Effects of large differential settlements on embankments on soft soils

E.M. Palmeira & A.R.S. Fahel
University of Brasilia, Brasilia, Brazil

Keywords: : Geosynthetics, Reinforcement, Soft Soils, Embankments, Settlements

ABSTRACT: This paper presents the performances of four geosynthetic reinforcement abutments built on soft foundations. The embankments are part of highway developments in the north-east and south regions of Brazil. The first embankment is a geogrid reinforced bridge abutment constructed as part of the BR-101 highway-widening programme. In this case the new embankment was built with part of its width resting on the existing embankment. The other three reinforced embankments were built in the Linha Verde highway to allow the crossing of rivers in the region. One of these embankments was built on a concrete slab and on piles with caps, another on a group of piles with caps and the last one one directly on a top sand layer overlying a compressible clay layer. The designers of these works heavily reinforced the abutment, but little concern was put on the effects of differential settlements caused by foundation consolidation and erosion. This paper shows that, in spite of the large deformations imposed to the structures, the overall performances of the embankments were satisfactory mainly because of the flexible nature of the geosynthetic reinforced mass.

INTRODUCTION

Geosynthetics can be effectively used to reinforce embankments on soft soils. High strength and modulus geosynthetic reinforcements can be used to save construction time, to allow the use of embankment steeper slopes and to provide short or long term overall stability for the embankment. Large and some times rather deep soft soil deposits are frequently found in different regions of Brazil. Under these circumstances the construction of the embankment can be very problematic and failures can cause important delays in the construction and considerable additional costs to repair the work. When one deals with abutments on soft deposits the problem is even more complex, because of the characteristics of the subgrade, the geometry of the problem and the proximity of structures sensitive to lateral ground movement that are likely to be induced by the embankment construction.

This paper describes some damages caused by excessive settlements of embankments on soft soils built adjacent to existing structures. Four different case histories will be addressed. Three are reinforced abutments and one is the work of widening an existing highway.

1 CHARACTERISTICS OF THE SITES AND MATERIALS

The locations of the sites considered in this work are presented in Figure 1. Case history 1 is an abutment built as part of the widening of the BR-101 highway, in the south of Brazil. Case histories 2 to 4 are reinforced abutments built in the Linha Verde highway, in the north east region of Brazil. Both highways are very important to those regions economies and tourism.

1.1 *Case History 1*

Case history 1 is a reinforced abutment built as part of the programme of duplication of the BR-101 highway, in Santa Catarina, Brazil. The abutment provides access to a bridge over the river Infern-inho.The reinforced embankment final height is equal to 6.0 m, but during construction an additional 1.5 m high fill surcharge was installed to act in combination with vertical band shaped drains to accelerate consolidation settlements (Fig. 2 a to c). The vertical drains (100 x 5 mm) were spaced 1.3 m and installed along a length of 30 m from the embankment toe. The vertical drainage system was designed so as to provide 80% consolidation in a period of 8 months. Part of the new embankment rests on the side slope of the existing embankment and an excavation of the berm of the latter was made before the construction of the new embankment. Eight layers of geogrid reinforcement spaced 0.4 m were installed in the embankment base. A 30 m wide berm was also prescribed by the designers for the stabilisation of the side slope of the new embankment. The need for this berm can only be explained by the smaller reinforcement strength along the embankment transversal direction, as described below. However, the berm might have been avoided (or shortened) had the strongest reinforcement direction been also orientated along the embankment transversal direction or biaxial grids or geotextiles had been used.

The fill material used in Case History 1 was a coarse sand with a unit weight of 15.5 kN/m^3 and a friction angle of 33^o, determined by direct shear test. The foundation soil in the site is a 12 to 17 m deep soft clay deposit with the presence of sand layers (Fig. 2 a). The undrained strength obtained from CPTU and vane tests varied between 20 and 40 kPa.

Polyester geogrid layers with a polypropylene cover were used to reinforce the embankment. The geogrid tensile strengths along its longitudinal and transverse directions are 200 kN/m and 15 kN/m, respectively. The grid tensile strain at failure is equal to 12% and its tensile stiffness approximately 1800 kN/m. The grid strongest direction was orientated along the embankment longitudinal axis, as commented above.

The instrumentation of the embankment consisted of Casagrande and pneumatic piezometers, settlement plates, inclinometers, magnetic extensometers, a full profile settlement gauge and geogrid strain measurement devices (Fig. 2a).

