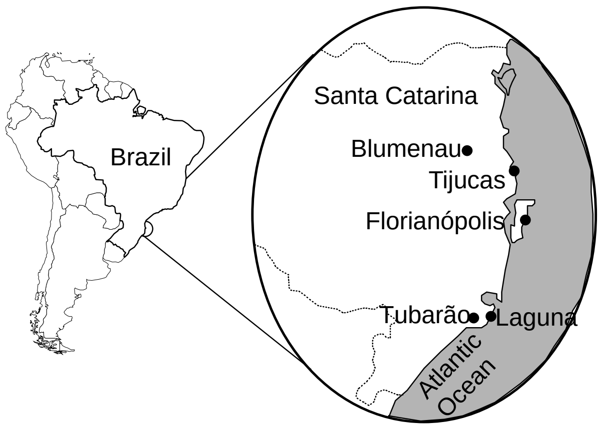
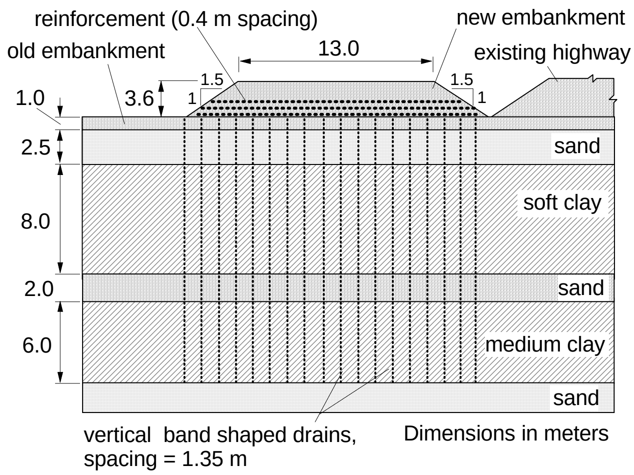


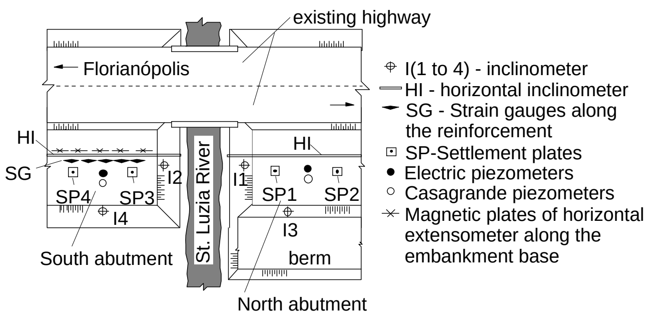
Figure 1 Location of the region of the reinforced abutments

For the widening of the highway in this part of the country the construction of a new embankment, 3.6 m high, was needed. This new embankment was built adjacent to an old one for the duplication of the highway. The geometrical characteristics of the project are presented in Figures 2(a) to (c) and the abutments will be hereafter referred to as North and South abutments. The inclination of the slopes of the abutments was equal to 1.5:1 (H:V). A stabilizing berm was constructed to avoid the progress of a failure mechanism in the lateral slope of the North Abutment (Figure 2a), to be described later in this paper. No berm was used in the South abutment.

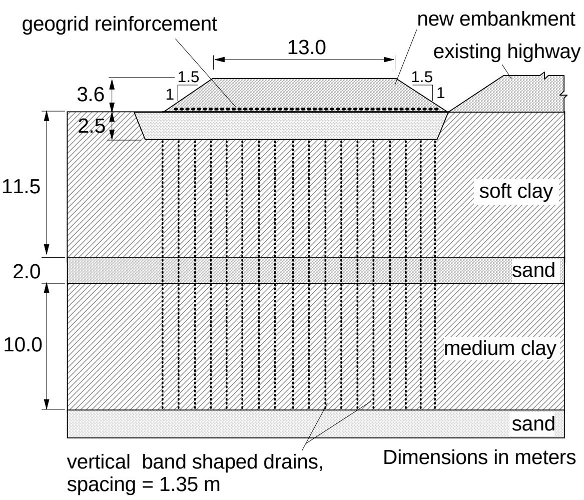
In the region of the North Abutment the depth of the soft deposit is equal to 23.5 m. A soft clay layer, 11.5 m thick, with undrained strength (S_u) varying between 5 kPa to 40 kPa, overlies a 2 m thick sand layer. Underneath the latter there is a 10 m thick medium to stiff clay, with S_u varying between 40 kPa and 90 kPa. Before the construction of the embankment for the North Abutment the first 2.5 m depth of the soft foundation soil was substituted by a sandy fill material (Figure 2b). Underneath the South Abutment (Figure 2c) the thickness of the soft clay layer is equal to 8 m and it underlies a 1 m thick old embankment and a 2.5 m thick sand layer. The medium to stiff clay layer below the South Abutment has a thickness of 6 m and underlies the 2 m thick sand layer. In both abutments the shallow soft clay layer controls the stability.



