GeoAmericas2020
4th PAN AMERICAN CONFERENCE ON GEOSYNTHETICS
26-29 APRIL 2020 • RIO DE JANEIRO • BRAZIL

A Model Study on the Performance of Piled Embankments on Soft Soils and the Influence of Reinforcement Damage

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ABSTRACT
Soil deposits along a significant part of the Brazilian coastline consist of plains with geotechnical profiles presenting a high water table level, low strength and high compressibility. With urban growth, areas where the thickness of the soft clay deposit can reach up to 40 m are becoming more and more occupied. In this scenario, reinforced embankments on piles have been increasingly used to stabilize embankments over soft soils. The presence of the reinforcement reduces the stresses transferred to the soft foundation and improves the efficacy of the transference of loads to the piles. Therefore, significant reductions in fill settlements and in lateral displacements of the soft soil can be obtained. However, the design of this type of work is still complex. Several researchers have studied this technique with the purpose of improving the understanding of the mechanical behavior of these embankments, either by means of in situ instrumentation, numerical analysis or laboratory tests on physical models. This paper investigates the effects of mechanical damages in the reinforcement on the performance of the system. Three different reinforcement types, including a geogrid and two geotextiles, were tested with varying values of tensile stiffness. A typical piled embankment section consisting of an instrumented fill layer subjected to surface surcharges of up to 40 kPa (200 kPa under prototype conditions) was simulated in the laboratory. Test measurements are presented and discussed. The results obtained show the levels of mechanical damage to the reinforcement and their influence on embankment performance.

RESUMO
Os depósitos de solos ao longo de áreas da zona costeira do Brasil comumente se constituem em planícies com perfis geotécnicos que apresentam elevado nível do lençol freático, baixa resistência e alta compressibilidade. Com o crescimento urbano, as áreas onde a espessura do depósito da argila mole pode atingir até 40 m estão sendo cada vez mais ocupadas. Neste cenário, aterros reforçados sobre estacas têm sido muito usados para estabilizar aterros sobre solos moles. A presença do reforço reduz as tensões transferidas para a fundação mole e melhora a eficácia da transferência de cargas às estacas. Portanto, reduções significativas nos recalques do aterro e nos deslocamentos laterais do solo mole podem ser obtidas. Entretanto, o projeto desse tipo de estrutura é ainda complexo. Diversos pesquisadores têm estudado esta técnica com o propósito de melhorar o entendimento do comportamento mecânico desses aterros, seja por meio de instrumentação de obras, análises numéricas ou ensaios de laboratório em modelos físicos. Este artigo investiga os efeitos de danos mecânicos no reforço sobre o desempenho do sistema. Três tipos diferentes de reforços, incluindo uma geogrelha e dois geotêxteis, foram ensaiados com variação dos valores de rigidez à tração. Uma seção típica de aterro estaqueado consistindo de camadas de preenchimento instrumentadas, submetida a sobrecargas de até 40 kPa (200 kPa sob condições reais), foram simuladas no laboratório. Medições de ensaios são apresentadas e discutidas. Os resultados obtidos mostram os níveis de dano mecânico ao reforço e sua influência no desempenho do aterro.

1. INTRODUCTION

A significant amount of Brazilians live in cities located in the coastal zone of the country. The geotechnical profiles presented at these locations may consist of clayey layers with low bearing capacity and high compressibility. In Rio de Janeiro, for example, deposits of soft to very soft clay of considerable thickness can be found (Almeida and Marques 2011). The design and construction of embankments on these types of subgrades is a difficult task for geotechnical engineers, due to the possibility of embankment rupture and significant settlements.

Several solutions have been developed over the years to provide improvements to the foundation soil (staged construction of embankments, use of vertical drains and of granular columns, for instance). However, often it is not possible to wait for soft soil to acquire stiffness and strength due to the consolidation process. Thus, among the many possible solutions, the technique called piled embankment deserves special mention, which is usually favourable for short schedules and has low cost compared to other solutions (Aslam 2008).