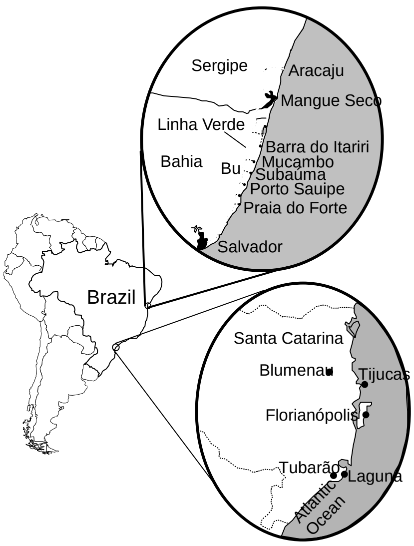


Figure 1. Location of the case histories

1.2 *Case History 2*

Case history 2 is a reinforced abutment built to cross the river Mucambo. The main geometrical characteristics of the abutment are presented in Figure 3 and Table 1. The abutment is 2.7 m high and provides access to a 60 m span bridge. Part of the reinforced mass was built on a concrete slab resting on piles (0.25 m diameter, 1.25 m spacing in a square pattern). The reinforcement length was 5.3 m in the upper part of the structure and 3.25 m in the lower part. The reinforcement layers were installed with a spacing of 0.3 m. Wall facing units consisted in "L" shaped concrete elements (1 m x 0.55 m x 0.6 m and 0.09 m thick). The unreinforced region of the embankment was built on piles with caps (1 m x 1 m in plan dimensions and 0.3 m high).

A fine sand whose relevant characteristics are presented in Table 2 was used to build the embankment. The foundation of this abutment consists of a 2m thick layer of clayey sand overlying a 13m thick layer of soft organic clay, with undrained strength obtained from in situ vane tests varying between 15 and 45 kPa (Fig. 4).

The reinforcement used in the Case History 2 abutment was a non woven geotextile made of continuous polyester fibres, with a mass per unit area of 300 g/m^2. Its tensile strength, strain at failure and secant tensile stiffness (at 50% maximum tensile load) obtained from in-isolation wide strip tests (ASTM D4595, ASTM, 1996) are equal to 20 kN/m, 45 % and 50 kN/m, respectively. Although being a weak and extensible reinforcement, the reinforcement length and spacing between reinforcement layers provided a very conservative design with regard to the embankment stability. The authors believe that the designers of case histories 2 to 4 intended a stiff reinforced mass by adopting a low value of reinforcement spacing.

Table 1. Characteristics of Case Histories 2 to 4.

Case History	h (m)	s (m)	l_t (m)	l_b (m)	n	Foundation treatment $^{(3)}$
2	2.7	0.3	5.3	3.25	8	piles
3	7.3	0.2 - 0.3$^{(4)}$	3.2	8.9	28	piles
4	2.0	0.3	3.2	3.2	7	none

Notes:
(1) See also Figures 3, 5 and 6;
(2) h = height of the reinforced structure, s = spacing between reinforcement layers, l_t = length of the reinforcement layers in the upper part of the structure, l_b = length of the reinforcement layers in the lower part of the structure, n = number of reinforcement layers;
(3) Type of solution for load-transference to stronger foundation layers below the reinforced zone;
(4) 0.2 m spacing between reinforcements along the lower part of the structure (first 2.5 m from the base) and 0.30 m spacing along the upper part of the structure.

Table 2. Characteristics of the fill materials in Case Histories 2 to 4.

Case History	D_{10} (mm)	D_{50} (mm)	CU	γ (kN/m^3)	c' (kPa)	ϕ (deg.)
2	NA$^{(4)}$	NA	NA	18.5	0	30
3	0.0001	0.25	3000	20.2	16.3	41
4	0.001	0.20	280	19.9	31.8	36

Notes:
(1) D_{10} and D_{50} = particle diameters corresponding to 10 and 50% passing, respectively, CU = coefficient of uniformity (= D_{60}/D_{10});
(2) γ, c' and ϕ' = specific weight, effective cohesion and effective friction angle at optimum moisture content, respectively;
(3) c' and ϕ' obtained from drained direct shear tests;
(4) NA = value not available.