(a) Plan view and instruments location



(b) Cross-section of the North Abutment



(c) Cross-section of the South Abutment

Figure 2 Geometrical characteristics and instrumentation of the reinforced abutments

Table 1 summarises the main properties of the soft foundation soil. The range of variation of undrained strength with depth obtained by different field test techniques is depicted in Figure 3. As it is common for this type of deposit, a significant scatter in the value of S_u can be noted.

The fill material used to build the abutments was a random cohesionless one, with grain sizes varying from fine sand to blocks of rock. The fill material was installed under a loose state. The main geotechnical parameters of the fill material are 21 kN/m^3 unit weight and friction angle of 32^o obtained in direct shear tests.

For the North Abutment, the designers of the project specified a single layer of a uniaxial geogrid (more like a geostrip) made of polyester fibres protected by a polypropylene cover installed 0.4 m above the original foundation soil surface. The geogrid layer was 30 m long and its strength in the longitudinal direction was equal to 200 kN/m, whereas in its transverse direction it was equal to 15 kN/m. The values of tensile stiffness along the longitudinal and transverse directions were 1800 kN/m and 150 kN/m, respectively. The maximum tensile strain was equal to 12%. The reinforcement presents very different values of tensile strength and tensile stiffness in its longitudinal and transverse directions. During the installation of the reinforcement layers an overlapping length of only 200 mm was adopted with overlapping parallel to the embankment axis direction. As it will be seen later in this paper, this fact associated with improper geogrid orientation and construction speed, had a major influence on the abutment performance. The authors of this paper are unaware of the reasons for choosing an uniaxial grid as reinforcement. Probably, cost issues must have played a significant role in this decision. For the South Abutment the designers specified 3 layers (30 m long) of the same geogrid used as reinforcement in the North Abutment. The spacing between geogrid layers in this case was equal to 0.4 m.

Table 1 Properties of the soft foundation soil

Property	
Moisture content (%)	99.7
Void ratio	2.33
Liquid limit (%)	67
Plastic limit (%)	34
Percentage of organic matter (%)	5.4
Vertical coefficient of consolidation (m^2/year)$^{(1)}$	16.1 to 3.6
Effective cohesion (kPa)$^{(2)}$	2.7
Effective friction angle (degrees)$^{(2)}$	23
Horizontal coefficient of consolidation (m^2/year)$^{(3)}$	41 to 115
Maximum shear modulus (MPa)$^{(4)}$	6.1 to 8.5 and 7.7 to 13.2$^{(5)}$

Notes: (1) From laboratory consolidation tests in the 5 kPa to 100 kPa range (Carvalho 2000); (2) From isotropically consolidated undrained triaxial tests (Carvalho 2000); (3) From CPTU tests (Fahel 2003); (4) From seismic cone penetration tests (Fahel 2003); (5) North abutment between 3.0 m and 7.0 m deep and South abutment between 4.2 m and 8.3 m deep, respectively.

To accelerate consolidation settlements band shaped vertical drains were employed in both abutments. The drains were driven through the entire thickness of the soft deposit and had cross-section dimensions of 100 mm x 5 mm. The drains consisted of a plastic drainage core enveloped by a resin bonded nonwoven geotextile filter. They were installed according to a square pattern with spacing equal to 1.35 m. At the base of each embankment there is a drainage blanket made of sand with thickness of 0.4 m.

The instrumentation of the North Abutment consisted of two vertical inclinometers installed at the embankment slope facing the river and at the embankment lateral slope, respectively (Figure 2a). Settlement plates and a horizontal inclinometer provided measurements of settlements and settlement profiles along the abutment base. In the South Abutment, besides the instruments used in the North Abutment, there were also a magnetic extensometer probe at the base of the embankment, two electric piezometers (5.1 m and 8.0 m deep installed on the vertical passing by the interception of the diagonals formed by 4 neighbouring vertical drains) and strain gauges in the bottom reinforcement layer to measure reinforcement strains (Figure 4)

Additional information on the case-history reported in this paper can be found in Fahel (2003).

