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Effect of Geosynthetic Reinforcement Inclusion on the Strength Parameters and Bearing Ratio of a Fine Soil

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Abstract
This paper reports an investigation on the beneficial effects of reinforcing a fine soil with a geosynthetic (reinforcement geocomposite) and their behaviour under loading. The effectiveness of the reinforcement was investigated through triaxial and California Bearing Ratio, CBR, tests. The triaxial tests showed that including the reinforcement provided additional confinement to the reinforced soil samples, causing an increase in the corresponding strength parameters. However, the reinforcement decreased the secant stiffness modulus of the composite material, particularly for low strains. The CBR tests were performed on soaked samples, compacted for different initial water content values. The influence of increasing the number of reinforcement layers was also analysed. The results showed that the reinforced samples had a maximum bearing capacity larger than the unreinforced material. The reinforcing mechanisms observed in the CBR tests were membrane tension support and bearing capacity increase. Increasing the number of reinforcement layers induced an improved response of the soil-geosynthetic composite material, particularly for a water content lower than the optimum. An increase in the initial water content induced reductions of the bearing capacity of the soil, with different values, depending on position of the initial value relative to the optimum water content.
Keywords: Fine soil, geosynthetic, reinforcement, triaxial, CBR, water content

1 Introduction

Traditionally reinforced soil structures are built using good quality granular fill materials. However, these are not always available locally. Nevertheless, in some cases, local (marginal) soils can be used as backfill materials without compromising stability or serviceability. Geosynthetics have been used widely as reinforcements for a wide range of structures (roads, slopes, retaining walls and embankments). Abu-Farsakh et al. (2015) summarise a series of studies on unpaved roads where geosynthetics have been used to extend the service life of pavements, reduce base course thickness for a given service life and delay rutting development. Geosynthetics can also be used to reinforce weak subgrade layers, or they can be placed at the base-subgrade interface or within the base layer.

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There are several studies in the literature where the response of reinforced soil with geosynthetics is analysed using triaxial tests for granular soil (Chen et al., 2014, Nair and Latha, 2014, Nguyen et al., 2013) or fine soils (Noorzad and Mirmoradi, 2010). Although the California Bearing ratio (CBR) test is only valid for uniform materials, performing CBR tests of reinforced soil can demonstrate the qualitative benefit of adding reinforcement under the same test conditions (Kamel et al., 2004). Similar approaches have been used to assess the influence on the bearing ratio of reinforced soil of parameters such as plasticity index and gradation of soils (e.g., Adams et al., 2016). Moayed et al. (2013) studied the bearing ratio of a two-layered soil (granular soil as base layer; cohesive soil as subgrade layer) for three conditions (unreinforced, with geotextile and with geogrid at the interface between the two soils).

The data presented herein is part of a wider research project focused on designing new solutions for building and rehabilitating existing structures using local fine soils reinforced with geosynthetics. The structures are small dykes, used as boundaries of salt pans and the canals in a tidal lagoon. Using the local fine soil has the additional advantage of providing adequate low permeability to the structures, while the reinforcements improve the mechanical response.

2 Test Program

The effectiveness of reinforcing a fine soil with a geosynthetic was studied by performing triaxial and CBR tests. The materials used (geosynthetic and soil) were characterised in laboratory. The results presented in this paper are part of a wider research project in which several geosynthetics (with different structures) and different soils (granular and fine) were used.

2.1 Materials

The geosynthetic studied was a reinforcement geocomposite (GC) consisting of continuous filament non-woven, reinforced by high tenacity polyester yarns material. Table 1 summarises some characteristics of GC, with indication of the corresponding test methods: tensile strength (T_{max}); strain for maximum load (ε_{max}); thickness for different normal pressures, 2kPa (t_{2kPa}), 20kPa (t_{20kPa}) and 200kPa (t_{200kPa}); mass per unit area (μ). Figure 1 summarises the load-strain curves of GC in both machine and cross-machine direction (MD and CMD, respectively).

