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Soil-geosynthetic interface shear behaviour: Insights from inclined plane and direct shear tests

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ABSTRACT: The assessment of soil-geosynthetic interface shear strength properties is essential for the safe design of geosynthetic-reinforced soil systems. In this study, a series of inclined plane and direct shear tests was carried out to evaluate the shear strength parameters of the interfaces between a residual soil from granite and two different geosynthetics: an extruded geogrid and a geocomposite reinforcement. The influence of soil moisture content was analysed under inclined plane and direct shear modes by compacting the soil at the optimum moisture content (w_{opt}) and 2% wet of the w_{opt}. The direct shear test results show that the increase in soil moisture content may lead to a considerable reduction in the apparent cohesion of the soil-geosynthetic interface. In general, higher shear stresses were reached at the interface involving the geocomposite reinforcement. Moreover, the interface shear strength parameters established from direct shear test results generally exceeded those obtained by inclined plane tests, particularly in terms of the apparent cohesion value.

1 INTRODUCTION

Understanding the shear behaviour of soil-geosynthetic interfaces is of the utmost relevance for the safe design of geosynthetic-reinforced systems. One of the major benefits from the use of geosynthetics as reinforcement elements in geotechnical structures is the possibility of using lower quality locally available backfill materials, such as cohesive and/or residual soils and recycled waste materials, particularly in cases where higher quality granular soils are not readily available. However, low quality soils are often more susceptible to the detrimental effects of moisture content increase, and thus the influence of moisture content on the behaviour of soil-geosynthetic interfaces involving these soils should be thoroughly evaluated (Ferreira *et al.* 2013, 2015).

The direct shear test is commonly used to investigate the shear strength properties of soil-geosynthetic interfaces subjected to relatively high normal stresses. The inclined plane test can alternatively be used to assess the interface shear strength under lower confining pressures. This method is particularly well suited to study the interaction between soils and geosynthetics when these materials are installed on inclined plane surfaces (e.g. erosion protection systems and landfill liner/cover systems), since it can more accurately reproduce the slippage that may occur on slopes (Ferreira *et al.* 2014; Lopes *et al.* 2014).

The main aim of the present study is to characterise the shear behaviour of the interfaces between a locally available residual soil from granite and two geosynthetics, specifically a

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uniaxial extruded geogrid and a uniaxial geocomposite reinforcement, by using two distinct test methods (the inclined plane and the direct shear test), while establishing a comparison between the interface shear strength parameters estimated from both methods. Special care was taken to ensure that the specimen preparation was identical for both types of test to avoid any external factors that could hamper the comparison of results. The influence of soil moisture content and geosynthetic type on the soil-geosynthetic interface shear strength under inclined plane and direct shear modes is also evaluated and discussed in this paper.

2 MATERIALS AND METHODS

2.1 *Materials*

The soil used in this study was a locally available residual soil from granite with the particle size distribution curve presented in Figure 1a. This soil is classified as well-graded sand with silt and gravel (SW-SM) according to the Unified Soil Classification System. The maximum dry unit weight and optimum moisture content ($w = w_{opt}$) obtained from the Modified Proctor test (CEN 2010) are 18.9 kN/m^3 and 11.5%, respectively. The internal soil strength was evaluated by large-scale direct shear tests. For $w = w_{opt}$ and dry unit weight of 17.5 kN/m^3, the soil shear strength can be characterised by a friction angle of 42.4° and cohesion of 13.6 kPa.

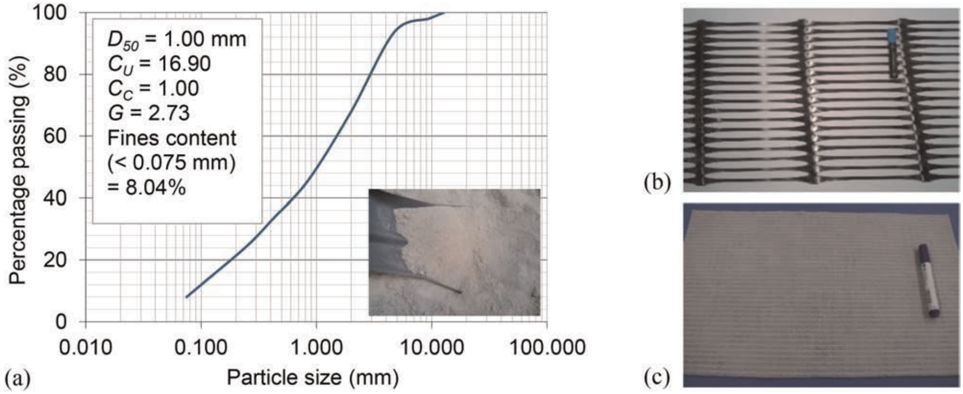

Figure 1. (a) Particle size distribution curve of the residual soil from granite; (b) GGR; (c) GCR.