1.3 *Case History 3*

The reinforced soil mass in Case History 3 is 7.3 m high and its main characteristics are shown in Figure 5 and Table 1. In this case the reinforced abutment was constructed to allow the crossing of the Bu river and its reinforced mass was supported by a group of 0.25 m diameter concrete piles with caps (1 m x 1 m x 0.3 m), with a spacing between piles of 1.25 m. The distribution of reinforcement along the wall height was divided in two parts, as shown in Figure 5. In the lower region of the structure, up to 2.5 m above the wall base, the reinforcement spacing used was equal to 0.2 m with reinforcement length of 8.9 m. In the upper region the reinforcement spacing was increased to 0.3 m and the reinforcement length reduced to 3.2 m.

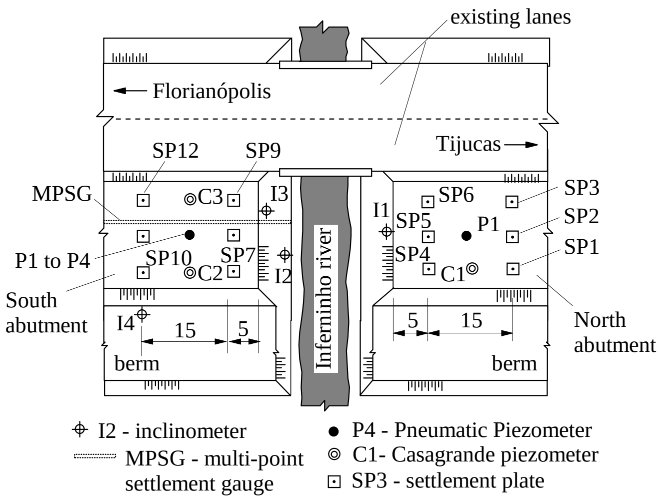
A clayey sand was used as fill material for Case History 3 (Table 2) and the foundation soil consisted of a 2.5 m thick top clayey sand layer on a 3.6 m thick organic silty clay deposit.

The reinforcement type and wall facing units used in Case History 3 were the same described for Case History 2.

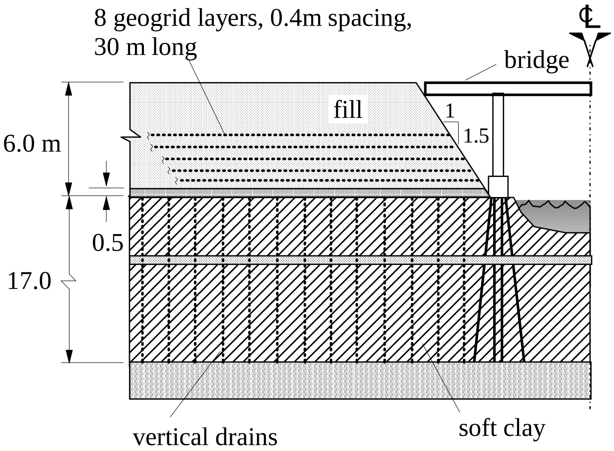
As discussed before for the previous case history described, the designers were also very conservative with respect to the reinforcement spacing adopted. On the other hand, with regard to the wall height and reinforcement length adopted in the upper part of the wall, the designers were rather daring.

