Acta Scientiarum
http://www.uem.br/acta
ISSN printed: 1806-2563
ISSN on-line: 1807-8664
Doi: 10.4025/actascitechnol.v35i4.16046

Study of the behavior of a reinforced embankment supported on alluvial soft soil

João Alexandre Paschoalin Filho[1*], Geraldo Vanzolini Moretti[2], Carlos Vinícius dos Santos Benjamin[3] and Atílio de Oliveira Moretti[2]

*[1]Programa de Pós-graduação em Gestão Ambiental e Sustentabilidade, Universidade Nove de Julho, Av. Francisco Matarazzo, 612, São Paulo, São Paulo, Brazil. [2]Moretti Engenharia Consultiva, São Paulo, São Paulo, Brazil. [3]Ober Geosynthetics do Brazil, São Paulo, São Paulo, Brazil. *Author for correspondence. E-mail: joao@morettiengenharia.com.br; jalexandre@uninove.br*

ABSTRACT. This paper presented a study on the behavior of an embankment, 5.0 m high, reinforced with geogrids, and constructed over a soft soil 7.0 m thick. In order to determine the design strength (T_d) of the reinforcement, it was carried out a limit equilibrium analysis using the following methods: Simplified Bishop (1955) and Corrected Janbu (1954), for the hypothesis of circular and non circular slip surfaces respectively. In order to verify the behavior of the reinforced embankment, finite element analyses were performed using the software Phase[2]. Therefore, this work presented the determination of the reinforcement load design, verification of the magnitude of reinforcement strains, determination of the plastification zones in the foundation soil due to the elevation of the compacted soil, and values of distortion and horizontal displacement of the soft soil and mechanism of mobilization of reinforcement load.

Keywords: reinforced soil, soft soil foundation, limit equilibrium methods, finite element methods, geogrids.

Estudo do comportamento de um aterro reforçado apoiado sobre solo aluvionar mole

RESUMO. Este trabalho apresenta o estudo do comportamento de um aterro com 5 m de altura reforçado com geogrelhas e executado sobre uma camada de solo mole de 7 m de espessura. No intuito de se determinar a resistência de projeto (T_d) do reforço, foram executadas análises por meio de métodos de equilíbrio limite utilizando-se as seguintes metodologias: Bishop (1955) Simplificado e Janbu (1954) corrigido, considerando a hipótese de ocorrência de superfície de ruptura circular não circular. Com o objetivo de se observar o comportamento do aterro reforçado foram conduzidas análises por meio de elementos finitos utilizando-se o programa Phase[2]. Dessa maneira, este trabalho apresenta a determinação da resistência de projeto, a verificação das tensões no reforço e a determinação das zonas de plastificação no solo de fundação do aterro e os valores de distorção e deslocamentos horizontais do solo mole na mobilização da resistência do reforço.

Palavras-chave: solo reforçado, solo mole de fundação, métodos de equilíbrio limite, metodologia de elementos finitos, geogrelhas.

Introduction

With the expansion of urban areas, increasing the necessity of occupation, sites that were not historically occupied are now being increasingly occupied.

This situation concerns the technical means to find alternative solutions for mitigation of structural problems caused in the buildings and embankments by soil settlements. Geotechnical engineers face several challenges when constructing embankments over softsoils. These include potential bearing failure, intolerable settlement, and global or local instability (MIM; PING, 2008).

When constructing an embankment over soft subsoil of low strength and high compressibility, the engineering tasks are to prevent the failure of the embankment and to control the subsoil deformation. Several methods have been developed for economically and safely constructing embankments on soft subsoil. Placing a layer of reinforcement at the base of the embankment is one of the methods (CHAI et al., 2002).

According to Indraratna et al. (2007) a wide variety of geosynthetics with different properties have been developed to meet highly specific requirements corresponding to a range of different uses. Geosynthetic materials perform five principal functions in civil engineering applications: separation, reinforcement, filtration, drainage and moisture barrier.

Geotextiles can notably improve the construction height, decrease the horizontal displacements and asymmetrical settlement of the soft soil, and increase dissipation speed of pore water pressure.

Geosynthetics are increasingly being used as reinforcement in permanent earth structures constructed in conjunction with transportation facilities, including retaining walls, steep slopes, and bridge abutments. In many cases, the inclusion of geosynthetics in soils allows constructing structures with significantly reduced costs as compared to unreinforced soil structures. In recent years there have been many advances in the understanding of issues related to the use of geosynthetics (ROWE, 2007).