Two commercially-available geosynthetics were employed, specifically a uniaxial extruded geogrid (GGR) manufactured from high-density polyethylene, with a mean grid size of 22 × 235 mm (Figure 1b), and a uniaxial geocomposite reinforcement (GCR), consisting of a continuous filament nonwoven polypropylene geotextile reinforced with high modulus polyester yarns (Figure 1c). The tensile load-strain behaviour of the geosynthetics was assessed by wide-width tensile tests following the EN ISO 10319:2015 (CEN 2015). The average values of maximum tensile strength (T_{max}), strain at maximum load (ε_{Tmax}) and secant stiffness at 5% strain ($J_{5\%}$) for the geogrid (machine direction) were 52.2 kN/m, 12.4% and 509.8 kN/m, respectively. For the geocomposite, the corresponding values were 54.6 kN/m, 10.6% and 600.9 kN/m, respectively.

2.2 *Test devices and methods*

Two prototype test facilities were used to carry out the experimental programme: an inclined plane test device and a large-scale direct shear test device. The inclined plane device is composed of a rigid lower box (510 mm × 350 mm in plan and 80 mm high), a rigid upper box (300 mm × 300 mm in plan and 80 mm high) and a rigid base (620 mm × 430 mm in plan and 10 mm high). According to the EN ISO 12957-2:2005 (CEN 2005), the test can be conducted by using a rigid support for the geosynthetic or with the lower box filled with soil. The tests reported herein were performed using the latter method. During the tests, the base plane was raised at a rate of 0.5º/min and the relative displacement between the upper box and the geosynthetic was monitored. The vertical force was applied by weights transmitted to a rigid steel plate installed over the soil specimen. To ensure that the vertical force approximately passed through the centre of gravity of the upper box, two wedges inclined 1:2 were placed on its front and back walls. The vertical stress (σ_v) was applied for 60 min prior to raising the base plane. Further details about the device, instrumentation and test procedures can be found elsewhere (Ferreira *et al.* 2014; Lopes *et al.* 2014).

The direct shear test device consists essentially of a steel shear box (including upper and lower boxes), a support structure, hydraulic actuators and respective power unit, an electric cabinet and a set of displacement and pressure transducers. The internal dimensions of the upper and lower boxes are 600 mm × 300 mm in plan and 50 mm height, and 800 mm × 340 mm in plan and 100 mm height, respectively. The tests may be performed according to the method of constant or reduced contact area. The present tests were conducted according to the latter method by placing a rigid ring inside the lower box, which enabled equally sized upper and lower halves to be filled with soil. Prior to shearing, the normal stress was applied to the specimens by a rigid metal plate for a period of 60 min. Following the EN ISO 12957-1:2018 (CEN 2018), a constant displacement rate of 1 mm/min was used. A detailed description of the test facility and procedures can be found in Vieira *et al.* (2013) and Ferreira *et al.* (2013, 2015).

2.3 *Test programme*

A series of inclined plane (IPT) and direct shear tests (DST) was carried out to estimate the shear strength parameters of the interfaces between the residual soil from granite and the geosynthetics. The IPT were performed under vertical stresses (σ_v) of 5, 10 and 25 kPa, as recommended by the EN ISO 12957-2:2005 (CEN 2005), and each test was carried out twice under identical conditions. The DST were conducted under normal stresses (σ_n) of 25, 50, 100 and 150 kPa. Soil dry unit weight was kept constant in all of the tests (17.5 kN/m^3). The interface shear strength variation associated with an increase in soil moisture content (w), which may potentially occur under field conditions, was evaluated under inclined plane and direct shear modes by compacting the soil at the optimum moisture content (w_{opt} = 11.5%) and 2% wet of the w_{opt} (w =13.5%).