The performance of piled embankments on soft grounds can be improved by the presence of the geosynthetic reinforcement layer. In this context, several works in the literature have reported good-performance of geosynthetic-reinforced piled embankments (Jones et al. 1990; Jenner et al. 1998; Alexiew and Gartung 1999; Alexiew and Vogel 2001; Hsi 2001; Habib et al. 2002; Raithel et al. 2002; Arulrajah et al. 2003; Atalar et al. 2003; Vega-Meyer and Shao 2005; Rowe and Liu 2015). Some case-histories and field and laboratory experiments are available in the literature (Collin et al. 2005; Almeida et al. 2007; van Eekelen 2015; van Eekelen et al. 2015; Cao et al. 2016; Fagundes et al. 2017; Fonseca and Palmeira 2019). Although many studies have focused on soil arching to better understand the behavior of geosynthetic-reinforced piled embankments, approaches to investigate the influence of mechanical damage to the geosynthetic are still rare and such damages may compromise the performance of this type of solution.

This paper presents a model study on the use of different reinforcement types and the influence of reinforcement damage on the performance of piled embankments on soft soil. Tests were carried out employing a large-scale apparatus in order to simulate conditions close to those in the field in geosynthetic-reinforced piled embankments.

2. EXPERIMENTS

2.1 Apparatus

Large-scale laboratory tests were carried out to examine the behavior of reinforced piled embankments. The apparatus used in the experiments consisted of a rigid tank shown in Figure 1. A square arrangement of caps was simulated in the tank with the reinforcement clamped along its periphery by a rigid steel clamp.

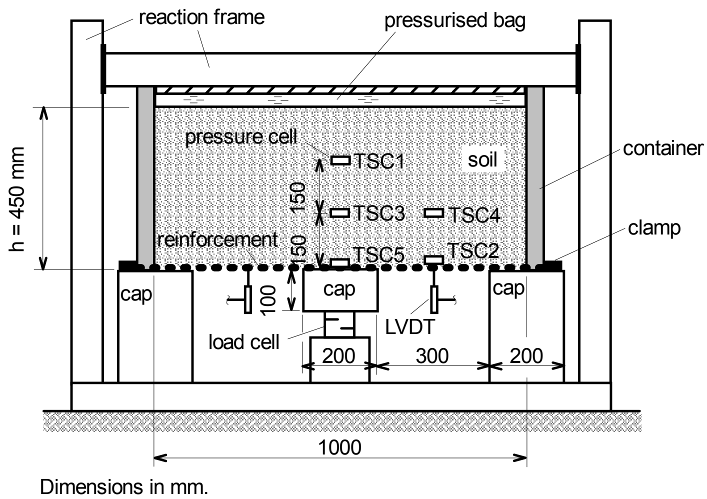

Figure 1. View of the equipment used during one of the tests (Fonseca and Palmeira 2019).

The internal walls of the tank were lubricated with oil and double layers of plastic film to reduce friction along these boundaries. A 450 mm thick gravel fill layer was built and a pressurised rubber bag was employed to apply surcharge pressures. The monotonic vertical stresses of 10 kPa, 25 kPa and 40 kPa were applied to the fill surface, corresponding to equivalent embankment heights of approximately 1 m, 2 m and 3 m under model conditions. Caps 200 mm wide and 500 mm spacing (centre to centre), made of concrete, supported the fill layer in a square arrangement. From the dimensions of the apparatus and typical dimensions of prototype conditions, the scale factor (λ) of the test can be assumed as 5. The similitude laws for the model tests are presented in Table 1. Similar modelling technique was used in other studies on piled supported reinforced fills (Demerdash 1996; Pinto and Cousens 1999; Zaeske 2001; Heitz 2006; Leong 2006; van Eekelen et al. 2012; van Eekelen 2015; Shahu 2016 and Ou Yang et al. 2017).

Table 1. Similitude laws for the model tests.