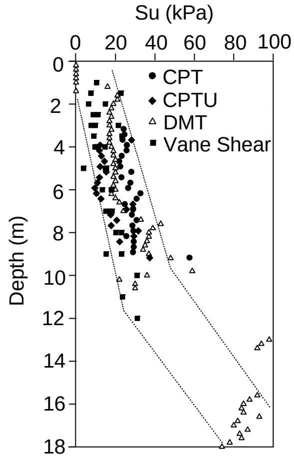


Figure 3 Variation of undrained shear strength with depth

Figure 4 Strain gauges along the bottom reinforcement layer of the South abutment

3. RESULTS

3.1 Performance of the North Abutment

3.1.1 Instability Mechanism of the Abutment Side Slope

Figure 5 presents the variation of embankment height versus time for the North Abutment. The embankment was built at a rate of approximately 0.35 m/day. Large cracks (Figure 6a) along the embankment longitudinal direction were noticed at the embankment surface when it reached the height of 3.6 m. This was a consequence of the initiation of a failure mechanism of the embankment side slope due to insufficient reinforcement tensile force mobilization along this direction. Excavation of the embankment confirmed sliding of adjacent reinforcement panels along the overlapping length (Figure 6b). Because of the clear signs of instability of the embankment, construction was interrupted and the designers and contractor decided to reduce the embankment height in approximately 2.5 m and to use the excavated material to build a 30 m wide stabilizing berm (Figure 2a) ahead of the lateral slope with a height of 1.1 m. After the reduction of embankment height and berm construction the embankment was left for 4 months for soft soil consolidation and shear strength increase before construction continued.

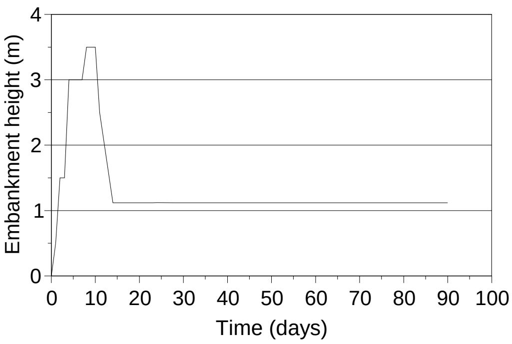


Figure 5 Variation of embankment height with time – North abutment

(a) Cracks along the embankment surface

(b) Location of reinforcement slippage

Figure 6 Instability of the embankment of the North abutment

3.1.2 Vertical Displacements

The evolution of the settlement profiles along the abutment axis with time from the horizontal inclinometer is depicted in Figure 7. Similar patterns of settlement profile can be noted up to an embankment height of 2.4 m. For a height of 3.6 m a significant increase of settlement can be observed as a consequence of the influence of the instability of the abutment lateral slope. The settlements continued to increase with time even under the lower embankment height (1.1 m) after fill excavation and berm construction due to consolidation of the soft foundation soil. Low values of settlements were observed at the embankment toe due to lower surcharge along the embankment slope facing the river in addition to the influence of the substitution of part of the soft foundation soil (Figure 2b).

3.1.3 Horizontal Displacements

Figure 8(a) presents the horizontal displacements profiles obtained by inclinometer I1 (Figure 2a) for the abutment slope facing the river. Maximum horizontal displacements of approximately 40 mm, at a depth of 4 m, can be noticed when the embankment reached a height of 3.6 m, at which the side slope showed signs of instability. An increase of 8 mm in the maximum horizontal displacement was noted after the reduction of embankment height.