3 RESULTS AND DISCUSSION

3.1 *Influence of soil moisture content*

Figures 2a, b present the evolution of the upper box displacement with the inclination of the shear box from inclined plane tests on the soil-GGR and soil-GCR interfaces performed under σ_v =10 kPa and different soil moisture contents (w_{opt} and w_{opt} + 2%). It can be observed that the influence of soil moisture content on the soil-geosynthetic interface shear strength under inclined plane mode was more significant when the geogrid was used (Figure 2a). For this interface, the slipping angle of the upper box (i.e. the inclination leading to the displacement of 50 mm) increased with the moisture content, implying that the moisture content increase (from w_{opt} to w_{opt} + 2%) produced a beneficial effect on the interface shear strength. However, for the interface involving the geocomposite reinforcement (Figure 2b), the variation in soil moisture content did not significantly influence the slipping angle, and hence the interface shear strength. This is possibly associated with the structure and moisture absorption capacity of this geosynthetic.

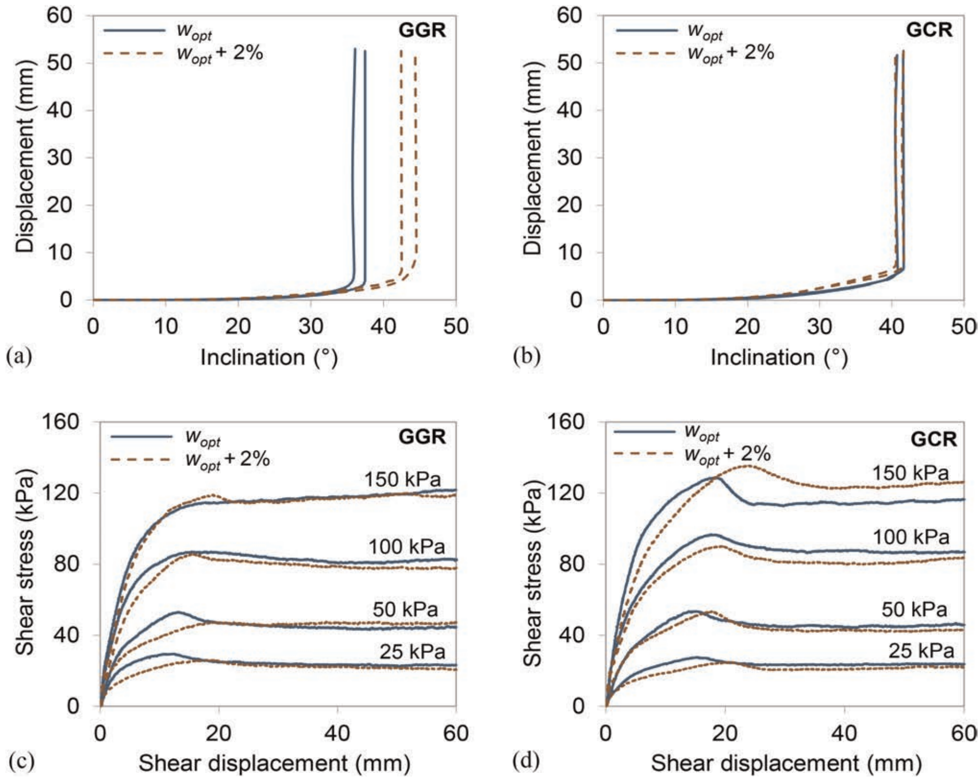

Figure 2. Influence of soil moisture content on soil-geosynthetic interface behaviour: (a) inclined plane tests on soil-GGR interface (σ_v = 10kPa); (b) inclined plane tests on soil-GCR interface (σ_v = 10kPa); (c) direct shear tests on soil-GGR interface; (d) direct shear tests on soil-GCR interface.

Figures 2c, d compare the shear stress-shear displacement behaviour of the soil-GGR and soil-GCR interfaces obtained from direct shear tests carried out under different soil moisture conditions (w_{opt} and w_{opt} +2%). For the soil-GGR interface (Figure 2c), the increase in moisture content led to a considerable reduction in the maximum interface shear strength under lower normal stress values (by up to 11.5% for σ_n=25 kPa). When subjected to higher normal stresses (σ_n=100 and 150 kPa), the influence of soil moisture content on the maximum interface shear strength was almost negligible. Similarly, the maximum shear strength of the soil-GCR interface (Figure 2d) also tended to reduce with increasing moisture content (by up to 10.5% for σ_n=25 kPa).

3.2 *Influence of geosynthetic type*

The influence of geosynthetic type on the soil-geosynthetic interface behaviour under inclined plane and direct shear modes is illustrated in Figure 3. It is noteworthy that the same type of support for the geosynthetic was used in all of the tests (i.e. lower box filled with soil) to enable the comparison of results. The inclined plane test data show that, when the soil moisture content is equal to w_{opt}, the slipping angles achieved at the interface involving the GCR were greater than those for the interface involving the GGR (Figure 3a). However, when the moisture content was increased (Figure 3b), the slipping angles were similar for both interfaces.