The behaviour and design of geosynthetic-reinforced embankments over soft soil have attracted considerable attention in both practice and the literature. The behavior of basal reinforced embankments over typical soft soils is now well understood: a number of papers have addressed these issues (ROWE; LI, 2005). Issues related to the design and factors affecting the performance of reinforced soil have been addressed by many researches in recent times (AL HATTAMLEH; MUHUNTHAN, 2006; BATHURST et al., 2005; HATAMI; BATHURST, 2005; HUFENUS et al., 2006; PARK; TAN, 2005; SKINNER; ROWE, 2005; YOUWAI; BERGADO, 2004).

The construction of reinforced embankments needs special consideration when the foundation soil exhibits an increase in undrained shear strength of 10% or greater for one order of magnitude increase in strain rate. Under these circumstances, the strain in the reinforcement at a constant fill thickness can significantly increase due to the creep of the rate sensitive foundation soil (LI; ROWE, 2002).

As stated by Leshchinsky and Boedeker (1989), the geosynthetic tensile strength and its interaction properties are two fundamental parameters required for designing reinforced slopes and walls. The reinforcement effects attributed to the geosynthetic tensile strength, can develop only thought its interaction with the soil along common interfaces, that is, due to friction, adhesion, and passive resistance, the geosynthetics sheet is restrained from pullout, this allowing the mobilization of its tensile resistance. Consequently, the behavior of embedded geosynthetics subjected to tensile load is of major importance in reinforcement applications.

The original geogrids were made in the United Kingdom and brought to the United States by way of Canada by the Tenser Corp. Georgics using polyester fibers as the reinforcing component were developed in the United Kingdom around 1980. This led to the development of polyester georgics made on textile weaving machinery. In this process, according to Koerner (1996), many fibers are gathered together to form longitudinal and transverse ribs with large open spaces between. The crossovers are joined by knitting or intertwing before the entire structural unit protected by a separate coating.

The geogrids that result from the aforementioned process are relatively high-strength, high-modulus, low-creep-sensitive polymers with apertures varying from 1 to 10 cm. These holes are either elongated ellipses, near squares with rounded corners, squares or rectangles. Under some circumstances, separation may be a function, but usually it is not. Invariably, geogrids are involved in some form of reinforcement. The following uses have been reported in the literature as reported by Koerner (1996):

a) Reinforcement of embankments fills and earth dams;

b) Repairing slope failures and landslides;

c) As gabions for wall construction;

d) As gabions for erosion control structures;

e) As sheet anchors and facing panels to form an entire retaining wall;

f) As asphalt reinforcement in pavements;

g) As inserts between geotextiles

h) To reinforce landfills to allow for vertical and horizontal expansion.

This paper presents a numerical study using finite elements theory for an embankment with 5.0 m high, constructed over an alluvial clay deposit, very soft, with 7.0 m thickness. This configuration was chosen because this is a geometry commonly found in constructions in metropolitan regions of the sedimentary basin of the city of São Paulo, São Paulo State in Brazil. Figure 1 presents the embankment configuration, and Table 1 presents the geotechnical characteristics of these materials.

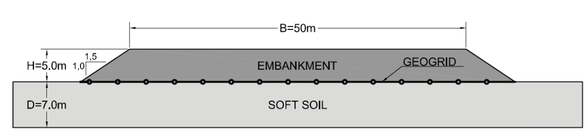

Figure 1. Cross-section of the embankment constructed over a soft soil.

Table 1. Geotechnical characteristics of the materials.

Geotechnical characteristics	Unity	Material	
		Embankment	Alluvium
γ_{nat}	kN m^{-3}	18	14
γ_{sat}	kN m^{-3}	19	15
c	kPa	10	10
φ	°	30	0
E	kPa	30 x 10³	2 x 10³
ν	----	0.30	0.49

where: γnat = Natural volume weight, γsat = saturated volume weight, c = cohesion, φ = internal friction angle, E = Elasticity modulus, ν = Poisson´s ratio.

Material and methods

For the development of this work it was performed the following studies:

a) Determination of the design load of the reinforcement (T_d);

b) Verification of the behavior of the soft soil foundation and geosynthetic reinforcement;

c) Determination of the mobilized loads in the reinforcement;

d) Determination of the horizontal displacements and distortions under the embankment;

e) Determination of the plastification areas resultant from the deformation of the soft soil foundation.