Parameter	Scale factor
Length	λ
Displacement	λ
Acceleration	1
Stress	λ
Strain	1
Force	λ^3
Tensile strength	λ^2
Tensile stiffness	λ^2

The instrumentation employed in the tests consisted of LVDTs for the measurement of settlements of the base of the fill layer, a load cell for the measurement of the load on the central cap and electric total stress cells distributed in the fill mass were used to measure vertical stresses at distinct points during the experiments. Results obtained by these cells will not be addressed in this paper. Discussions on total stress cell measurements can be found in Fonseca and Palmeira (2019). The displacements of markers fixed to the reinforcement layer combined with a photographic technique provided the measurement of average reinforcement tensile strains between caps with an accuracy of ±1%. A 8 channels data acquisition system (Spider 8), connected to a microcomputer, acquired the readings from the instrumentation during fill construction and after surcharge application on the fill top.

2.2 Materials used in experiments

A fine gravel was used as embankment material. Figure 2 shows the grain size distribution curve of the fill material. The main geotechnical characteristics of the gravel are listed in Table 2. Eighty per cent (in mass) of the gravel grain diameters ranged from 3 mm to 19 mm, with an average diameter (D_{50}) equal to 5.8 mm and a coefficient of uniformity (C_U) of 2.63. The fill layer was lightly compacted by tamping up to a target dry unit weight of 16.1 kN/m^3. Large-scale direct shear experiments (300 mm × 300 mm × 175 mm) were carried out to determine the shear strength of the gravel. An average friction angle of 46° was determined for the material for the range of vertical stresses of the tests.

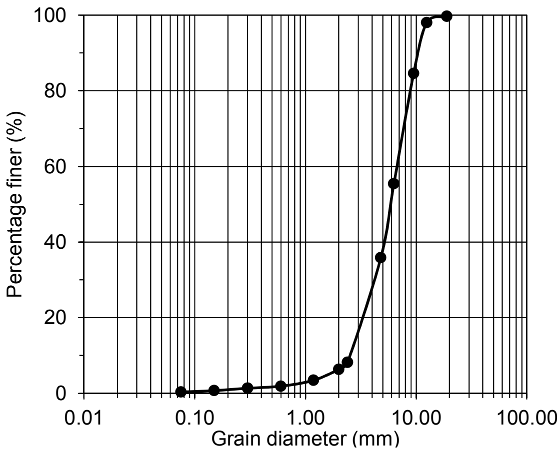

Figure 2. Grain size distribution curve of the fill material.

Three types of "geosynthetics" were used in the tests, and their main characteristics are summarised in Table 3. Figure 3 shows the reinforcements used in the tests. The geogrid (denoted as GGR) is manufactured with polyester fibres (PET) encapsulated by a PVC cover, with 20 mm × 20 mm apertures. GGR was the stiffest one, with secant tensile stiffness values at 5% strain (from wide strip tensile tests as per ASTM D6637) equal to 280 kN/m and 152 kN/m along machine direction (MD) and cross-machine direction (CMD), respectively. The tensile strengths of GGR along machine and cross-machine directions are equal to 17.2 kN/m and 10.5 kN/m, respectively. A very extensible PVC film (tensile stiffness of 0.16 kN/m and maximum strain at failure greater than 500%) was placed between the GGR and the base of the fill layer to avoid the passage of fill grains through the grid apertures during the experiments. The other two "geosynthetics" are geotextile like materials. "Geosynthetics" denoted as GTX-A and GTX-B can be considered as woven geotextiles with tensile stiffness values in machine and cross-machine directions varying between 38 kN/m and 113 kN/m and 29 kN/m and 65 kN/m, respectively. For the scale of the experiments, under prototype conditions these materials would simulate actual geosynthetic layers with tensile stiffness values in the range of 725–7000 kN/m, depending on the geosynthetic element and direction considered. These ranges of values are typical for geosynthetics commonly employed in reinforced piled embankments in the field.

Table 2. Geotechnical characteristics of the fill material.

Characteristic	Value
D_{10} (mm)	2.58
D_{50} (mm)	5.8
D_{60} (mm)	6.77
D_{85} (mm)	9.52
C_U	2.63
Soil grain specific gravity	2.67
Maximum soil unit weight (kN/m^3)	16.6
Minimum soil unit weight (kN/m^3)	15.4
Relative density (%)	61.0
Soil unit weight (kN/m^3)	16.1
Average friction angle (°)	46

Table 3. Reinforcement characteristics.