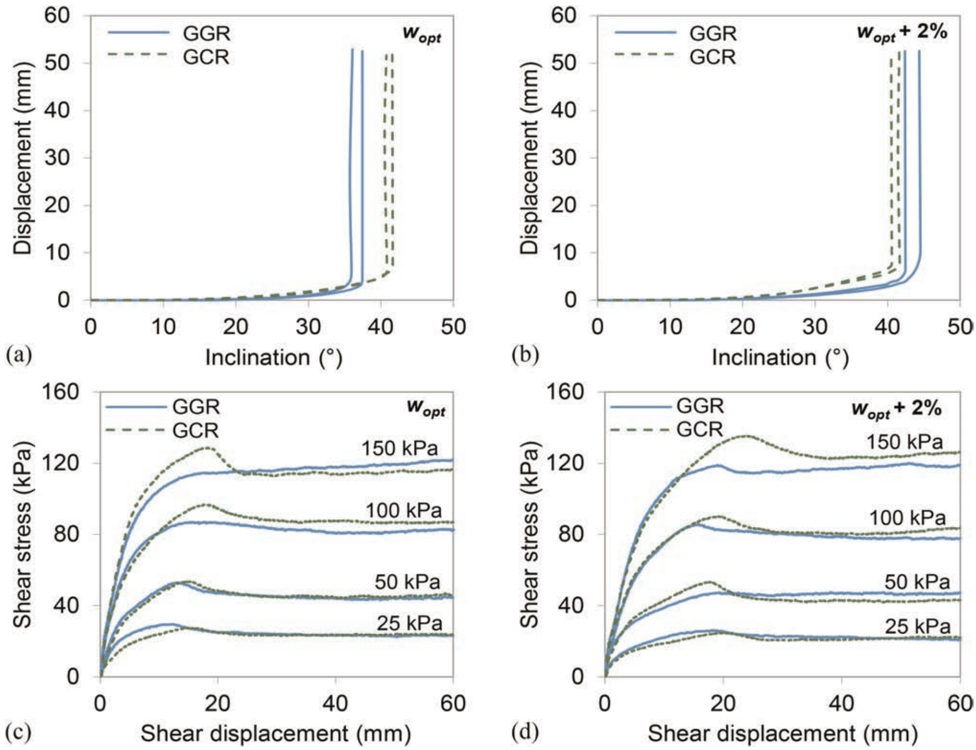

Figure 3. Influence of geosynthetic type on soil-geosynthetic interface behaviour: (a) inclined plane tests for $w = w_{opt}$ (σ_v = 10kPa); (b) inclined plane tests for $w = w_{opt}$ +2% (σ_v = 10kPa); (c) direct shear tests for $w = w_{opt}$; (d) direct shear tests for $w = w_{opt}$ +2%.

The results from the direct shear tests, shown in Figures 3c, d indicate that the maximum shear stresses reached at the soil-GCR interface generally exceeded those at the soil-GGR interface, regardless of moisture content. In addition, the increase in the maximum interface shear strength attributed to the use of the geocomposite became more pronounced as the normal stress was progressively increased. The higher values of shear stress reached for the interface with the geocomposite may be associated with the higher surface roughness of this geosynthetic, comparatively with the geogrid, which promoted higher mobilisation of frictional resistance during shearing.

3.3 *Inclined plane test versus direct shear test*

Soil-geosynthetic interface shear strength parameters are generally estimated from direct shear tests by fitting a straight line through the plot of peak shear stress versus normal stress, which represents the interface shear strength envelope. Based on the Mohr-Coulomb failure criterion, the shear strength parameters, specifically the interface friction angle (δ) and apparent cohesion (c_a) are derived. A comparable analysis may be performed from the inclined plane test results, taking into account the shear stress and normal stress acting at the interface when failure occurs (i.e. at the base inclination leading to a relative displacement of 50 mm) (Ferreira *et al.* 2016).

Figure 4 compares the shear strength envelopes and associated shear strength parameters estimated from inclined plane and direct shear tests on the soil-GGR (Figures 4a, b) and soil-GCR (Figures 4c, d) interfaces under different soil moisture conditions (w_{opt} and w_{opt} + 2%).

