Geosynthetics - 7th ICG - Delmas, Gourc & Girard (eds) © 2002 Swets & Zeitlinger, Lisse ISBN 90 5809 523 1

Evaluation of soil-geosynthetic interface resistance using inclined plane shear tests and pullout tests

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ABSTRACT: The aim of this paper is to compare the evaluation of soil-geosynthetic interface resistance using two different tests: inclined plane shear tests and pullout tests. The adequacy of the tests is discussed. The tests were carried out according with the European standards: prEN ISO 12957-2 and prEN 13738. This paper reports the results of inclined plane shear and pullout tests on three different geosynthetics, which have approximately the same nominal strength but different structures, embedded in two granular soils. Both soils were compacted for the same relative density. The influence of soil particle size and geosynthetic structure is discussed by analysing the results of the tests. In inclined plane shear tests the role of the test method is discussed, and in the pullout tests the influence of geogrids bearing members is also studied. The main conclusions that can be outlined from the study are presented.

1 INTRODUCTION

The aim of this paper is to compare the evaluation of soil-geosynthetic interface resistance using two different methods: inclined plane shear tests and pullout tests.

The adequacy of each test to be used to evaluate the interface resistance depends on the relative movement that occurs in the interface. On one hand, inclined plane shear tests should be used to characterize the interaction mechanism at soil-geosynthetic interface when the relative movement that occurs is of shearing and the geosynthetic is placed over an inclined surface; on the other hand, pullout tests should be used to characterize soil-geosynthetic reinforcement interaction with relative movement, as the geosynthetic is subjected to pullout.

2 MATERIALS

2.1 *Soils*

The soils used in the tests are two sands (referred as Soil 1 and Soil 2), with different particle size distributions (Figure 1).

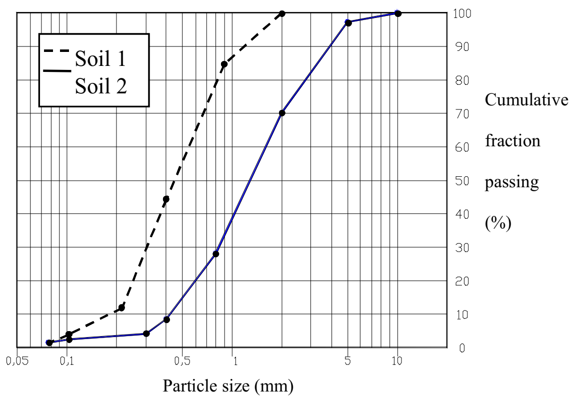

Figure 1. Particle size distributions for Soils 1 and 2.

Soil 1 minimum and maximum unit weight values range from 15.0 to 17.9kN/m^3 for soil particle diameter values that range typically from 0.0074 to 2.00mm (Figure 1). Soil 2 minimum and maximum unit weight values range from 15.6 to 18.7kN/m^3 for soil particle diameter values ranging typically from 0.0074 to 9.54mm (Figure 1). The physical properties of the two soils are presented in Table 1.

Both soils were compacted to a target relative density of I_D=50%, so the results obtained with the two different granular materials could be comparable.

Table 1. Physical properties of the soils used in the test program.

Soil	γ_{min} (kN/m^3)	γ_{max} (kN/m^3)	$\gamma(I_D=50\%)$ (kN/m^3)	D_{50} (mm)	D_{10} (mm)	C_u (D_{60}/D_{10})	ϕ (º)
Soil 1	15.0	17.9	16.32	0.43	0.18	2.94	36
Soil 2	15.6	18.7	17.01	1.3	0.44	3.64	45

2.2 *Geosynthetics*

The following geosynthetics were used in the study:

1. a high density polyethylene uniaxial geogrid (GG1);
2. a polypropylene biaxial geogrid (GG2);
3. a polypropylene nonwoven spunbounded geotextile (GT1).

The most important geosynthetic properties for the present study are presented in Tables 2 to 4 and in Figures 2 and 3.