In order to determine the design strength (T_d) of the reinforcement, it was performed the following methods of limit equilibrium: Simplified Bishop (1955) and Corrected Janbu (1954). According to Rowe and Li (2005) limit equilibrium methods have been used extensively to assess the short-term (undrained) stability of reinforced embankments constructed on soft foundation soils.

The stability analysis is usually performed using methods of slices with the support of computer programs. The Simplified Bishop (1955) method is used for areas that present circular potential slip surfaces. However the Corrected Janbu (1954) method is commonly used for slopes that present potential non circular slip surfaces.

The value of T_d adopted was the one who led the embankment to a global factor of safety equal to 1.3, the minimum value recommended in terms of slope stability, according to the NBR 11682 (ABNT, 2009). The stability analysis was processed using the software Slide 5.0 developed by Rocscience geomechanical software, commonly used for geotechnical projects. Slide 5.0 (ROCSCIENCE..., 2012b) is complete with finite element groundwater seepage analysis, rapid drawdown, sensitivity and probabilistic analysis and support design. All types of soil and rock slopes, embankments, earth dams and retaining walls can be analyzed.

To verify the behavior of this construction, numerical simulations were run using the software Phase^2, also from Rocscience... (2012a). The software Phase^2 is a 2D elasto-plastic finite element stress analysis program for underground or surface excavations in rock or soil. It can be used for a wide range of engineering projects and includes support design, finite element slope stability, groundwater seepage and probabilistic analysis. The finite elements analysis consists of a computer model of a material or design that is stressed and analyzed for specific results. It is used for new product design, and existing product refinement.

Methods of analysis such as limit equilibrium and plasticity solutions provide no information about deformations or strains, which develop in the reinforcement for a given reinforced embankment. Reinforced embankments are a composite system consisting of three components: the foundation soil, the reinforcement, and the embankment fill. Their performance is highly dependent on deformations and on the interaction between these components (ROWE; LI, 2005).

Results and discussion

The Table 2 presents the safety factors obtained by the back-analysis using limit equilibrium.

Table 2. Obtained safety factors.

Methodology	Safety Factor
Simplified Bishop (1955)	1.3
Corrected Janbu (1954)	1.6

The Table 2 shows that the Simplified Bishop (1955) method, that considers a circular slip surface geometry, presented a lower safety factor when compared to the Corrected Janbu (1954) method. The ultimate tensile strength (T_d) of the reinforcement was obtained by a back-analysis of the studied embankment, necessary to reach a safety factor of 1.3 (minimum value established by the NBR 11682 (ABNT, 2009) for the studied conditions). It was used in back-analysis the Simplified Bishop (1955) method and the software Slide 5.0. The obtained value of T_d was equal to 700 Kn m^{-1}.

From the determined value of T_d, it was obtained the reinforcement modulus of stiffness (J), considering deformations of work for the reinforcement's ultimate tensile strengths of 5% for geogrids made with PVA (polyvinyl alcohol) and 12% for a geogrid made with high strength polyester (PET). The considered values of ε_a are frequently used by the technical means in geotechnical designs of soil reinforcements. The modulus of stiffness was obtained by the following equation:

$$J=T_d/\varepsilon_a \qquad (1)$$

which:

Td = ultimate tensile strength;

J = modulus of stiffness;

εa = deformation of work for reinforcement ultimate tensile strength.

Table 3 presents the characteristics of the considered reinforcements and Figure 2 shows the graphic T_d versus strain.

Table 3. Reinforcement characteristics.

ε_d (%)	T_d (kN m^{-1})	Stiffness modulus (J)
5	700	14000 kN m^{-1}
12		5383 kN m^{-1}

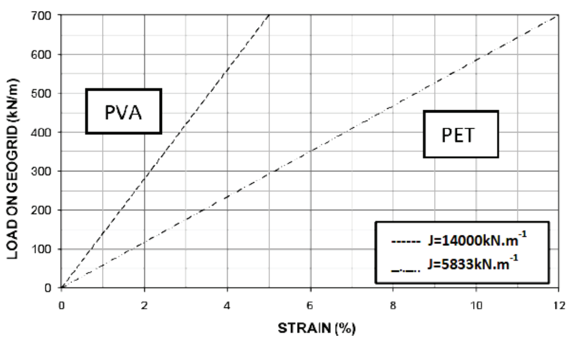

Figure 2. Load on the geogrid T_d *versus* strain.