<table>
<tr><td rowspan="2">Direction</td><td>T_{max}
kN/m</td><td>ε_{max}
%</td><td>t_{2kPa}
mm</td><td>t_{20kPa}
mm</td><td>t_{200kPa}
mm</td><td>μ
g/m^2</td></tr>
<tr><td>EN ISO
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9864</td></tr>
<tr><td>MD</td><td>54.6</td><td>10.6</td><td rowspan="2">2.14</td><td rowspan="2">1.59</td><td rowspan="2">1.07</td><td rowspan="2">325</td></tr>
<tr><td>CMD</td><td>15.6</td><td>79.9</td></tr>
</table>

Table 1: Properties of geosynthetic GC

The soil, collected from a wall of the salt pans in Aveiro lagoon (Portugal), was characterised in laboratory and classified using USCS, Unified soil classification system (ASTM D2487–11), and AASHTO classification system (AASHTO M 145-91-UL) as ML, sandy silt, or A-4, respectively. Table 2 includes: percentage of fine particles (<0.074 mm); 10% (D_{10}), average (D_{50}) and maximum (D_{max}) grain sizes; liquid limit (w_L); plastic limit (w_P); plasticity index (I_P); unit weight (γ); classification of the soil samples; and compaction characteristics of the soil (ASTM D1557-12, modified Proctor tests), maximum dry density (ρ_{dmax}) and optimum water content (w_{opt}). Figure 2 illustrates the particle size distribution of the soil.

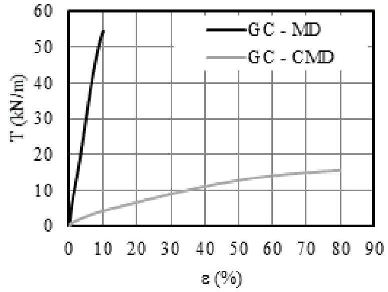

Figure 1: Average load-strain curves for GC in both machine (MD) and cross-machine direction (CMD)

%< 0.074 mm	D_{10}	D_{50}	D_{max}	w_L	w_P	I_P	γ	Soil classification		ρ_{dmax}	w_{opt}
	(mm)	(mm)	(mm)	(%)	(%)	(%)	(kN/m^3)	USCS*	AASHTO+	g/cm^3	%
65.7	0.0001	0.0112	12.7	35	25	10.4	18.3	ML	A-4	1.845	13.9

*USCS - Unified soil classification system (ASTM D2487–11)

+AASHTO classification system (AASHTO M 145-91-UL)

Table 2: Properties and classification of the fine soil

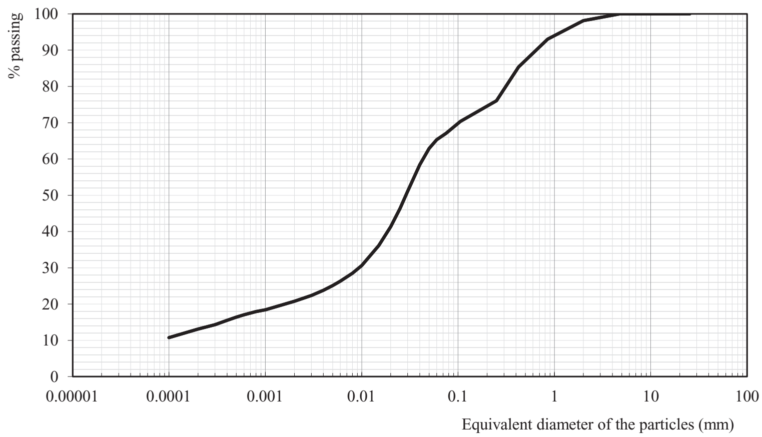

Figure 2: Particle size distribution of the fine soil

2.2 Characterisation Tests

The response of the reinforced fine soil was compared to that of the unreinforced soil tested under the same conditions, using triaxial and CBR tests. Table 3 summarises the tests reported in this paper.

The soil strength was assessed using unconsolidated undrained (UU) compression triaxial tests (ISO/TS 17892-8: 2004): 0.7 mm/min axial strain rate; cylindrical specimens (70 mm diameter, D; 140 mm height, H); relative density, G_C, 77% (corresponding to a dry density ρ=1.42 g/cm^3). All triaxial specimens were prepared similarly and the reinforced soil specimens had a layer of reinforcement (disc with 70 mm diameter) placed at mid height (H/2=70 mm) of the specimen. The triaxial tests reported were obtained for dry specimens tested for confining stresses of 50, 100 and 150 kPa.