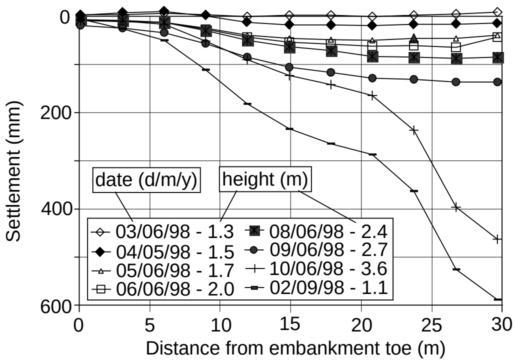
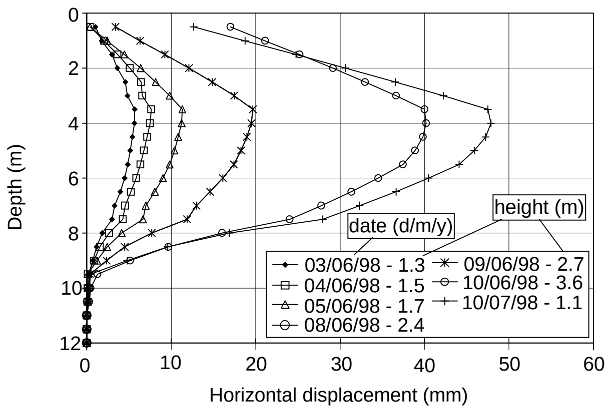


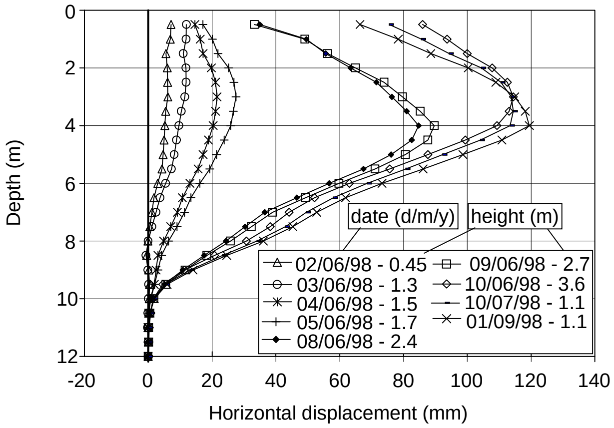
Figure 7 Abutment settlement profiles along its longitudinal axis

The horizontal displacements obtained from inclinometer I3 at the embankment lateral slope are shown in Figure 8(b). As for the vertical displacements, an abrupt increase in horizontal displacements of the foundation soil can be noticed for an embankment height of 2.4 m. At 3.6 m high (lateral slope failure initiation) the maximum horizontal displacement reached approximately 114 mm, of the order of 2.9 times greater than that observed for the slope facing the river at the same embankment height. This larger displacement was associated with less tensile stiffness and sliding of the reinforcement panels along the overlapping length in the direction normal to the abutment axis. Further increases on horizontal displacements of the foundation soil can also be noticed after the reduction of the embankment height.

The influence of reinforcement orientation and tensile properties on the maximum horizontal displacements registered by the inclinometers can also be viewed in the results in Figure 9. In this figure the maximum horizontal displacements measured in inclinometers I1 and I3 versus embankment height are presented. A similar pattern of behaviour and displacement magnitudes can be noted up to an embankment height of 1.3 m. Above this value the horizontal displacements increased at a much higher rate in the direction normal to the abutment axis as a consequence of lower reinforcement tensile force mobilization along that direction. In addition, the great embankment construction speed (Figure 5) employed certainly contributed to worsen the stability conditions of the embankment side slope.



(a) Inclinometer I1



(b) Inclinometer I3

Figure 8 Horizontal displacements from inclinometer readings

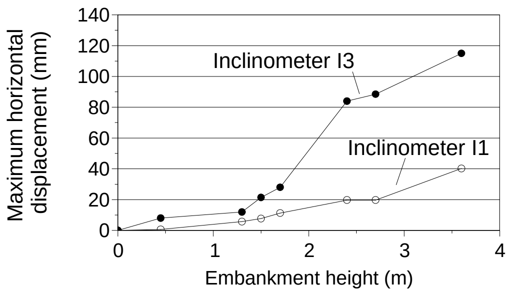


Figure 9 Maximum horizontal displacements in inclinometers I1 and I3 – North abutment