The Figures 3 and 4 were obtained by the Phase[2] analysis and illustrate the study on the behavior of the reinforced foundation in order to support the loading of the studied embankment. The analysis was processed using staged construction, with the embankment compacted in layers of 0.5 m.

Figures 3 and 4 evidenced that the reinforcements with stiffness equal to J = 5833 kN m^{-1} and J = 14000 kN m^{-1} presented a visible change in behavior for embankments with 3.0 m height. This inflection noticed in Figures 5 and 6 represents the moment when the soft soil foundation is starting to show areas of significant plastification, starting the failure process.

Thus, it can be assumed that the height of 3.0 m corresponds to the maximum height of the embankment before the plastification of the soft soil foundation, regardless the modulus of stiffness of the reinforcement. Figure 4 showed that, up to a height of 3.0 m, the distortion values obtained were very close, i.e., ranging between 3 and 4%.

However, from the height of 3.0 m, the soft clay foundation presented different behaviors for the conditions studied. For the embankment without reinforcement, it is evidenced a failure after a height of 3.0 m. It seems that in the embankment reinforced with the PET geogrid, the foundation presented distortion values of approximately 3 times greater for the embankment 5.0 m high.

The Figures 5 and 6 present the development of the plastification zones under the reinforcement embankment. It can be noticed, under the embankment slope, the occurrence of a slip surface originating from the plastification process of soft soil foundation, when the embankment reached the height of 5.0 m.

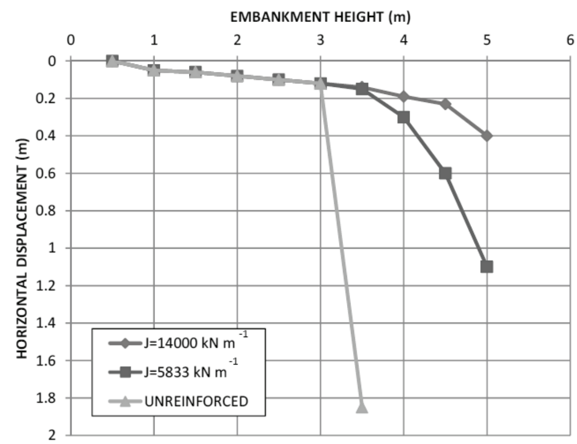

Figure 3. Horizontal displacement *versus* embankment height.

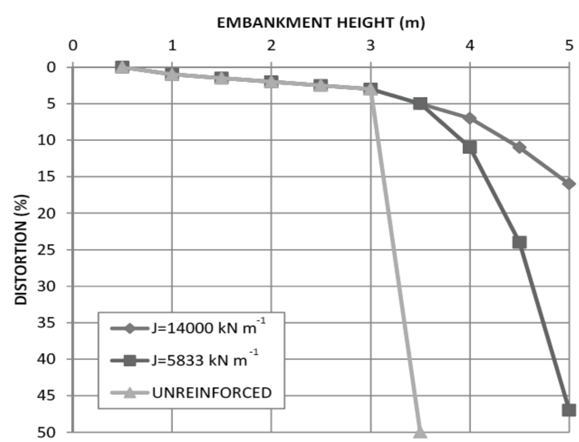

Figure 4. Distortion *versus* embankment height.

Figures 7 and 8 present the percentage of mobilized load and deformation on the reinforcement.

Figure 7 demonstrated that the percentage of load to be mobilized in the reinforcement to the height of 3.0 m was slightly lower than 10%. The reinforcement with the higher modulus of stiffness, until the embankment height of 3.0 m, presented the highest percentage of mobilized load. However, at this point, the difference between the modules of stiffness had small influence on the percentage of mobilized load, and, for the height of 5.0 m, both reinforcements had almost the same percentage of mobilized load.

As observed in Figure 8, for a height of 3.0 m, both reinforcements obtained deformations smaller than 0.5%. Considering the height of 5.0 m, the geogrid with lower modulus of stiffness presented the highest deformation value.