1.4 *Case History 4*

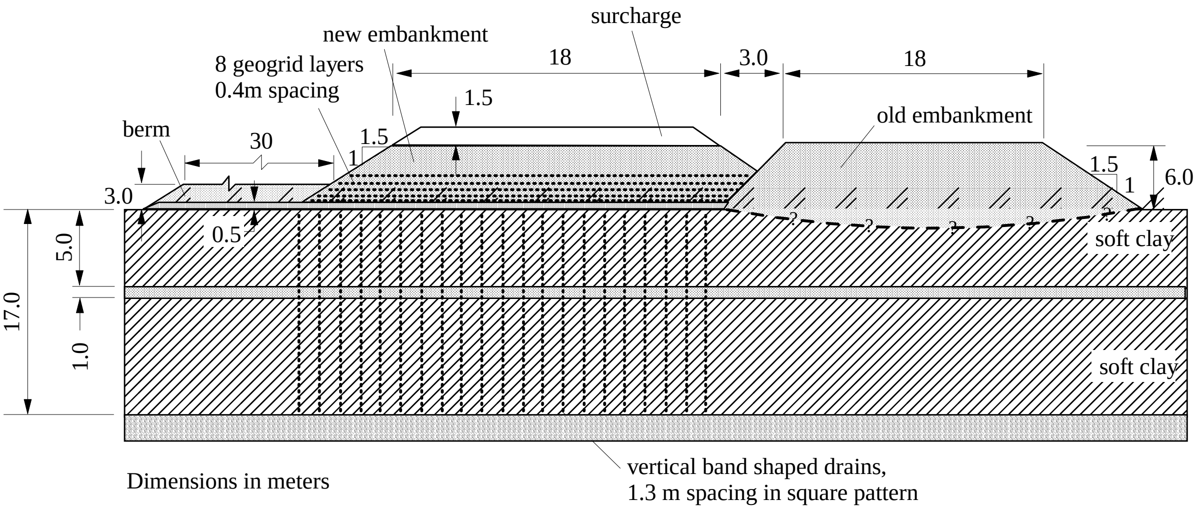
The reinforced abutment referred in this paper as Case History 4 was built to allow the crossing of the river Sauipe. Its geometrical characteristics are presented in Figure 6 and Table 1. The reinforced embankmet is rather short, with a height of 2 m, but constructed directly on the compressible foundation. Because of the characteristics of the foundation soil, large consolidation settlements could have been anticipated during the design stage of this abutment. Note that no provision was made by the designers to minimise embankment settlements. As in Case Histories 2 and 3, the abutment was heavily reinforced with reinforcement layers spacing of 0.3 m and uniform reinforcement length equal to 3.2 m.



(a) Plan view of the embankment of Case History 1



(b) Cross-section along the longitudinal direction of the embankment of Case History 1



(c) Cross-Section along the transversal direction of the embankment of Case History 1

Figure 2. Characteristics of the reinforced embankment of Case History 1.

The fill material used to build the embankment in Case History 4 was a fine sand (Table 2). The foundation soil consisted of a 4.5 m thick top layer of clayey sand overlying a 5.7 m thick organic clay layer.

The same types of wall facing units and geotextile reinforcement described for Case History 2 were used in Case History 4.

Additional information on the case histories presented in this paper can be found in Fahel (1998) and Fahel et al. (2000).

2 PERFORMANCE OF THE REINFORCED ABUTMENTS

2.1 *Case History 1*

The presence of the geogrid reinforcement had an important effect in reducing horizontal displacements of the foundation soil. This can be visualised in the results obtained by the inclinometers I3 and I4 (Fig. 2 a) shown in Figures 7 (a) and (b). The foundation soil movements were smaller along the embankment longitudinal direction even with a steeper slope close to the river, no berm and the proximity to the river channel. This direction coincides with the orientation of the reinforcement having greatest tensile strength and stiffness. In spite of the presence of the 30 m wide berm the displacements along the embankment transversal direction (weaker reinforcement direction) were rather large. This results suggest that the use of the berm might have been avoided with an appropriate reinforcement of the embankment transversal direction.

The consolidation of the soft foundation soil, accelerated by the vertical drains, caused some cracks on the existing embankment, as can be seen in Figures 8 (a) and (b). The fact that part of the new embankment rested on the old one pulled the latter down during consolidation, causing the cracks. However, these cracks occurred outside the traffic lanes and were of minor consequences to the existing embankment. The presence of the reinforcement in the new embankment is likely to have been beneficial in minimising damages to the existing embankment. This aspect is under investigation using numerical analyses (Fahel, 2000).