Characteristic	GGR	GTX-A	GTX-B
Polymer	PET	PET	PET
M_A (g/m^2)	---	75	185
Thickness (mm)	1.03	0.51	0.80
T_{max} (kN/m)*	17.2/10.5	5.6/5.1	10.1/6.0
ε_{max} (%)*	6.4/7.1	71.1/53.7	9.6/13.2
$J_{5\%}$ (kN/m)*	280/152	38/29	113/65
Aperture size (mm x mm)	20x20	NA	NA

Note: Tensile characteristics determined as per ASTM D4595 (for geotextiles) and ASTM D6637 (for geogrid)
*Values on the left and right are values for MD and CMD, respectively

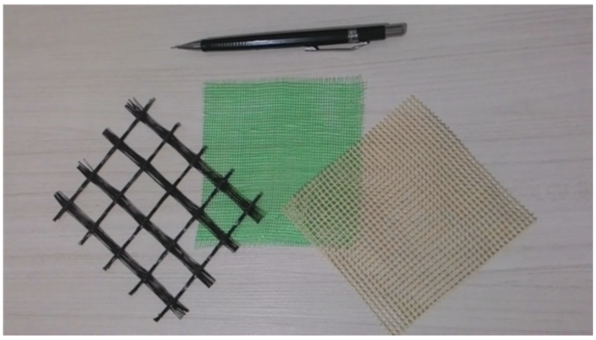

Figure 3. Reinforcement elements used in the tests.

It is worth mentioning that the mass per unit area of the “geotextiles” tested are 75 g/m^2 (GTX-A) and 185 g/m^2 (GTX-B). The low values of mass per unit area were a consequence of scaling, but also allowed the investigation of possible mechanical damages of the geosynthetic elements caused by the corners of the caps, because the reinforcement layers were intentionally installed directly on the pile cap. Wide strip tensile experiments (ASTM 2015, 2017) on the geosynthetics after the tests allowed the evaluation of the effects of mechanical damages.

The repeatability of the test results was assessed by replicating geogrid-reinforced tests. The largest difference between results of replicate tests was less than 8%, depending on the parameter considered. These levels of accuracy can be considered satisfactory for the test conditions.

3. RESULTS OBTAINED

3.1 Vertical displacements between pile caps

Figure 4 shows the variation of maximum vertical displacement (along the geosynthetic cross-machine direction) at the fill base between adjacent pile caps with the normalized equivalent fill height, *H/d*, where *H* is the equivalent fill height and *d* is equal to *s* – *a*, where *s* is the pile spacing and *a* is the cap width. The equivalent fill height is equal to the embankment thickness for thicknesses less than or equal to 0.45 m (Fig. 1) and equal to 0.45 m plus q/γ for embankment thicknesses greater than 0.45 m, where *q* is the surcharge at the embankment surface and γ is the embankment unit weight. The lower the geosynthetic tensile stiffness the greater the maximum fill vertical displacement. As commented earlier, good repeatability of experimental results is demonstrated in Figure 4. In general, the trends and scatters of results were similar to those along the machine direction.

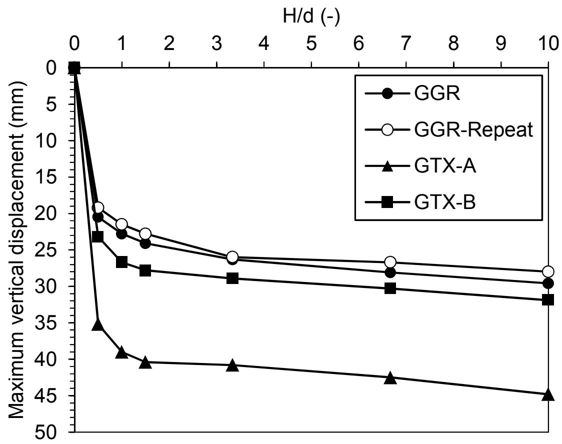

Figure 4. Maximum fill vertical displacement between adjacent pile caps versus normalized equivalent fill height.