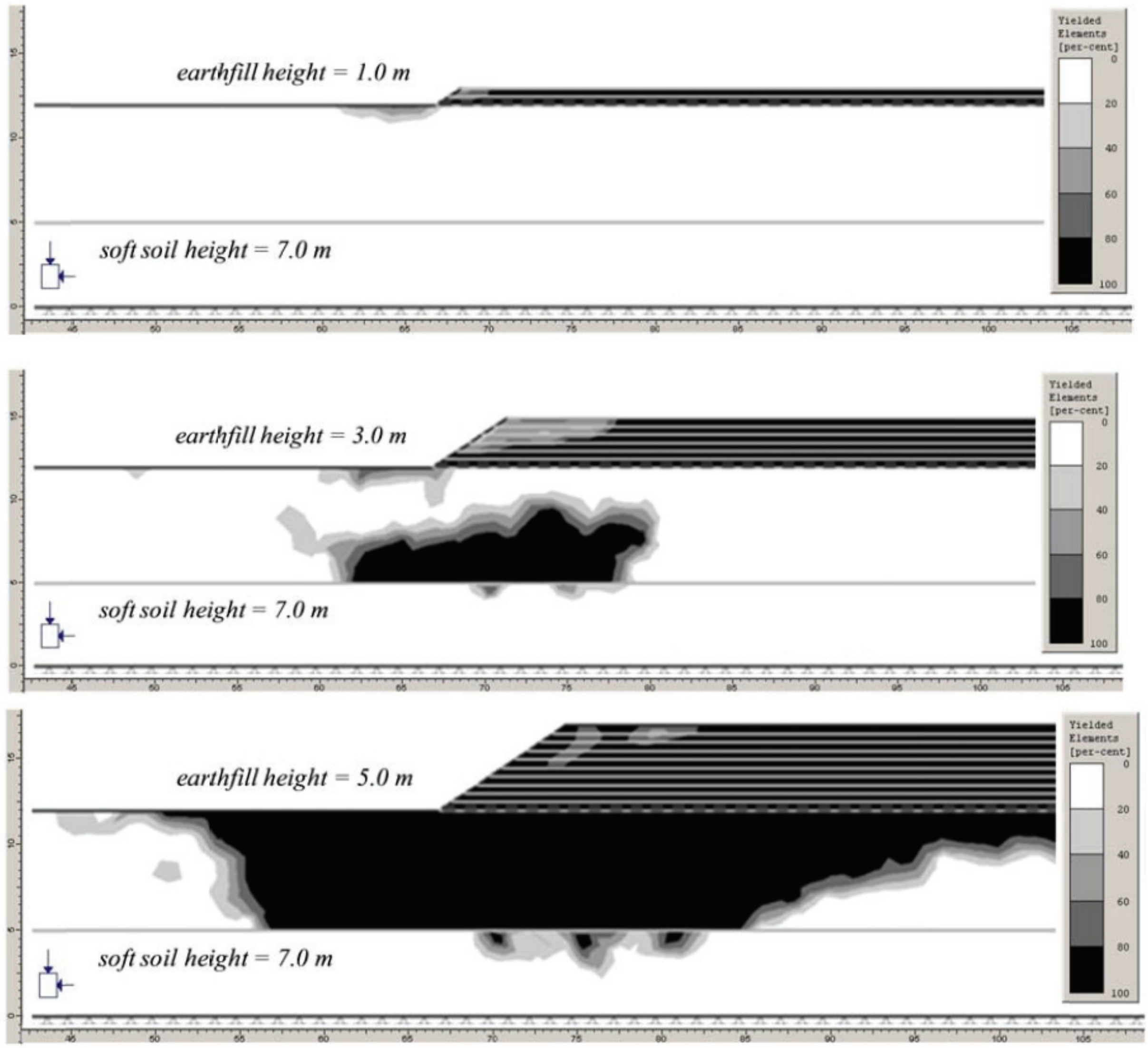

Figure 5. Geogrid with J = 5833 kN m^{-1}.

Analyzing Figures 7 and 8, it can be seen that for an embankment height smaller than 3.0 m, the percentage of mobilized load in the geogrid was independent of the magnitude of deformation and modulus of stiffness. Table 4 presents the obtained parameters.

The reinforcement strains were calculated regarding the modulus of stiffness equal to J = 5833 kN m^{-1} for a maximum strain, for a PET geogrid, (ε_a = 12%) and, J = 14000 kN m^{-1} for a maximum strain, for a PVA geogrid, (ε_a = 5%) with embankment heights of 3.0 m and 5.0 m.