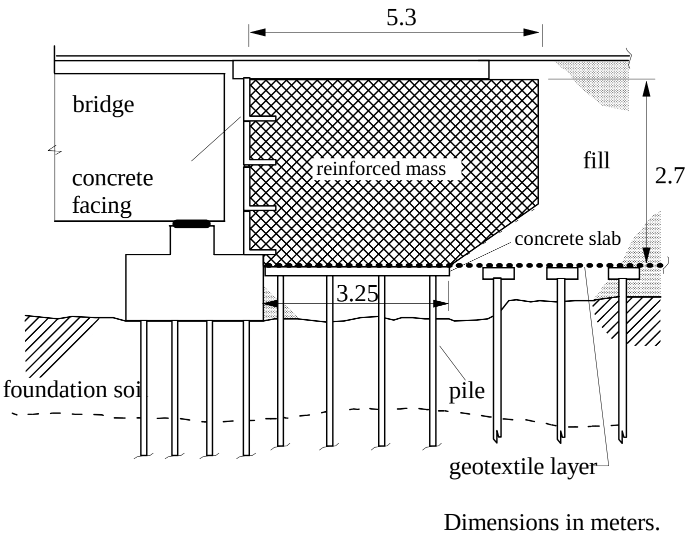


Figure 3. Cross-section of the reinforced abutment of Case

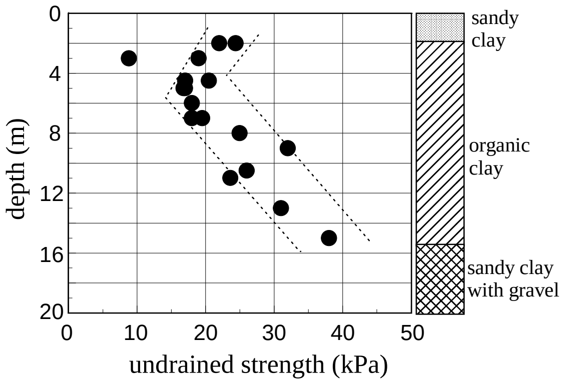


Figure 4. Undrained strength vesus depth – Case History 2

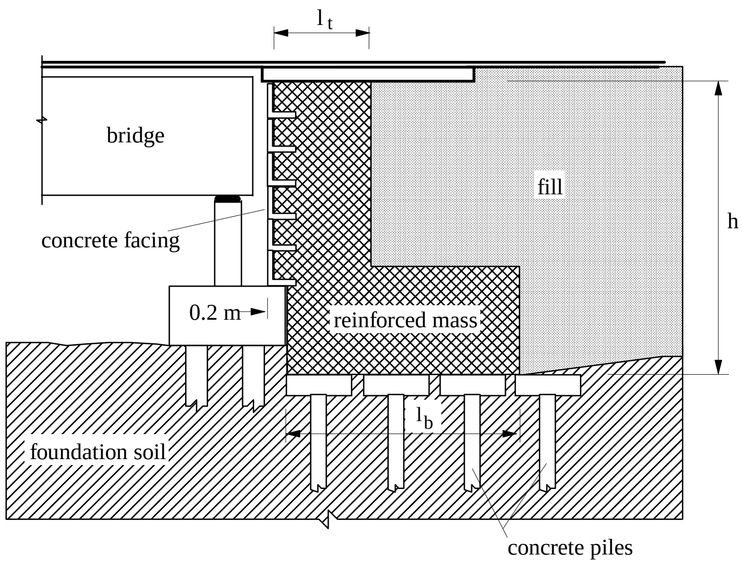


Figure 5. Cross-section of the reinforced abutment of Case History 3.

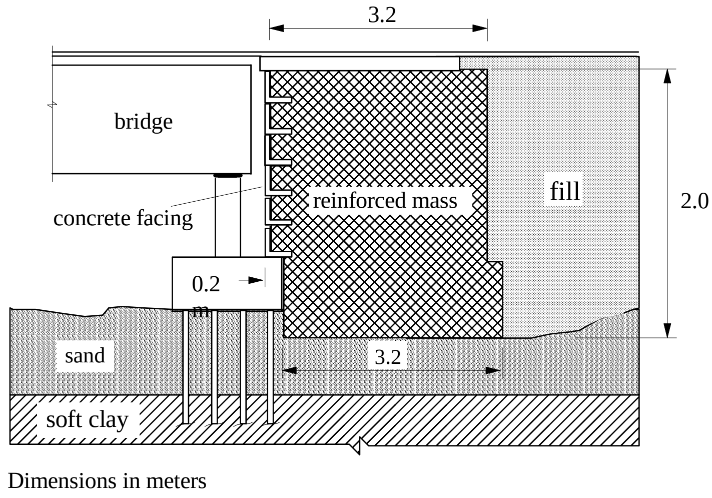
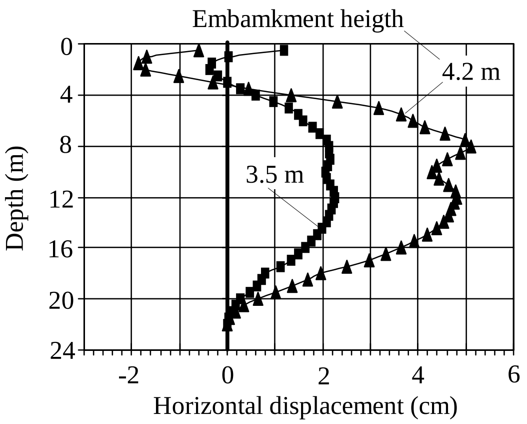
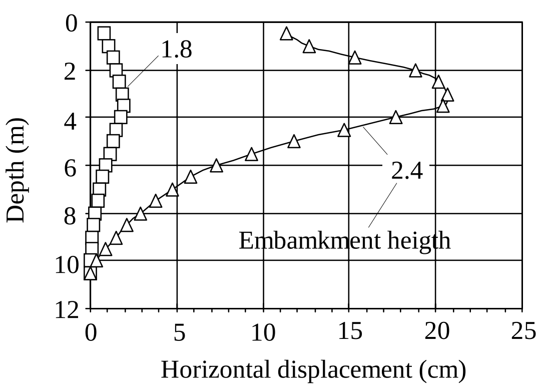