The variation of maximum fill vertical displacement (δ) with geosynthetic tensile stiffness (*J*) is represented in Figure 5 with results for MD and CMD for all experiments carried out and all surcharge pressures. A similar trend of variation of vertical displacement with geosynthetic tensile stiffness can be observed. As expected, the lower the tensile stiffness of the geosynthetic, the greater the maximum fill vertical displacement. It can also be observed the lower rate of variation of δ with *J* for values of the latter greater than 100 kN/m (2500 kN/m under prototype conditions). Similar behavior was obtained in numerical analysis of geosynthetic-reinforced piled embankments (Sá 2000 and Sá et al. 2001).

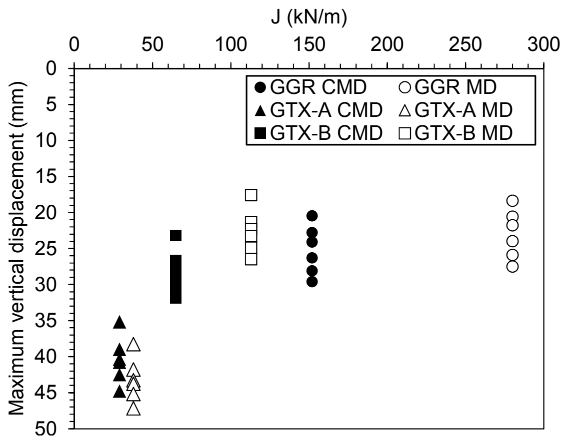

Figure 5. Maximum fill vertical displacement versus geosynthetic tensile stiffness.

The comparison between maximum fill vertical displacements in MD and CMD for each geosynthetic tested is shown in Figure 6. It is interesting to point out that there is linearity between maximum vertical displacements along MD and CMD directions which is quite parallel to the equality line (01:01 slope line). Values of maximum vertical displacements in one direction can be different from those in the other direction depending on differences between tensile stiffness values in each direction. Nevertheless, the results suggest that the variation of displacements in one direction is linearly associated with that in the other direction.

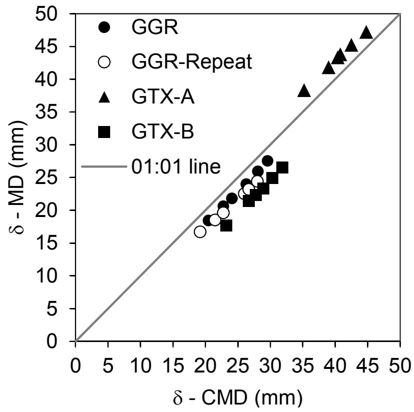

Figure 6. Comparison between maximum fill vertical displacements along machine direction and cross-machine direction.

3.2 Forces on the central cap

The variation of force on the central cap with normalized equivalent embankment height is shown in Figure 7. All geosynthetic elements behaved in a very similar way up to a normalized equivalent fill height of 1.5, which corresponds to the end of construction of the embankment layer (H = 0.45 m). Beyond this value the force on the central cap increases at decreasing rates for the stiffer geosynthetic (GGR), whereas it increases at increasing rates for the lighter reinforcements GTX-A and GTX-B. This trend of variation for the lighter geosynthetics may be associated with mechanical damages at their corners, as will be discussed later in this paper.

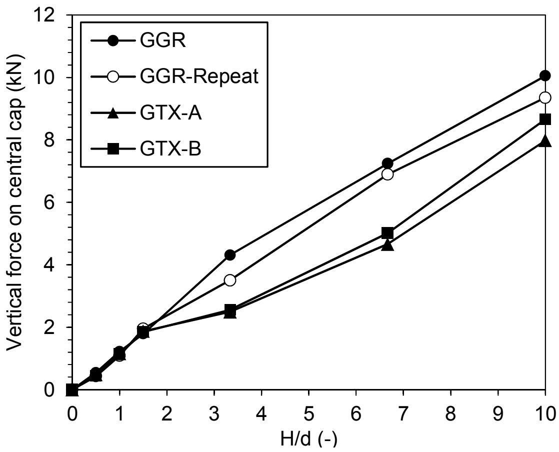

Figure 7. Vertical force on the central cap versus normalized equivalent embankment height.