For the embankment heights of 3.0 m and 5.0 m, the reinforcement deformations on reinforcements were equal to 0.45 and 1.8%, respectively. (J = 14000 kN m^{-1}). For the embankment heights of 5.0 m and 3.0 m, the deformations on reinforcements were respectively 4.7 and 0.5% (J = 5833 kN m^{-1}). The Table 5 presents the values of $T_{d,max}$ (for H = 5.0 m) and $T_{d,limit}$ (for H = 3.0 m) considering the deformations presented in Table 4, these values were calculated using the equation (1).

The mobilized loads regarding the embankment height of 5.0 m, for J = 5833 kN m^{-1} and J = 14000 kN m^{-1}, were respectively, 274 kN m^{-1} and 252 kN m^{-1}, approximately an average value of 38% of the value of T_d estimated. For the embankment height of 3.0 m, the mobilized loads were 29.1 kN m^{-1} and 56 kN m^{-1}, i.e., approximately an average value of 6% of the T_d estimated.

Using the values of $T_{d,\,max}$, and $T_{d,\,limit}$, presented in Table 5, it was carried out further analysis of stability using the methods of Simplified Bishop and Corrected Janbu. The Table 6 presents the factors of safety obtained.

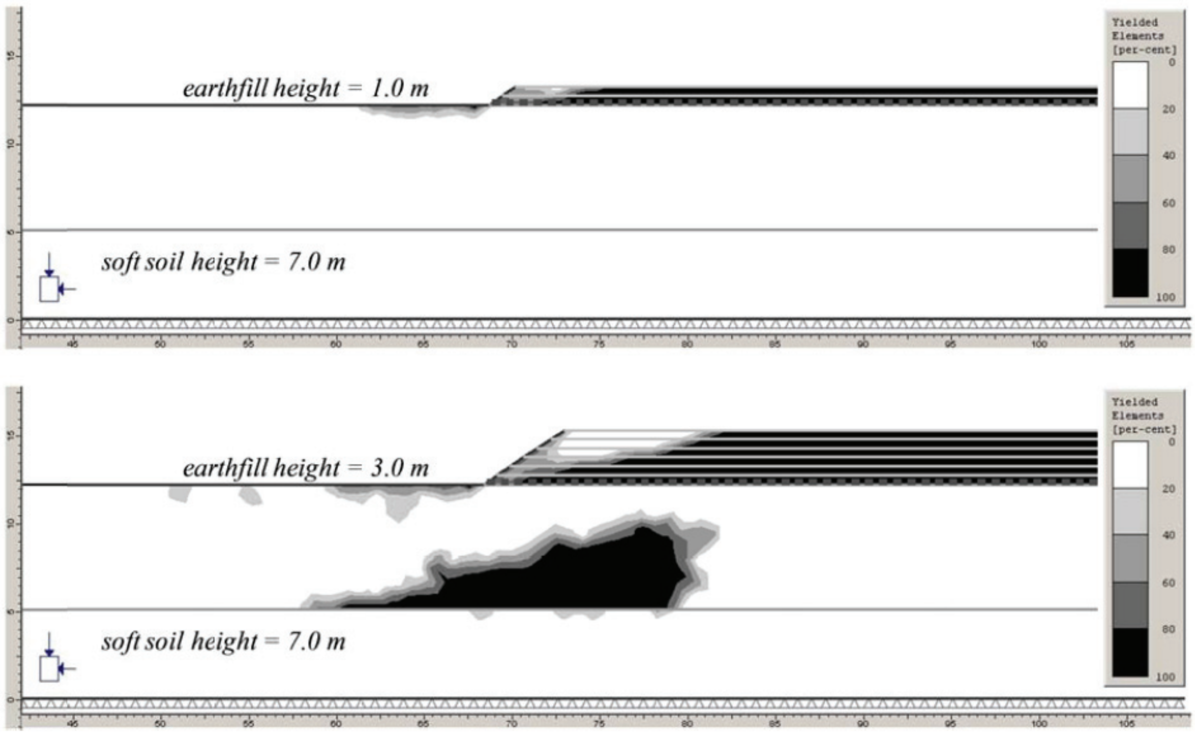

Figure 6. Geogrid with J = 14000 kN m^{-1}.