3.1.4 Stability Analysis

A stability analysis of the lateral slope of the abutment was carried out using limit equilibrium methods and the results of laboratory and field tests. The variation of undrained shear strength with depth obtained by field vane tests (Figure 3) was used in the calculations. The intention was to check if current and ordinary methods for slope stability would be able to predict failure of the embankment. Palmeira *et al.* (1998) examined the use of limit equilibrium methods for the calculation of factors of safety and embankment heights at failure, with good agreement between some traditional methods of stability analysis and results from case-histories for the conditions of the embankments studied. For the case of the North Abutment Bishop's slope stability method was chosen for the analysis and a software developed at the University of Brasilia (Palmeira 1998) for slope stability was employed. The relevant properties of the materials were those obtained in the field tests as well as from the geogrid manufacturer regarding geogrid tensile strength. Table 2 shows the results obtained in the analysis for the abutment slope facing the river and for its lateral slope for an embankment height of 3.6 m. The safety factor obtained for the lateral slope at this embankment height was equal to 1.07 and for the slope facing the river 1.23. As expected a greater (although still rather low) safety factor was obtained for the slope facing the river because of the greater reinforcement tensile strength of the reinforcement along this direction. It should be pointed out that the analysis did not consider the possibility of reinforcement panels sliding along the overlapping length. It is interesting to note that for an hypothetical unreinforced embankment under the same conditions as the actual one the factor of safety of the lateral slope would be equal to 1.05 (Table 2), close to the value (1.07) obtained for the reinforced embankment. Therefore, even if the sliding along the overlapping length had not occurred, the amount of reinforcement might be insufficient to guarantee the stability of the embankment lateral slope.

These results show that a simple and ordinary slope stability calculation would have been able to indicate the possibility of failure of the lateral abutment slope for the conditions found in the field. Even with a stronger reinforcement or a greater number of layers of the type of reinforcement used, instability of the lateral slope might still occur in case of improper or insufficient reinforcement overlapping length. In fact, joints of reinforcement panels parallel to the embankment axis should be avoided (Holtz *et al.* 1997).

Table 2 Results from slope stability analysis of the abutment slopes

Embankment slope	Reinforcement tensile strength available (kN/m)	Factor of Safety[(1)]
Lateral slope	15	1.07
Slope facing the river	200	1.23
Hypothetical unreinforced case	0	1.05

Note: (1) Factor of safety from Bishop's method (circular slip surfaces) for an embankment height of 3.6 m.

3.2 Performance of the South Abutment

3.2.1 Introduction

Because of the excessive deformations and almost complete failure of the North Abutment the designers decided to construct the South Abutment at a lower speed and reinforced with 3 geogrid layers. Figure 10 presents the variation of embankment height versus time for the South Abutment. In contrast to what was observed for the North Abutment, in this case the construction speed adopted was smaller, taking approximately 2 months to reach the final height (3.6 m) of the embankment. To some extent this greater amount of time to construct the South Abutment was also a consequence of construction activities disruptions due to weather constraints.

Three layers of the same geogrid used in the North Abutment were used in the South one, with a spacing of 0.4 m and the same orientation as in the North Abutment. No stabilizing berm was used in the lateral slope of the South Abutment. It should be noted that the soft soil thickness under this abutment is smaller than that under the North abutment. In addition, there is a 2.5 m thick sand layer at the foundation surface (Figure 2c), which improves the stability conditions of the embankment.

Although greater care was employed in the construction of the South Abutment, less information on its performance was made available to the authors of this paper. Nevertheless, the material available shows that the South Abutment performed well, in comparison to the North one, as will be seen in the following sections.

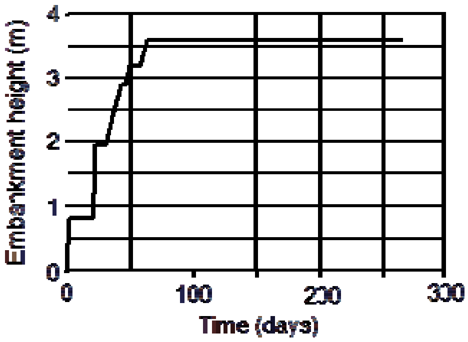


Figure 10 Embankment height versus time – South Abutment

3.2.2 Vertical Displacements

Figure 11 shows the settlement profiles of the embankment base at different times. After 47 days of construction a maximum vertical displacement of 271.5 mm was reached at a distance of 30 m from the embankment toe (towards the center of the embankment) and for an embankment height of 3.2 m. For the North Abutment, the settlement at the same distance from the abutment toe was equal to 130 mm for an embankment height of 2.7 m and 470 mm for an embankment height of 3.6 m. For the latter height the performance of the North Abutment was already affected by initiation of failure of the side slope. It should be noted the different times for which those heights were reached in each abutment. Longer time for consolidation was allowed in the case of the South Abutment, bearing in mind also the contribution of the vertical drains.