Table 2. Characteristics of uniaxial geogrid GG1.

a_L (mm)	a_T (mm)	b_B (mm)	b_R (mm)	t_B (mm)	t_R (mm)	Tensile strength (kN/m)	Peak strain (%)	Tensile stiffness (kN/m)
160.0	16.0	16.0	6.0	2.7_{max} 2.5_{min}	0.9	55.0	11.5	478.3

Table 3. Characteristics of biaxial geogrid GG2.

a_L (mm)	a_T (mm)	b_{LR} (mm)	b_{TR} (mm)	t_J (mm)	t_{LR} (mm)	t_{TR} (mm)	Tensile strength (kN/m)	Peak strain (%)	Tensile stiffness (kN/m)
33.0	33.0	2.2	2.5	5.8	2.2	1.4	40.0	11.5	347.8

Table 4. Characteristics of geotextile GT1.

Mass per unit area (g/m^2)	Thickness (mm)	Tensile strength (kN/m)	Peak strain (%)	Tensile stiffness (kN/m)
800.0	6	50.0	65.0	76.9

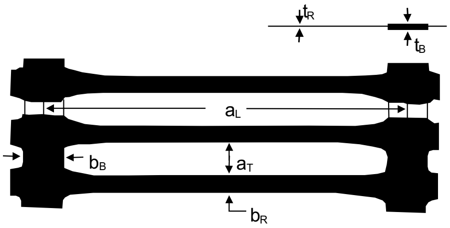

Figure 2. Geometry of uniaxial geogrid GG1.

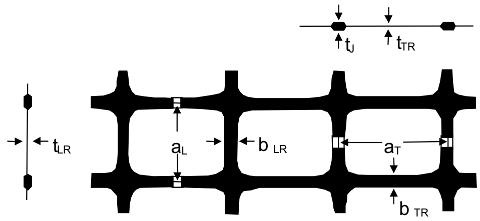

Figure 3. Geometry of biaxial geogrid GG2.

3 TEST PROGRAM

The test program carried out to study the friction characteristics of the soil-geosynthetic interface is presented in Table 5 and corresponds to 9 different inclined plane shear tests and 7 different pullout tests; as each one was repeated twice a total of 48 tests was carried out.

Table 5. Summary of the tests carried out in the inclined plane shear and pullout tests program.

Test	Geosynthetic	Soil	Test type	Test method
T1	GG1		Inclined	
T2	GG2	1	plane	1
T3	GT1		shear	
T4	GG1		Inclined	
T5	GG2	1	plane	2
T6	GT1		shear	
T7	GG1		Inclined	
T8	GG2	2	plane	1
T9	GT1		shear	
T10	GG1			
T11	GG2	1	Pullout	-
T12	GT1			
T13	GG1			
T14	GG2			
T15	GT1	2	Pullout	-
T16	$GG1_{WBM}$*			

*Tests performed with the bearing members of the geogrid cut between ribs in the direction of pullout.

The inclined plane shear and pullout tests were carried out according with the European standards prEN ISO 12957-2 and prEN 13738, respectively. In the case of the inclined plane shear test two different methods were used: with a rigid support for the geosynthetics (Test method 1); and with the geosynthetic supported on a lower box filled with soil (Test method 2).

The test apparatus and procedures related with these two tests are fully described in Lopes *et al.* (2001) and Lopes & Lopes (1999), respectively.

4 ANALYSIS OF TEST RESULTS

4.1 *Inclined plane shear tests*

To evaluate the influence of the geosynthetic structure, the results corresponding to tests carried out with Soil 1 and Test method 1 (Tests T1 to T3 in Table 5) are considered (Table 6). Soil particle size is one of the factors affecting the soil-geosynthetic interface behaviour, the results obtained in the tests carried out on all the geosynthetics, using Test method 1, with Soils 1 and 2 (Tests T1 to T3 and T7 to T9 in Table 5) are presented in Table 6. To evaluate the influence of the test method, the test results for Soil 1 and Test methods 1 and 2 (Tests T1 to T6 in Table 5) are considered (Table 6). In Figure 4 the inclined plane shear behaviour of uniaxial geogrid GG1 embedded in Soils 1 and 2 is presented.

Table 6. Average soil-geosynthetic interface friction angle values obtained from inclined plane shear tests.

Friction angle (º)	Soil	Test method	Geosynthetic GG1	GG2	GT1
	Soil 1	1	27.6	30.1	32.2
ϕ_{sg}*	Soil 1	2	31.8	30.8	33.2
	Soil 2	1	29.5	33.0	32.9

*Soil-geosynthetic interface friction angle value.