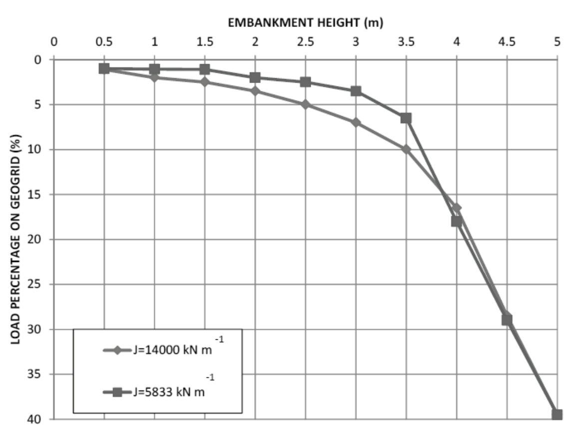

Figure 7. Geogrid load percentage *versus* embankment height.

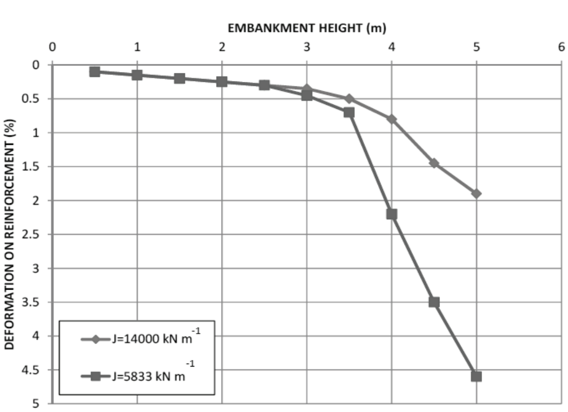

Figure 8. Deformation on reinforcement *versus* embankment height.

Table 4. Parameters of the analyses.

J (kN m^{-1})	H_{max} (m)	H_{limit} (m)	ε_{max} (%)	ε_{limit} (%)
5833	5.0	3.0	4.7	0.5
14000	5.0	3.0	1.8	0.45

In which: H_{limit} = limit height of the embankment before the foundation soil plastification (3.0 m); ε_{max}= maximum deformation of the reinforcement for the height of 5.0 m; ε_{limit} = reinforcement strain for the height before the plastification; J = stiffness modulus.

Table 5. Obtained parameters.

J (kN m^{-1})	T_d (kN m^{-1})	$T_{d, limit}$ (kN m^{-1})	$T_{d, max}$ (kN m^{-1})	$T_{d,max}$/Td	$T_{d,limit}$/Td
5833	700	29.1	274	0.39	0.04
14000	700	56	252	0.36	0.08

Which: H_{limit} = limit height of the embankment before the foundation soil plastification (3.0 m); T_d = project load of the reinforcement; $Td_{, max}$ = maximum load transferred to the reinforcement for the embankment height equal to 5.0 m; $Td_{, limit}$ = transferred load to the reinforcement for the embankment height equal to 3.0 m.

Table 6. Obtained safety factors.

Design Method	J (kN m^{-1})	Embankment height (m)	T_d (kN m^{-1})	FS
Simplified Bishop (1955)	5833	5.0	274	1.09
	14000	5.0	252	1.08
	5833	3.0	29.1	0.92
	14000	3.0	56	0.91
Corrected Janbu (1954)	5833	5.0	274	1.02
	14000	5.0	252	1.09
	5833	3.0	29.1	1.06
	14000	3.0	56	1.07

The Table 6 showed that for any method of stability, the use of values $T_{d,\ max}$ and $T_{d,\ limit}$ have conducted to safety factors close to one (condition of limit equilibrium), which proves the consistency of the calculated values of mobilized loads ($T_{d,\ max}$ and $T_{d,\ limit}$) and the reinforcement strains (ε_{max} and ε_{limit}).

The safety factors shown in Table 6 also confirm the use of limit equilibrium methods for calculating the reinforcement embankment, since the analysis used ultimate tensile strength values equivalent to the maximum mobilized loads on geogrid for $T_{d,\ max}$ H = 5.0 m and t_{limit} for H = 3.0 m.

Conclusion

The following conclusions can be taken from the present research:

The procedure used to impose the ultimate tensile strength in the reinforcements for further stability analysis presented implications against the safety of the structure, since it was not taken into account the effects of plastification and lateral extrusion of the foundation soil. However, since it is used the real ultimate tensile strength of the reinforcement, this methodology led to satisfactory results.

After the beginning of the plastification of the soil foundation, the mobilization of the loads on the reinforcements was independent of the modulus of stiffness.

The deformation values obtained for the studied embankment conditions (ε_{max} and ε_{limit}) were lower than the deformations commonly used by the technical means on soil reinforcement projects (PET=12% and PVA=5%) regardless of the studied stiffness modulus and geogrid material considered.