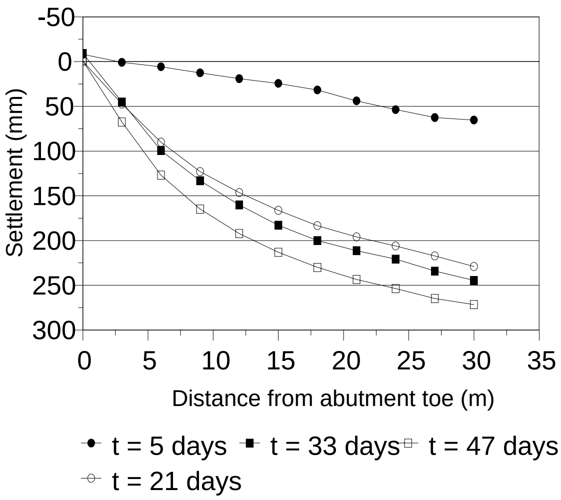


Figure 11 Settlement profiles at different times – South Abutment

Unfortunately, no settlement profiles were available to the authors for times greater than 47 days. However, Figure 12 shows the settlements measured in settlement plates SP3 (10 m from the abutment toe) and SP4 (25 m from the abutment toe, Figure 2a) up to 90 days after the start of embankment construction (embankment height of 3.6 m). The results obtained from the settlement plates are consistent with those from the horizontal inclinometer (Figure 11). It should be pointed out that the final embankment height of 3.6 m was reached in the South Abutment without any signs of instability.

3.2.3 Horizontal Displacements

Figure 13(a) and (b) show the variation of horizontal displacements and strains along the base of the embankment at different times obtained from the magnetic horizontal extensometer installed along the axis of the abutment. A maximum displacement of 18.8 mm was observed close to the embankment toe after 94 days of monitoring (Figure 13a). As a consequence of the horizontal displacement pattern, maximum strains were also observed close to the embankment toe (Figure 13b). Low strains (less than 0.19%) were observed. This was certainly due to the greater influence of the number and stiffness of the geogrid reinforcement along the abutment longitudinal direction as well as to the presence of the sand layer at the top of the foundation soil.

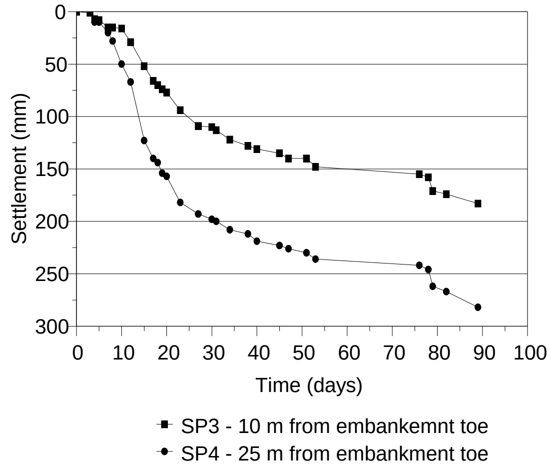
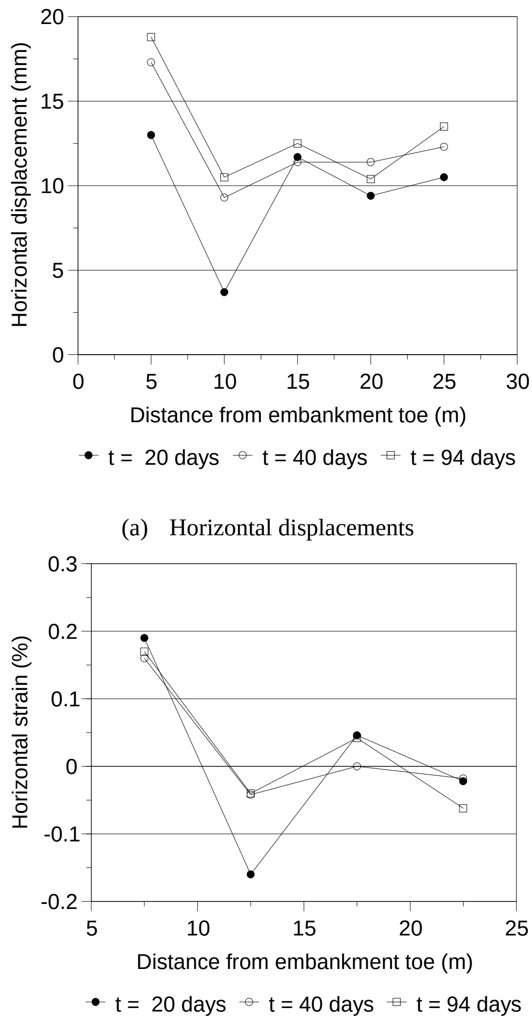