The CBR tests were performed according to the procedures described in LNEC E198 (1967), a specification by the *Laboratório Nacional de Engenharia Civil* (LNEC). This is similar to the procedure described in ASTM D1883–07. These differ mainly in the velocity of the test and the number of blows for the compaction procedure. The CBR specimens were cylindrical (152 mm diameter, D, and 125 mm height, H). The soil was prepared to the desired water content (w), and then allowed to rest for 24 hours, closed in plastic containers, in a standard atmosphere (temperature 20°; relative humidity 65%). The specimens were prepared in 5 layers, each 25mm high and compacted with 25 blows (hammer 4.54 kg; drop height 457 mm). The reinforced soil specimens were prepared in a similar way and included layer(s) of GC at different positions (1 layer, at 2/5H from the top of the specimen; 2 layers, at 2H/5 and 3H/5 from the top of the specimen). All specimens were soaked during 96 hours and tested with an imposed axial displacement of 1mm/min. The specimens were tested for three values of the initial water content (11.9%; 13.9% (optimum value); 15%). The unreinforced soil response was compared with that of reinforced soil samples using one or two layers of geocomposite GC.

Test	N. of specimens	Conditions	H (mm)	D (mm)	Reinforcement		w (%)
					N. layers	Position#	
Triaxial	3	UU	140	70	0	-	0
	3	UU	140	70	1	H/2	0
CBR	3	Soaked	125	152	0	-	11.9, 13.9, 15.0
	3	Soaked	125	152	1	2H/5	11.9, 13.9, 15.0
	1	Soaked	125	152	2	2H/5; 3H/5	11.9

From the top of the specimen

Table 3: Summary of the test program

3 Results and Discussion

3.1 Triaxial Tests

The results of the triaxial tests are summarised in Table 4: deviator stress, q; strain, ε; strength parameters, c' and ϕ'; secant stiffness modulus, E. The reinforced soil specimens did not fail, therefore, the corresponding maximum (peak) values could not be determined. In some cases the critical state was not reached; thus, the values at the end of the tests are designated as final (subscript fin).

Although the reinforced soil specimens did not reach failure, the results clearly show the influence of the reinforcement on the mechanical strength of the soil. The final deviator stress, q_{fin}, measured for similar test conditions, was much higher when a layer of reinforcement was included in the soil: increasing 29% to 34%, for comparable test conditions.

Sample	σ_n (kPa)	q_{max} (kPa)	q_{fin} (kPa)	ε_{qmax} (%)	$\varepsilon_{q\,fin}$ (%)	ϕ'_{peak}; ϕ'_{fin} (°)	c'_{peak}; c'_{fin} (kPa)	E_{50}# (MPa)	$E_{\varepsilon=5\%}$ (MPa)
Soil	50	265.1	228.0	8.1	18.0	37.6; 37.4	27.2; 18.0	11.05	4.89
	100	446.3	390.1	9.8	17.7			12.47	7.63
	150	573.5	536.8	12.6	17.5			11.16	8.60
Soil + GC	50	-	304.9	-	18.1	-; 41.2	-; 24.7	7.33	4.64
	100	-	509.7	-	18.1			8.97	7.12
	150	-	690.1	-	18.0			10.20	8.84

When the specimens did not fail, E_{50} was calculated for 50% of q_{fin}

Table 4: Summary of the triaxial tests results

Soil fail when the shear stress exceeds the corresponding strength of the soil, in a given plane. However, when the same soil is reinforced, the contribution of the reinforcement can be interpreted as an additional confining stress (Sieira, 2003; Ruiken and Ziegler, 2008; Ruiken et al., 2010). According to Ruiken et al. (2010), if the vertical spacing between adjacent layers of reinforcement is small enough, the effect of the reinforcement is similar to an additional confining stress acting along the height of the specimen. As a consequence, the strength parameters of the reinforced soil composite material increase relatively to those of the soil. As the reinforced soil specimens did not fail, the corresponding strength parameters in Table 4 are low estimates of their true values. The reinforcement moved the failure from the centre of the specimens, as illustrated in Figure 3. The reinforced soil specimens exhibited bulging between the layer of geosynthetic and the top and base of the specimen.

Table 4 and Figure 4 indicate that the reinforced soil specimens were less stiff than the corresponding unreinforced specimens, particularly for low strains. There are two main reasons for this response. On the one hand, geosynthetics are compressible materials and their thickness reduces with increasing normal stress: t_{2kPa}=2.14 mm, t_{20kPa}=1.59 mm and t_{200kPa}=1.07 mm (Table 1). On the other hand, the reinforcement is mobilised when deformations occur. All specimens (soil and soil + GC) exhibited initial contraction (reduction of volume), followed by expansion (for larger strains). Again, the compressibility of the reinforcement (more important for higher confining stresses) may explain the largest reductions of volume observed for the reinforced specimens.