Figure 8 shows the pile efficacy (E) in absorbing the force from the embankment plus surcharge versus normalized equivalent embankment height. The pile efficacy is defined as the force on the cap divided by the total force (embankment weight plus surcharge) acting on the tributary area of the pile. The results show that efficacy of the pile increased up to close to 100% at the end of fill construction (H/d = 1.5) independent of the geosynthetic type. However, it decreased with the increase of normalized equivalent embankment height. The stiffer the geosynthetic the lower the reduction observed Similar values of pile efficacy were obtained in similar large-scale laboratory tests performed by Leong (2006) and van Eekelen et al. (2012).

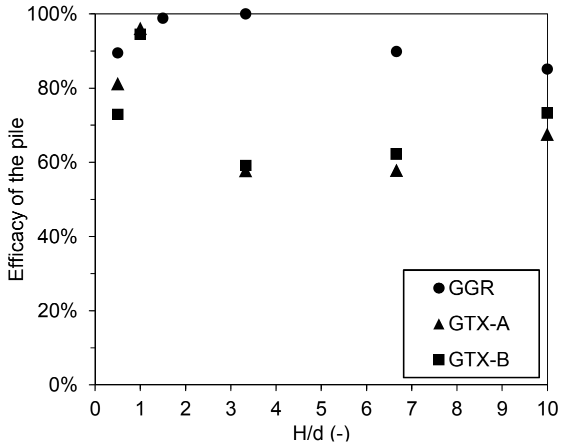

Figure 8. Efficacy of the pile.

3.3 Strains in the geosynthetic element

Strain (ε) in the geosynthetic (along the geosynthetic cross-machine direction) versus normalized equivalent embankment height is represented in Figure 9. Larger strains were obtained for the less stiff geosynthetic element (GTX-A). In the test with GTX-A significant mechanical damages were noticed. Strains in GGR and GTX-B remained constant for normalized equivalent embankment height values greater than 1.5. The tensile strain also kept increasing in the test with the thinner and lighter geotextile GTX-A. In general, the trends and scatters of results were similar to those along the machine direction.

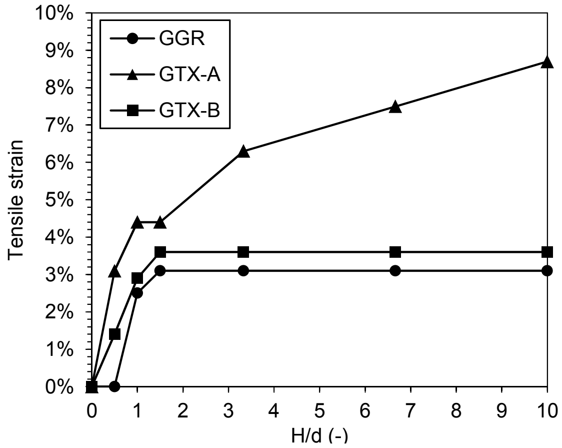

Figure 9. Tensile strains in reinforcement.

3.4 Mechanical damages to reinforcements

Mechanical damage may have compromised to some extent the performance of the geosynthetics GTX-A (M_A = 75 g/m^2) and GTX-B (M_A = 185 g/m^2), as mentioned earlier. Specimens of the geosynthetic elements (after model experiments) were collected to evaluate the level of damage and the extent to which these damages influenced the mechanical characteristics of the reinforcements. To investigate the levels of mechanical damages, wide strip tensile tests as per ASTM D4595 (for geotextiles) and ASTM D6637 (for geogrids) were performed along the geosynthetic machine and cross-machine directions. The reinforcement specimens tested were taken at the corners of the caps, as can be visualized in Figure 10. The greatest damages were visually confirmed in these regions. Since the damages reported took place at the sharp corners of the pile cap, it might also be interesting to compare the performance of different reinforcements in the light of their puncture resistances. However, this type of test would not be applicable to reinforcement GGR. In addition, the relevant overall ultimate failure mechanism of the reinforcement in this type of geosynthetic application is a tensile failure. Nevertheless, puncture resistance tests may be useful for the evaluation of possible reinforcement damages under the conditions of the tests performed.