Acknowledgements

The authors would like to thank to Moretti Engineering Consulting Ltda. and University Nove de Julho (Uninove).

References

ABNT-Associação Brasileira de Normas Técnicas. **NBR-11682**: Estabilidade de taludes. Rio de Janeiro: ABNT, 2009.

AL HATTAMLEH, O.; MUHUNTHAN, B. Numerical procedures for deformation calculations in the reinforced soil walls. **Geotextiles and Geomembranes**, v. 24, n. 1, p. 52-57, 2006.

BATHURST, R. J.; ALLEN, T. M.; WALTERS, D. L. Reinforcement loads in geosynthetic walls and the case for a new working stress design method. **Geotextiles and Geomembranes**, v. 23, n. 4, p. 287-232, 2005.

BISHOP, A. W. The use of the slip circle in the stability analysis of slopes. **Géotéchnique Journal**, v. 5, n. 2, p. 7-17, 1955.

CHAI, J. C.; MIURA, N.; SHEN, S. L. Performance of embankments with and without reinforcement on soft subsoil. **Canadian Geotechnical Journal**, v. 39, n.4, p. 838-848, 2002.

HATAMI, K.; BATHURST, R. J. Development and verification of a numerical model for the analysis of geosynthetic-reinforced soil segmental walls under working stress conditions. **Canadian Geotechnical Journal**, v. 42, n.4, p. 1066-1085, 2005.

HUFENUS, R.; RUEEGGER, R.; BANJAC, R.; MAYOR, P.; SPRINGMAN, S. M.; BRONNIMANN, R. Full-scale field tests on geosynthetic reinforced unpaved roads on soft subgrade. **Geotextiles and Geomembranes**, v. 24, n. 1, p. 21-37, 2006.

INDRARATNA, B.; SAHIN, M. A.; SALIM, W. Stabilization of granular media and formation soil using geosynthetics with special reference to railway engineering. **Journal of Ground Improvement**, v. 11, n. 1, p. 27-44, 2007.

JANBU, N. Application of composite slip surfaces for stability analysis. **Proceedings of the European Conference on Stability of Earth Slopes**, v. 3, p. 43-49, 1954.

KOERNER, R. M. **Designing with Geosynthetics**. New Jersey: Prentice Hall, 1996.

LESHCHINSKY, D.; BOEDEKER, R. H. Designing geosynthetic reinforced earth structures. **Journal of Geotechnical Engineering**, v. 115, n. 10, p. 1459-1478, 1989.

LI, A. L.; ROWE, R. K. Some design considerations for embankments on rate sensitive soils. **Journal of Geotechnical and Geoenvironmental Engineering**, v. 128, n. 11, p. 885-897, 2002.

MIM, Y. C.; PING, C. W. An experimental investigation of soil arching within basal reinforced and unreinforced piled embankments. **Geotextiles and Geomembranes**, v. 26, n. 2, p. 164-174, 2008.

PARK, T.; TAN, S. A. Enhanced performance of reinforced soil walls by the inclusion of short fiber. **Geotextiles and Geomembranes**, v. 23, n. 4, p. 348-361, 2005.

ROCSCIENCE GEOMECHANICS SOFTWARE AND RESEARCH. **Software Phase^2** - User's guide. Toronto: Rocscience Inc., 2012a.

ROCSCIENCE GEOMECHANICS SOFTWARE AND RESEARCH. **Software Slide 5.0** - User´s guide. Toronto: Rocscience Inc., 2012b.

ROWE, R. K. Advances and remaining challenges for geosynthetics in geoenvironmental engineering applications. **Soil and Rocks**, v. 30, n. 1, p. 3-30, 2007.

ROWE, R. K.; LI, A. L. G. Geosynthetic-reinforced embankments over soft foundations. **Geosynthetics International**, v. 12, n. 1, p. 50-85, 2005.

SKINNER, G. D.; ROWE, R. K. A novel approach to estimating the bearing capacity stability of geosynthetic reinforced retaining walls constructed on yielding foundations. **Canadian Geotechnical Journal**, v. 42, n. 3, p. 763-779, 2005.

YOUWAI, S.; BERGADO D. T. Numerical analysis of reinforced wall using rubber tire chipssand mixtures as backfill material. **Computers and Geotechnics**, v. 31, n. 2, p. 103-114, 2004.

Received on February 14, 2012.
Accepted on July 16, 2012.