Figure 12 Settlements versus time from settlement plates SP3 and SP4 – South Abutment

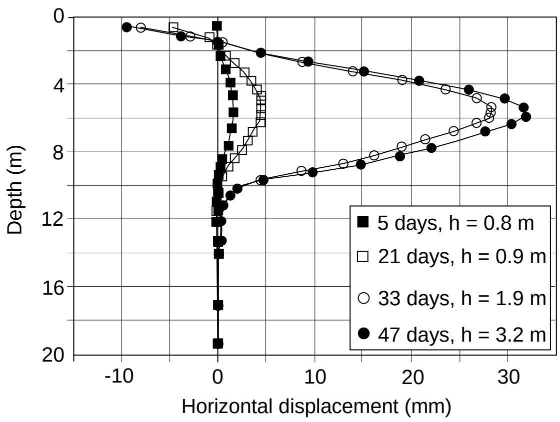


(a) Horizontal displacements

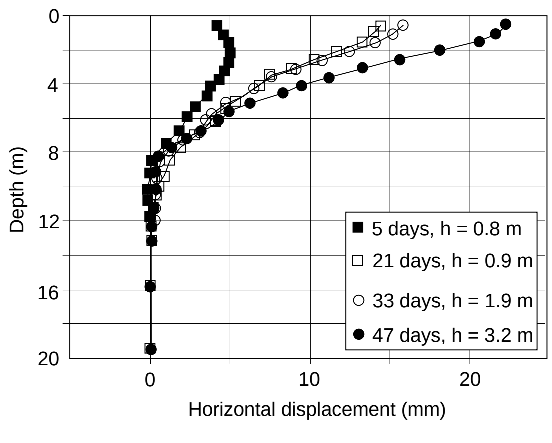
(b) Horizontal strains

Figure 13 Horizontal displacements and strains along the embankment base – South Abutment

The variation of horizontal displacements with depth as measured in inclinometers I2 (facing the river) and I4 (lateral abutment slope) are presented in Figures 14(a) and (b), respectively. The maximum horizontal displacement along the abutment axis 47 days after the start of construction (embankment height of 3.2 m) was equal to 32 mm. Along the direction normal to the abutment axis at the same time the maximum horizontal displacement was equal to 26 mm. For an embankment height between 2.7 m and 3.6 m the North Abutment presented a maximum horizontal displacement towards the river between 40 mm and 48 mm, which is greater than that of the South Abutment along the same direction. In the direction normal to the South Abutment axis the maximum horizontal displacements occurred close to the foundation surface with a value of 26 mm for an embankment height of 3.2 m, which is considerably smaller than the values 89 mm and 114 mm measured in the North Abutment for embankment heights of 2.7 m and 3.6 m, respectively. Thus the horizontal displacements of the South Abutment normal to its axis were considerably smaller than those of the North Abutment. That was certainly due to the greater number of reinforcement layers, presence of the sand layer at the foundation top, lower soft soil thickness, lower construction speed and probably stronger joints of the reinforcement panels along the embankment transversal direction.



(a) Inclinometer I2



(b) Inclinometer I4

Figure 14 Horizontal displacements of the soft soil – South Abutment

3.2.4 Pore pressures

Figure 15 presents the variation of excess pore pressure versus time in piezometers P1 (depth of 5.1 m) and P2 (depth of 8 m). The results show low maximum excess pore pressures (13 kPa in piezometer P1 and 14.5 kPa in piezometer P2). A faster rate of pore pressure increase was noticed in the first days of embankment construction, with continuous dissipation in piezometer P1, whereas the excess pore pressure continued to increase in piezometer P2 until 30 days from the beginning of construction. Some dissipation in this piezometer started only after the end of embankment construction.