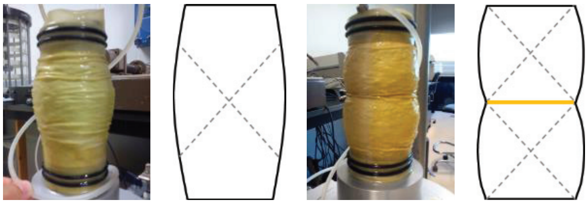

Figure 3: Types of failure observed and typical failure surfaces observed in the triaxial tests: a) and b) unreinforced soil; c) and d) soil reinforced with one layer of geosynthetic

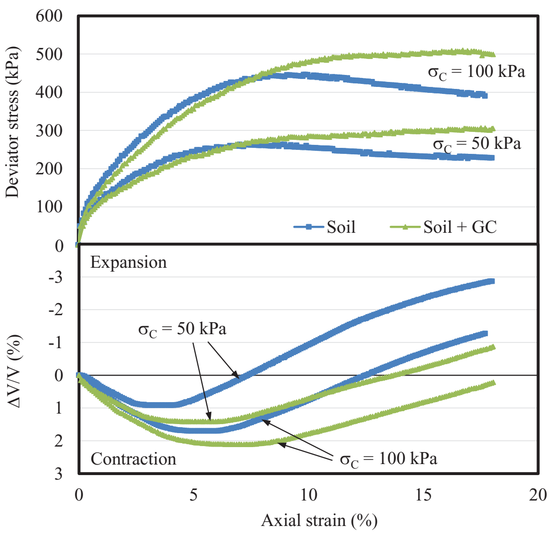

Figure 4: Deviator stress and volumetric strain *versus* axial strain curves from triaxial tests – unreinforced and reinforced soil: confining pressure of 50kPa and 100kPa

According to Saez (1997), two different mechanisms can develop during the triaxial testing of reinforced soil: 1) mobilisation of tensile forces in the reinforcement; 2) occurrence of relative displacements between the soil and the reinforcement. In Mechanism 1 the reinforcement provides strength to the composite material, but its benefit is limited by the tensile strength of the reinforcement. In Mechanism 2 the geosynthetics slides relative to the adjacent soil; the beneficial effect is limited by the direct shear strength between the soil and the reinforcement. The results did not allow distinguishing the two mechanisms mentioned before. However, when the specimens were dismantled, no obvious tensile failure of the reinforced was observed.

3.2 CBR Tests

The results of the CBR tests are summarised in Table 5: desired (w) and real or measured (w_{real}) water content; CBR values for penetrations of 2.5 mm, $CBR_{2.5}$, and 5.0 mm, $CBR_{5.0}$; maximum measured penetration force, F_{max}. In addition, some force-penetration curves are plotted in Figure 5. For most specimens tested, $CBR_{5.0}$ was higher than $CBR_{2.5}$. For the specimens prepared to the optimum water content, the CBR values obtained for the unreinforced soil are: $CBR_{2.5}$=4.67 and $CBR_{5.0}$=4.73 (in good agreement with the expected range for this type of soil).

Sample	Reinforcement N. of layers	w (%)	w_{real} (%)	$CBR_{2.5}$ (%)	$CBR_{5.0}$ (%)	F_{max} (kgf)
Soil	0	11.9	11.9	9.5	10.0	374.4
	0	13.9	13.8	4.7	4.7	185.8
	0	15.0	15.1	3.2	3.5	142.2
Soil + GC	1	11.9	11.9	12.3	12.2	464.0
	1	13.9	13.6	4.7	4.9	192.6
	1	15.0	15.0	3.4	3.7	161.1
	2	11.9	11.9	13.6	13.9	533.0