As far as the inclined plane shear tests results are concerned the main conclusions are: a) geosynthetics' structure has an important role on the friction angle of soil-geosynthetic interface; b) geotextiles lateral surface (GT1) is associated with higher friction angle of soil-geosynthetic interface; c) soil particle size has an important influence on the friction angle of soil-geosynthetic interface; d) wider soil particle size distributions, with larger average soil particle size, allows an increase on the soil-geosynthetic interface resistance; e) the validity of evaluating the soil-geogrid interface resistance using Test method 1 depends on the structure of the geogrid; it is suggested to use Test method 2 for geogrids.

This last conclusion demands an explanation. In fact, the results obtained using geogrid GG2 show a slight variation of the soil-geosynthetic interface friction angle (increase of 2.3%) when using Test method 2 instead of Test method 1. For geogrid GG1, the increase of the soil-geogrid interface friction angle is higher, about 15%, when using Test method 2. These results can be explained as follows:

1. the relationship between aperture area and solid surface area of the two geogrids is larger for geogrid GG2 than for geogrid GG1;
2. the joints of the geogrid GG2 exhibit a rebound in the lateral superior and inferior surfaces, while the joints of geogrid GG1 are perfectly smooth;
3. using Test method 1 on geogrid GG2, the rebound in the joints on the lateral inferior surface and the significant lateral open area allow the existence of a thin layer of soil beneath the geogrid; this is similar to the test conditions using Test method 2; when testing geogrid GG1 this layer of soil is not as significant;
4. the mobilized friction on the geogrid apertures is much lower using Test method 1 on geogrid GG1 than Test method 2, due to the formation of a soil-polished metal interface and soil-soil interface, respectively.

Therefore, it is verified that Test method 2 is more adequate to study soil-geogrids interface resistance.

The influence of the test method on the results obtained for the geotextile tested is small.

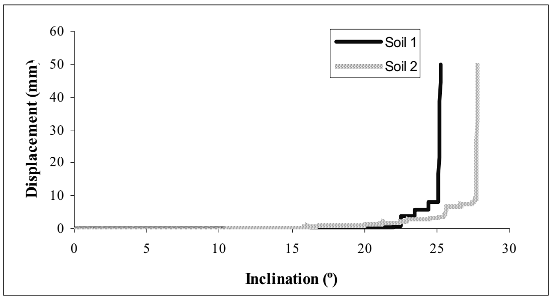

Figure 4. Inclined plane shear behaviour for uniaxial geogrid GG1 embedded in Soils 1 and 2.

4.2 *Pullout tests*

To evaluate the soil-geosynthetic interface resistance using pullout tests, the results corresponding to tests carried out with Soil 1 and with Soil 2 (Tests T10 to T16 in Table 5) are considered (see Table 7). The test T16 was carried out to evaluate the influence of the presence of the bearing members of geogrid GG1 (Table 7). In Figure 5 the pullout behaviour of uniaxial geogrid GG1 embedded in Soils 1 and 2 is presented.

As it is clear in Table 7, most of the tests carried out with these geosynthetics (except for GG1) fail by lack of tensile strength. This means that the pullout resistance of those materials is higher than the geosynthetics' tensile strength.

Table 7. Results obtained from the pullout tests: maximum pullout force, F_{max}, and the corresponding frontal displacement, u_{max}.

Parameter	Soil	Geosynthetic			
		GG1	GG2	GT1	$GG1_{WBM}$
F_{max} (kN/m)	Soil 1	40.4	32.5*	14.2*	-
	Soil 2	47.3	31.2*	12.3*	25.4
u_{max} (mm)	Soil 1	106.3	58.7	161.3	-
	Soil 2	131.1	47.4	199.8	70.5

*Failure occurs by lack of tensile strength.

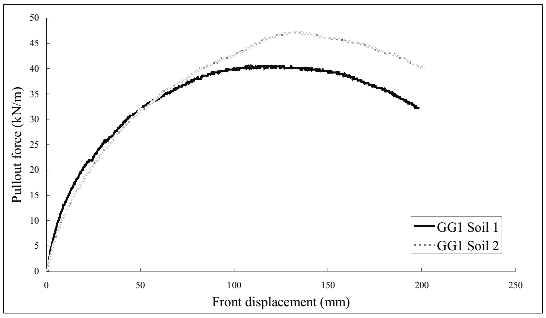

Figure 5. Pullout behaviour for uniaxial geogrid GG1 embedded in Soils 1 and 2.