The low values of excess pore pressure can be a result of the influence of the presence of drainage layers close to the piezometers tips. In the case of P1, this piezometer is only 2.6 m far from the 2.5 m sand layer at the top of the foundation soil and 0.95 m away from the closest vertical drains. Some delay in the responses of the piezometers can be noted with regard to the beginning of embankment construction. This, and to some extent the low excess pore pressure values measured and low dissipation rate in piezometer P2, can be associated to loss of saturation of the porous stones at the piezometers tips. Some level of clogging of the porous stones cannot be discarded either.

3.2.5 Reinforcement Strains

Geogrid tensile strains along the abutment longitudinal axis are depicted in Figure 16. Very low levels of geogrid strains can be noted up to 262 days after the beginning of embankment construction (peak strain values of 0.23% and 0.27% at 7 m and 16.5 m away from the embankment toe, respectively). There was a reduction of strain magnitudes between 10 m and 13 m away from the embankment toe. It is interesting to note that the variation of tensile strains along the geogrid length is consistent with the pattern and magnitude of horizontal strains measured along the embankment base (Figure 13b). Beyond 20 m from the embankment toe the reinforcement strains were very low. Such low strain levels show that the embankment slope facing the river was far from failure, as a consequence of the number of reinforcement layers and more favourable conditions to slope stability than those of the North Abutment, as far as the foundation soil is concerned.

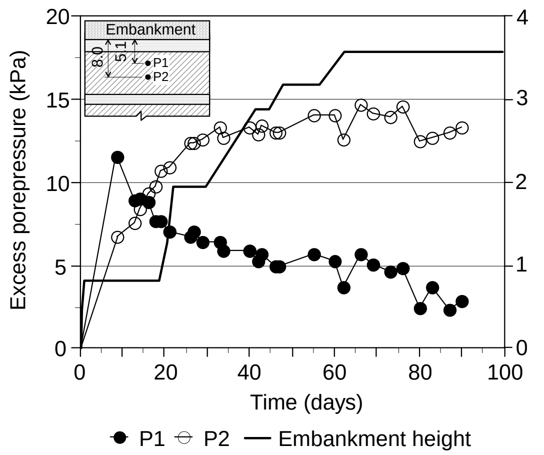


Figure 15 Excess pore pressure versus time – South Abutment

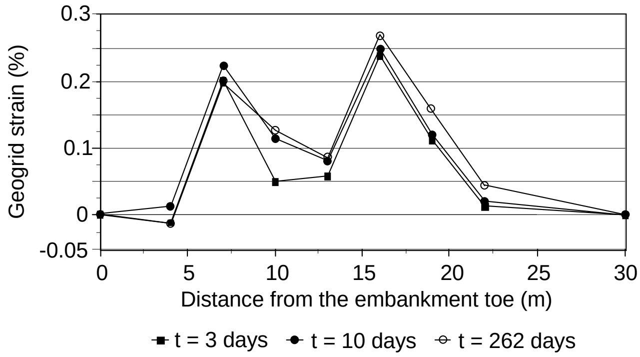


Figure 16 Variation of tensile strain along geogrid length – South Abutment

4. CONCLUSIONS

This paper presented the performance of two geogrid reinforced abutments constructed on soft soils. The results obtained highlight the importance of the use of geosynthetic reinforcement to increase embankment stability. However, in one of the case-histories examined the benefits that might be brought by the reinforcement presence were somewhat reduced due to an improper choice of reinforcement type or lack of proper reinforcement orientation in addition to weak reinforcement panels' joints. The use of a uniaxial geogrid (or geostrip) was barely sufficient to stabilize the embankment slope facing the river in the North Abutment, whereas an initiation of a failure mechanism took place in the embankment side slope due to less reinforcement force mobilization along the transversal direction of the abutment. An ordinary limit equilibrium slope stability analysis would have indicated that less reinforcement than actually needed was being employed.

In the embankment of the South Abutment the designers increased the number of reinforcement layers, but kept the direction of greater tensile strength and stiffness coinciding with the abutment longitudinal direction. In this embankment advantage was taken of the presence of a sand layer at the top of the foundation soil, which improved the stability conditions of the abutment slopes.

Commonly, the tight schedule for designing and constructing embankments on soft soil puts the designers in situations where quick decisions must be taken to obey schedule constraints. Probably, these were the conditions the designers of the abutments reported in this paper had to face. Nevertheless, the results obtained highlight the need for proper stability analysis, reinforcement specification and installation in reinforced embankments on soft foundation soils.

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