Table 5: Summary of the CBR tests results

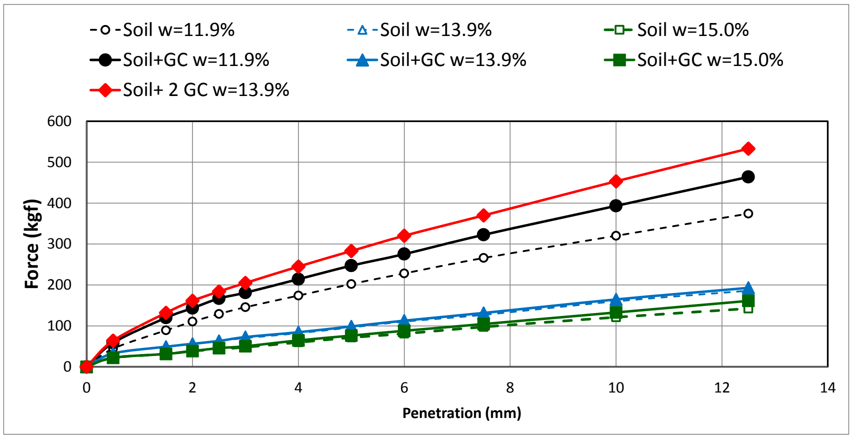

Figure 5: Force-penetration curves from the CBR tests

The results show that the reinforcement has enhanced the response of the soil-geosynthetic composite material, when compared with the unreinforced solution. Reinforcing the fine soil led to an increase of the CBR value and the maximum penetration force measured. The observation of the specimens after the tests showed that for the unreinforced soil specimens, the density of the soil under the loaded area increased. For the reinforced soil specimens there was an additional effect – the reinforcement deformed, following the deformations of the adjacent soil, assuming a concave shape (Figure 6). This indicates a transference of stresses from the soil to the reinforcement. The latter reinforcement mechanism is often designated as membrane tension support, as introduced by Giroud and Noray (1981). Part of the load applied, which would have been transmitted to the lower layers of soil, was transferred laterally to the adjacent soil by the geosynthetic, increasing the bearing capacity of the composite material. Another reinforcing mechanism occurring is the bearing capacity increase. In this case the reinforcement forces the potential bearing capacity surface failure to follow alternate paths, along surfaces with higher shear strength (Holtz et al. 1998).

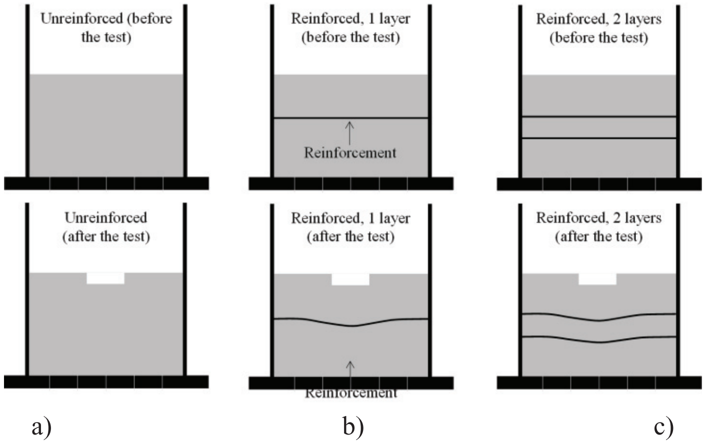

Figure 6: Schematic representation of the specimens: a) before and b) after the CBR tests

The force-penetration curves (Figure 5) showed that changing the initial water content of the specimens affected their CBR value. Higher initial water content led to lower CBR values, for both the unreinforced and reinforced specimens. Additionally, the specimens tested with higher initial water content values (optimum or on its wet side), exhibited lower benefit from including the reinforcement compared with specimens tested with water content on the dry side. Using one layer of reinforcement increased the CBR value of the samples relatively to those of the unreinforced sample. However, only for the specimens prepared with w=11.9 % the CBR value of the soil was significantly improved. Although there are significant variations in percentage, for the remaining values of the soil water content, the absolute CBR values are very similar to those of the unreinforced soil sample prepared to the same conditions, reflecting a low mechanical strength and bearing capacity of the soil.

Increasing the number of reinforcement layers (from 1 to 2) led to a further increase of the bearing ratio, when compared to the CBR of the reinforced soil with one layer of GC (11% for $CBR_{2.5}$, 14% for CBR_5) and when compared to that of the unreinforced soil (43% for $CBR_{2.5}$, 39% for CBR_5). A similar trend was observed for the maximum penetration force (increase of 15% and 42%, relatively to that of reinforced soil with one layer of GC and unreinforced soil, respectively).