Figure 10. Specimens of the geosynthetics (GTX-A).

Figure 11 presents the values of reduction factors (RF_T) obtained for the tensile strength along MD and CMD. It can be observed that the values of RF_T were close to 1 for GGR. For the lighter geosynthetic (GTX-A), the values of RF_T were significantly greater than that values for the stiffer reinforcement GGR. GTX-B seems not to have been damaged or the sampling technique may not have captured relevant damages.

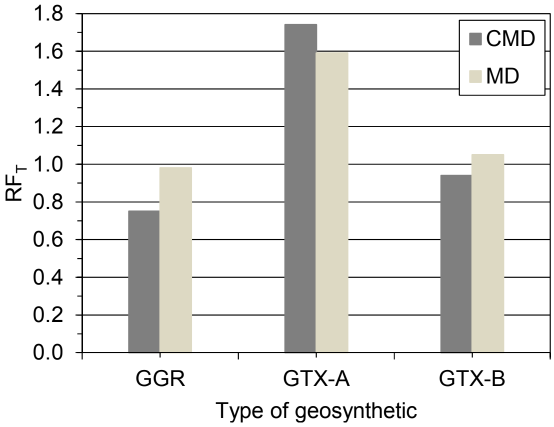

Figure 11. Mechanical damages in light geosynthetics at the corners of the pile caps.

As expected and commented earlier, the damages at higher surface surcharges must have influenced the behavior of the GTX-A (lighter reinforcement). No significant reduction in tensile strength was noted for geosynthetic GTX-B. For GGR, no influence of possible damage was identified by the wide strip tensile tests. The results highlight the importance of avoiding direct contact between the pile cap and the geosynthetic reinforcement, particularly when employing weak geogrids and light geotextiles. Mechanical damages may be intense during fill construction because of repetitive loading caused by the traffic of construction vehicles on low fill thicknesses.

4. CONCLUSIONS

Relevant parameters (pile efficacy, maximum fill settlement and reinforcement tensile strain) associated with the design of geosynthetic-reinforced piled embankments, measured from model tests, were presented in this paper. Large-scale laboratory tests were carried out and the results obtained were used to evaluate the influence of reinforcement damage on the performance of piled embankments on soft soil. The main conclusions obtained are presented below.

The performance of piled embankments is clearly related to the geosynthetic characteristics employed. Test results showed that the magnitude of maximum settlement depends of the reinforcement tensile stiffness. The stiffer the reinforcement, the less fill base settlements and reinforcement strains and the greater the load transferred to the pile cap. Pile efficacy also depends of the reinforcement tensile stiffness, as well as of the ratio between the embankment height and the spacing between caps. Therefore, it is important to note that current analytical design methods still do not take into account some relevant characteristics of the reinforcement.

In the later loading stages in tests on the lighter reinforcement (GTX-A), mechanical damage was observed and it influenced reinforcement performance. No failure of the reinforcement was noted, but considerable damages were observed at the cap corners. For the other lighter reinforcement tested (GTX-B), some overstress could be visually identified at the contacts with the caps. However, this seems not to have influenced the overall performance of this reinforcement. The stiffer geosynthetic suffered no mechanical damage. Therefore, it is always recommended to avoid contact between reinforcement and pile cap in such works.

The results obtained in the experiments carried out in this work show that further investigation is necessary for a better understanding on the behavior of reinforced piled embankments. Despite the study having been based on results from a specific laboratory test arrangement, the results presented are important to better understand how the mechanical damage in the geosynthetic may influence the performance of reinforced piled embankments.

ACKNOWLEDGEMENTS

The authors would like to thank the following institutions, which contributed in different ways to the research activities described in this paper: The University of Brasilia, CNPq-National Council for Scientific and Technological Development, CAPES-Brazilian Ministry of Education, and geosynthetic manufacturers.

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