These results show that the shear strength parameters obtained from direct shear tests were generally higher than those estimated from inclined plane tests for a specific interface. The interface friction angles derived from the direct shear tests slightly exceeded those from the inclined plane tests (by up to 5%). On the other hand, the values of the interface apparent cohesion determined from the inclined plane tests did not exceed 1.7 kPa, whereas the values obtained from the direct shear tests were significantly higher. The above findings are consistent with previous studies on soil-geosynthetic (Izgin & Wasti 1998, Ferreira *et al.* 2016) and geosynthetic-geosynthetic interfaces (Girard *et al.* 1990; Wasti & Özdüzgün 2001), in which the interface shear strength parameters derived from direct shear tests exceeded those from inclined plane tests conducted on an identical interface. This indicates that the extrapolation of the linear interface shear strength envelope established from direct shear test results for normal stresses below the range over which the tests are conducted may be nonconservative. Therefore, in cases where the soil-geosynthetic interface is expected to be subjected to low normal loads during construction or throughout the service life of the structure (e.g. erosion protection systems and landfill liner/cover systems), the inclined plane test should be used for more accurate prediction of the soil-geosynthetic interface shear strength.

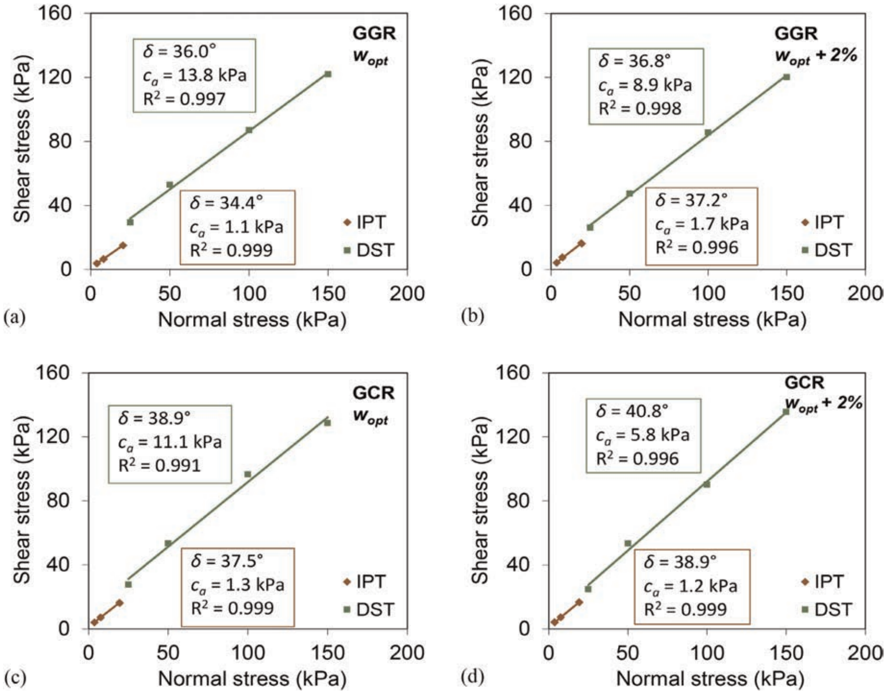

Figure 4. Comparison of soil-geosynthetic interface shear strength envelopes from inclined plane and direct shear tests: (a) soil-GGR interface ($w = w_{opt}$); (b) soil-GGR interface ($w = w_{opt} + 2\%$); (c) soil-GCR interface ($w = w_{opt}$); (d) soil-GCR interface ($w = w_{opt} + 2\%$).

4 CONCLUSIONS

A series of inclined plane and direct shear tests was conducted to investigate the effect of soil moisture content and geosynthetic type on the soil-geosynthetic interface shear strength and to compare the shear strength parameters estimated on the basis of the aforementioned test methods.

Under inclined plane shear mode, the increase in soil moisture content from w_{opt} to w_{op} + 2% did not induce any reduction in the soil-geosynthetic interface shear strength. However, under direct shear test conditions, the interface shear strength tended to reduce with increasing moisture content, particularly under lower normal stress values. Maximum shear strength reductions of 11.5% and 10.5% were obtained for the interfaces with the GGR and the GCR, respectively.

In general, higher shear stress values were reached at the interface involving the geocomposite reinforcement, regardless of soil moisture content, which may be attributed to the rougher surface of this geosynthetic, in comparison to that of the geogrid.

The interface shear strength parameters obtained from the direct shear tests generally exceeded those from the inclined plane tests conducted on the same interface. While the values of friction angle determined from the direct shear tests were only slightly higher (by up to 5%) than those estimated from the inclined plane tests, the apparent cohesion values were significantly greater when derived from the former method.

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