Figure 6. Cross-section of the reinforced abutment of Case History 4.



(a) Inclinometer I3 – Displacements towards the north direction.



(b) Inclinometer I4 – Displacements towards the east direction.

Figure 7. Horizontal displacements in Case History 1.

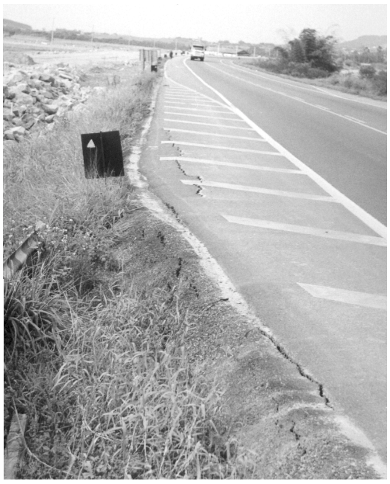
A similar mechanism of damage of a existing highway embankment due to the construction of a new one is reported in Fahel et al. (2000).

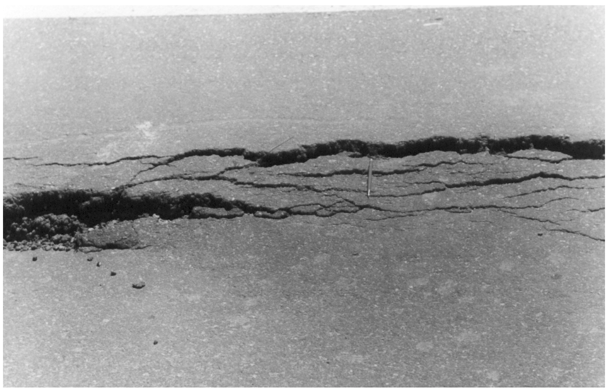
2.2 *Case History 2*

The reinforced abutment in Case History 2 was not affected by foundation settlements because most of the reinforced mass rested on a piled slab (Fig. 3). However, significant damage was caused to one of the lateral wall faces because of erosion of the fill material underneath the rear part of the reinforced mass, as shown in Figure 9. A river flood caused the removal of the soil below the base of the reinforced mass adjacent to the concrete slab, causing a severe differential settlement in that region and the collapse of some facing units. In spite of the large distortion of the reinforced mass, only some rather minor repairs were needed along the pavement surface. This behaviour of the geosynthetic reinforced mass emphasises the benefits of having a flexible retaining structure in this type of work, capable of accommodating large settlements with relatively minor damages.

2.3 *Case History 3*

No significant settlements were observed in Case History 3, which was the one having the highest reinforced structure (7.3 m). This was certainly due to the efficiency of the piles along the embankment base. However, horizontal displacements of the wall crest of the order of 40 mm (approximately 0.55% of the wall height) were measured, as shown in Figure 10. Bearing in mind that the wall was heavily reinforced, that value of displacement and the pattern a wall face deformation must be associated with the reduction of reinforcement length in the upper region of the wall.





(a) Cracks along the edge of the pavement.

(b) Detailed view of the cracks.

Figure 8. Damages to the pavement in Case History 1.

2.4 *Case History 4*

The reinforced abutment in Case History 4 was the one that presented the largest overall deformations, because of the lack of measures to minimise surface settlement, such as the use of piles. For this abutment the measured settlement at the wall face was equal to 290 mm. Settlement were not noticeable beyond a distance of 21 m from the wall face. The maximum horizontal displacement of the wall crest was equal to 55 mm (2.8% of the wall height) and the wall face rotated with respect of its crest, as can be visualised in Figure 11. Similar mechanisms of reinforced walls on soft subgrade have been identified in model scale testing and in numerical analyses (Monte, 1996, Dellabianca, 1999).