Therefore, regarding the pullout tests the main conclusions can be summarized: a) soil particle size has an important influence on soil-geosynthetic interface behaviour, clear for GG1; b) an increase of soil-geogrid shear resistance of 17% was observed when the soil contained a significant percentage of particle sizes slightly greater than the geogrid bearing member thickness, but lower than the geogrid aperture size; c) cutting the geogrid bearing members leads to a significant decrease in the soil-geogrid interface shear resistance (46%), thus, clarifying the important role of the mobilization of passive resistance on transverse geogrid members.

4.3 *Soil-geosynthetic interface coefficient: f*

To compare the values obtained from the different tests carried out to characterize the resistance of the soil-geosynthetic interface, the interface coefficient was calculated for each case.

For inclined plane shear tests, the soil-geosynthetic interface coefficient (f) is determined using Equation 1:

$$f = \frac{\tan\delta}{\tan\phi'} \tag{1}$$

where $0 \leq f \leq 1$, δ is the average soil-geosynthetic interface friction angle (obtained from the test data) and ϕ' is the soil internal friction angle, in terms of effective stresses.

As far as pullout tests are concerned, the resistance to shear in the soil-reinforcement interface (measured from the test data) can be determined by Equation 2:

$$T = 2\, b\, l\, \sigma'_n\, f \tan\phi' \tag{2}$$

where $0 \leq f \leq 1$, σ'_n is the normal effective stress in the interface, b is the reinforcement width and l is the resistant length. Then it is possible to determine f, from the pullout tests data.

Therefore, using Equations 1 and 2, the soil-geosynthetic interface coefficient was determined from the tests data. The values determined are presented in Table 8.

Table 8. Soil-geosynthetic interface coefficient values obtained from the inclined plane shear and pullout tests carried out.

Test	Geosynthetic	Soil	Test type	Test method	f
T1	GG1		Inclined		0.72
T2	GG2	1	plane	1	0.80
T3	GT1		shear		0.87
T4	GG1		Inclined		0.85
T5	GG2	1	plane	2	0.82
T6	GT1		shear		0.90
T7	GG1		Inclined		0.57
T8	GG2	2	plane	1	0.65
T9	GT1		shear		0.65
T10	GG1				0.76
T11	GG2	1	Pullout	-	0.61**
T12	GT1				0.27**
T13	GG1				0.65
T14	GG2				0.43**
T15	GT1	2	Pullout	-	0.17**
T16	$GG1_{WBM}$*				0.35

*Tests performed with the bearing members of the geogrid cut between ribs in the direction of pullout.
** Failure occurs by lack of tensile strength.

Comparing the results for GG1 embedded in Soil 1 obtained from inclined plane shear (Test method 1) and pullout tests, it is clear that the interface coefficient is higher in the case of the pullout tests. This was expectable since in the inclined plane shear test the passive resistance on the bearing members of the geogrid is not mobilized.

In the case of GG2 and GT1, under those test conditions, apparently the soil-geosynthetic interface coefficient for pullout tests is lower than for inclined plane shear tests. However, as in the pullout tests the failure occurs by lack of tensile strength, the meaning of those values is questionable. In fact, as the pullout strength is higher than the materials' tensile strength, it is only possible to conclude that the value for the interface coefficient from pullout should be greater. Therefore, the f values for the two test types should not be compared.

Analyzing the results for GG1, GG2 and GT1 embedded in Soil 2 obtained from inclined plane shear (Test method 1) and pullout tests, similar conclusions are obtained.

As expected, for the pullout tests, the results obtained for $GG1_{WBM}$ correspond to a lower interface coefficient, emphasizing the role of the passive resistance mobilized on the geogrid bearing members: a reduction of 46% on the *f* value was observed.

The results obtained with the geosynthetics embedded in Soil 2 instead of Soil 1, for both inclined plane shear and pullout tests, show lower interface coefficient. In fact, the increase observed, when using Soil 2, in soil-geosynthetic interface friction angle values, from inclined plane shear tests, and maximum pullout force, from pullout tests, is less than the difference between the internal friction angles of Soils 2 and 1.

From experimental data (Palmeira & Milligan, 1989), obtained in pullout tests of metallic grids embedded in different granular materials, Jewell (1996) suggests the use of a scale factor, F_1, which represents the scale effects, due to particle average size (D_{50}). This factor is defined as:

$$F_1 = \left(2 - \frac{B}{10D_{50}}\right) \text{ when } B/D_{50}<10 \tag{3}$$

$$F_1 = 1.00 \text{ when } B/D_{50}>10 \tag{4}$$

where B is the height of the longitudinal members of the geogrid.