4 Conclusions

In this paper the strength and bearing ratio of a fine soil reinforced with a geocomposite were analysed using triaxial and CBR tests. From the results the following conclusions can be established:

- Adding one layer of geocomposite improved the strength parameters and the bearing ratio of the soil-reinforcement composite material relatively to that of the unreinforced soil.
- The stress-strain response of the reinforced soil was less stiff than that of the fine soil, mostly due to the compressibility of the geocomposite.
- The beneficial effect of the reinforcement on the bearing ratio was highly dependent on the initial water content of the soil. The best response was observed for specimens with initial water content on the dry side of its optimum value.
- Increasing the number of reinforcement layers from one to two led to the best response observed, in terms of bearing ratio of the composite material.

References

AASHTO. (1991). M 145. Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes.

Abu-Farsakh, M.Y., Akond, I. & Chen, Q. (2015). Evaluating the performance of geosynthetic-reinforced unpaved roads using plate load tests. International Journal of Pavement Engineering, 1-12.

Adams, C.A., Tuffour, Y.A & Kwofie, S. (2016). Effects of Soil Properties and Geogrid Placement on CBR Enhancement of Lateritic Soil for Road Pavement Layers. American Journal of Civil Engineering and Architecture, 4(2), 62-66.

ASTM. (2007e2). D1883. Standard Test Method for CBR (California Bearing Ratio) of Laboratory-Compacted Soils.

ASTM. (2011). D2487. Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System.

ASTM. (2012). D1557. Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft3 (2,700 kN-m/m3)).

Chen, X., Zhang, J. & Li, Z. (2014). Shear behaviour of a geogrid-reinforced coarse-grained soil based on large-scale triaxial tests. Geotextiles and Geomembranes, 42(4), 312-328.

Giroud, J.P., & Noiray, L. (1981). Geotextile-reinforced unpaved roads. Journal of the Geotechnical Engineering Division, ASCE, 107(9), 1233-1254.

Holtz, R.D., Christopher, B.R., & Berg, R.R. (1998). Geosynthetic design and construction guidelines. Washington, D.C.: U.S. Department of Transportation, Federal Highway Administration.

ISO (2004). TS 17892-8. Geotechnical investigation and testing - Laboratory testing of soil. Part 8: Unconsolidated undrained triaxial test.

Kamel, M.A., Chandra, S. & Kumar, P. (2004) Behaviour of Subgrade Soil Reinforced with Geogrid. International Journal of Pavement Engineering, 5(4), 201-209.

LNEC. (1967). E198: Determinação do CBR. Especificação LNEC. Lisboa.

Moayed, R.Z., Nazari, M. & Allahyari, F. (2013). Effect of geosynthetic inclusion on the bearing ratio of two-layered soil. Journal of the Chinese Institute of Engineers, 36(7), 914-931.

Nair, A. & Latha, G. (2014). Large Diameter Triaxial Tests on Geosynthetic-Reinforced Granular Subbases. Journal of Materials in Civil Engineering, 27(4), 04014148.

Nguyen, M.D., Yang, K.H., Lee, S.H., Wu, C.S. & Tsai, M.H. (2013). Behavior of nonwoven-geotextile-reinforced sand and mobilization of reinforcement strain under triaxial compression. Geosynthetics International, 20(3), 207-225

Noorzad, R. & Mirmoradi, S.H. (2010). Laboratory evaluation of the behavior of a geotextile reinforced clay. Geotextiles and Geomembranes, 28(4), 386–392.

Ruiken, A., & Ziegler, M. (2008). Effect of reinforcement on the load bearing capacity of geosynthetic reinforced soil (paper 200). 4th European Geosynthetics Conference. Edinburgh, United Kingdom.

Ruiken, A., Ziegler, M., Vollmert, L., & Duzic, I. (2010). Recent findings about the confining effect of geogrids from large scale laboratory testing. 9th International Conference on Geosynthetics, (pp. 691-694). Guarujá, Brazil.

Saez, J. (1997). Caracterizacion geomecanica de geotextiles. Curso sobre técnicas generales de refuerzo del terreno y sus aplicaciones. Madrid, Espanha: Centro de estudios y experimentación de obras públicas (CEDEX).

Sieira, A. C. (2003). Estudo experimental dos mecanismos de interação solo-geogrelha. Rio de Janeiro, Brazil: Tese de Doutoramento em Ciências de Engenharia Civil (Geotecnia), PUC-Rio.