The displacements and rotation of the wall face caused relative displacements of the wall facing units of 90 mm, as well as some cracks (Fig. 11). The differential settlement also required a rather heavy maintenance work of the pavement surface, as shown in Figure 12 (note bent guard-rails along the sides of the highway).

Figure 9. Facing units colapse in Case History 2.

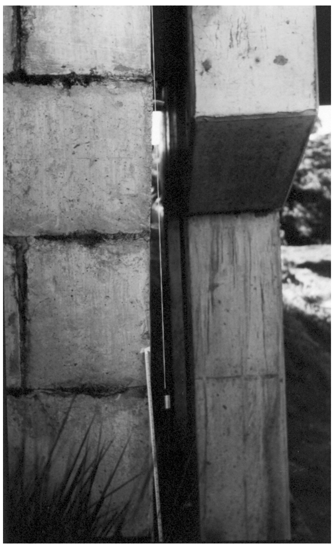


Figure 10. Wall face displacement in Case History 3.

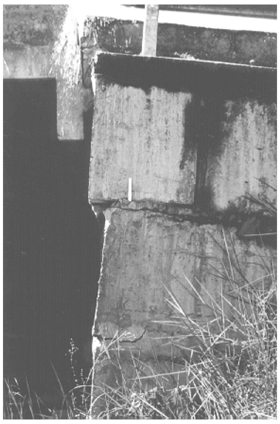


Figure 11. Rotation of wall face and facing units in Case History 4.

Figure 12. Repairs in the pavement of Case History 4.

3 CONCLUSIONS

This paper described the performance of reinforced embankments subjected to large differential settlements. The following conclusions can be drawn:

- The construction of a new embankment adjacent to an existing one is a complex problem. Even if the consolidation of the soft foundation soil is accelerated by means of vertical drains, some damages to the existing embankments are likely to be caused. In this sense the use of geosynthetic reinforcement provides a good solution to minimise those damages.
- The performance of the embankment in Case History 1 showed that the reinforcement was very effective in reducing lateral movements of the soft foundation soil.
- Some of the problems in the reinforced structures reported in this paper could have been anticipated and avoided had the designers properly considered the effects of foundation settlements.
- The extensibility of the geosynthetic reinforced structure proved to be an important advantage for this type of work when differential settlements can occur.

ACKNOWLEDGEMENTS

The authors are indebted to the following persons and institutions for supporting the research described in this paper: University of Brasilia, CNPq-Brazilian National Research Council, Prof. J.A.R. Ortigao (Federal University of Rio de Janeiro), Mr. E.C. Fonseca and Prof. L.E.P. Campos (Federal University of Bahia), DERBA-Bahia State Highway Department, Bidim-BBA Geosynthetics and CAPES/Brazilian Ministry of Education.

REFERENCES

ASTM 1991. *ASTM Standards on Geosynthetics*, American Society of Testing Materials, USA, 104 p.

Dellabianca, L.M. 1999. *Numerical Analysis of Geosynthetic Reinforced Retaining Structures*. MSc. Thesis, University of Brasilia, Brazil, (in Portuguese), 141 p.

Fahel, A.R.S. 1998. *Instabilities and Construction Problems in Geosynthetic Reinforced Structures*. MSc. Thesis, University of Brasilia, Brazil, (in Portuguese), 117 p.

Fahel, A.R.S., Palmeira, E.M. and Ortigao, J.A.R. 2000. Behaviour of Geogrid Reinforced Abutments on Soft Soil in the BR 101-SC Highway, Brazil. Proc. of *GeoDenver'2000*, ASCE, Denver Co., USA (in press).

Fahel, A.R. 2000. *Geosynthetic Reinforced Abutments on Soft Soils*. PhD. Thesis, University of Brasilia, Brazil, (in progress).

Monte, L.M. 1996. A Study of the Deformation and Failure Mechanisms of Reinforced Embankments on Soft Subgrades. MSc Thesis, University of Brasilia, Brazil, (in Portuguese), 124 p.

Palmeira, E.M. and Fahel, A.R.S., 2000. Lessons Learned from Failures of Wall Facings in Two Geotextile Reinforced Walls. *Lessons Learned from Failures Associated with Geosynthetics*, Editor: J.P. Giroud, in progress.