Applying these expressions to the pullout tests data, it is possible to predict the increase on the interface strength of GG1 embedded in Soil 1 and Soil 2 (Table 9).

The increase on the interface strength, when using Soil 2 instead of Soil 1, is obtained by Equation 5 for the experimental results. For the predicted values the maximum force, $F_{max,}$ is replaced by the scale factor, F_1.

$$\Delta F_{max} = \frac{F_{max(Soil2)} - F_{max(Soil1)}}{F_{max(Soil1)}} \times 100\,(\%) \tag{5}$$

The predicted increase on the soil-geogrid interface global strength is approximately the double of the value obtained experimentally. Beyond the different test procedures and conditions, this difference can be explained by the types of reinforcements considered: GG1 is in HDPE material, extensible, that deforms while moving relatively to the soil, due to the different stages of strength mobilization along the reinforcement and in the bearing members; while the grids used by Palmeira & Milligan (1989) were metallic, inextensible, allowing simultaneous mobilization of the strength mobilization along the reinforcement and in all its bearing members. Therefore, extensible materials can mobilize lower strength than inextensible ones. Further more, in the case of extensible materials, the increase of the passive thrust on the bearing members due to the soil particles size can be responsible for the increase on the deformation during pullout.

Also according with Jewel (1996), the theoretical upper and lower limits for the ratio to the normal effective stress in the interface of the effective passive stress mobilized, (σ'_p/σ'_n), can be defined as Equations 6 and 7, respectively:

$$\left(\frac{\sigma'_p}{\sigma'_n}\right) = \tan^2\left(\frac{\pi}{4} + \frac{\phi'}{2}\right) e^{\pi \tan\phi'} \tag{6}$$

$$\left(\frac{\sigma'_p}{\sigma'_n}\right) = \tan\left(\frac{\pi}{4} + \frac{\phi'}{2}\right) e^{\left(\frac{\pi}{2}+\phi'\right)\tan\phi'} \tag{7}$$

According with the same author, the soil-geogrid interface resistance due to the passive strength mobilized on the geogrid bearing members can be obtained by Equation 8, where (l/s) is the number of bearing members of the reinforcement and σ'_p is the effective passive stress mobilized.

$$T_p = \left(\frac{l}{s}\right) a_b b B \sigma'_p \tag{8}$$

The upper and lower limit values were determined, for GG1, and compared with the experimental result (Table 10). The result obtained from the pullout tests with GG1 is in the range of the theoretical values, and is closer to the lower limit.

Table 9. Scale effects (F_1) due to particle average size (D_{50}) for GG1, according with Jewel (1996) and the predicted and obtained increases on the interface strength (ΔF_{max}).

Geogrid	Soil 1 B/D_{50}	F_1	Soil 2 B/D_{50}	F_1	ΔF_{max} Jewel (1996)	Tests data
GG1	6.05	1.40	2.00	1.80	29%	17%

Table 10. Upper and lower limit values of for the bearing strength (kN/m) on GG1, according with Jewel (1996), and experimental result.

Geogrid	Upper limit Equation 5	Lower limit Equation 6	Experimental result
GG1	17.51	3.41	6.90

5 CONCLUSIONS

From both inclined plane shear and pullout tests the main conclusions that can be stated are: 1) geosynthetics' structure and soil particle size have an important influence on soil-geosynthetic interface behaviour; 2) wider soil particle size distributions, with larger average soil particle size, induce an increase of the strength and of the friction angle, for pullout and inclined plane shear tests, respectively, but a decrease in the interface coefficient; 3) comparing the values obtained for the two types of test (for GG1), the interface coefficient is higher for pullout tests; 4) it is suggested to use Test method 2 in inclined plane shear tests of geogrids; 5) in the pullout tests, cutting the geogrid bearing members leads to a significant decrease of the interface coefficient; 6) the difference observed between the experimental results and the theoretical values, suggests an adoption of a scale factor F_1, lower than the proposed by Jewel (1996) for inextensible grids; 7) the experimental results for the the soil-geogrid interface resistance, due to the passive strength mobilized on the geogrid bearing members, are in the range of the theoretical values.

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