This manuscript is the accepted version of the paper:

Direct shear behaviour of residual soil-geosynthetic interfaces - influence of soil moisture content, soil density and geosynthetic type, Geosynthetics International, Vol. 22, Issue 3, pp. 257-272, https://doi.org/10.1680/gein.15.00011

Direct shear behaviour of residual soil-geosynthetic interfaces – influence of soil moisture content, soil density and geosynthetic type

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ABSTRACT: Soil-geosynthetic interface shear strength is an essential parameter for the design and stability analysis of geosynthetic-reinforced soil structures. Economic and environmental reasons have led to an increasingly use of locally available residual soils with significant percentage of fines and lower draining capacity, when compared to the traditional good-quality backfill materials. This paper describes an extensive laboratory study carried out using a large-scale direct shear test device, in which the influence of soil moisture content, soil density and geosynthetic type on the direct shear behaviour of the soil-geosynthetic interface was evaluated. The study involved a locally available granite residual soil and four geosynthetics: two geogrids (one uniaxial and the other biaxial), one geocomposite reinforcement (high-strength geotextile) and one geotextile. Test results have revealed that the increase in soil moisture content can measurably reduce the soil-geosynthetic interface shear strength. Regardless of soil moisture content, soil density proved to have a remarkable influence on the interfaces shear strength, particularly when geogrids were used. Among the different geosynthetics tested, the biaxial geogrid was found to be the most effective reinforcement for this particular type of soil, concerning the direct shear mechanism. For soil-geogrid interfaces, the coefficients of interaction ranged from 0.71 to 0.99. For soil-geotextile interfaces, the coefficients of interaction varied from 0.54 to 0.85.

KEYWORDS: Geosynthetics, Soil-geosynthetic interface shear strength, Direct shear tests, Granite residual soil, Soil moisture content, Soil density

1 INTRODUCTION

The interaction between soils and geosynthetics is of utmost importance in many geotechnical engineering applications, particularly in the design and stability analysis of geosynthetic-reinforced soil structures (GRSS). Although factors such as the geometry of a reinforced soil system and its constructive process may affect soil-geosynthetic interaction properties, they are strongly determined by the mobilised interaction mechanism, the physical and mechanical properties of the soil and the mechanical and geometrical properties of the reinforcements. Several experimental methods have been developed for the analysis of soil-geosynthetic interaction, including direct shear tests, pullout tests, in-soil tensile tests and inclined plane tests (e.g., Alfaro *et al*. 1995; Lopes and Ladeira 1996a, 1996b; Raju and Fannin 1998; Pinho-Lopes and Lopes 1999; Costa-Lopes *et al.* 2001; Ramirez and Gourc 2003; Mendes *et al*. 2007; Liu *et al*. 2009; Ferreira *et al.* 2012, 2013, 2014; Vieira *et al*. 2013; Lopes *et al*. 2014). Among them, pullout and direct shear tests are the most commonly used. Whereas the pullout test is a valuable method to investigate the anchorage strength of the reinforcement, the direct shear test is the most suitable test method to simulate soil-geosynthetic interaction in cases where sliding of the soil mass on the reinforcement surface may occur (Lopes 2012; Palmeira 2009).

Although good-quality granular soils are recommended in the design of GRSS, many of these structures have been constructed using on-site native residual soils in areas where good backfill materials are difficult to obtain. The use of locally available soils can lead to significant economic and environmental benefits, but it is important to be aware that the inadequate hydraulic conductivity that often characterises these soils may affect the stability and serviceability of the reinforced structure.

The main concern regarding the internal stability of GRSS constructed using soils with high percentage of fines and poor draining capacity is the behaviour of soil-geosynthetic interfaces when the soil moisture content increases significantly. Several factors such us precipitation, ground water infiltration and seasonal humidity variations may affect the soil moisture content and reduce the shear strength of soil-geosynthetic interfaces, resulting in large deformations or failure (Hatami *et al.* 2012). Some case histories of inadequate performance (serviceability problems or failures) of GRSS constructed with low permeability backfill soils have been reported by several authors (Mitchell and Zornberg 1995; Yoo and Jung 2006; Koerner and Soong 2001). Most of these structures failed after being subjected to heavy rainfalls resulting in the increase of positive pore-water pressures and the reduction in soil matric suction.

Matric suction in the soil is defined as the difference between the pore-air pressure and the pore-water pressure (Fredlund and Rahardjo 1993). The relationship between the soil matric suction and the soil water content is usually expressed through the soil-water characteristic curve. Figure 1 presents typical soil-water characteristic curves for different soil types. The saturated water content and the air-entry value (i.e., matric suction where air starts to enter the largest pores in the soil) generally increase with the plasticity of the soil (Fredlund and Xing 1994). Matric suction contributes to the stability of GRSS increasing the soil stiffness and improving the interface shear strength behaviour (Khoury *et al.* 2011; Portelinha *et al.* 2012; Hatami *et al.* 2013; Riccio *et al*. 2014; Esmaili *et al*. 2014).

Despite the problems arising from the use of poorly draining backfill, some studies have reported excellent performance of GRSS constructed with fine-grained soils reinforced with nonwoven geotextiles (Tatsuoka and Yamauchi 1986; Benjamim *et al*. 2007; Portelinha *et al.* 2013). The hydraulic properties of nonwoven geotextiles can help to dissipate pore-water pressures, contributing to the internal stability of the structure (Ling *et al.* 1993; Tan *et al.* 2001). As the reinforcement layers are able to provide internal drainage, the drainage capacity of the backfill is increased (Portelinha *et al.* 2013).

Over the last decades, numerous experimental studies have been conducted to assess the shear strength parameters of soil-geosynthetic interfaces through direct shear tests (Bergado *et al.* 1993; Bakeer *et al.* 1998; Bergado *et al.* 2006; O´Kelly and Naughton 2008; Liu *et al.* 2009; Anubhav 2010; Ferreira *et al.* 2012, 2013; Vieira *et al*. 2013). However, few studies have compared the direct shear behaviour of different soil-geosynthetic interfaces for various conditions of soil moisture content and density.

Bergado *et al.* (2006) evaluated the shear strength properties of several interfaces involved in the composite liner systems of modern landfills through large-scale direct shear tests performed in dry and wet conditions. For the clay-geomembrane interface, the clay was compacted in the upper shear box at the maximum dry density and optimum moisture content. To simulate the wet condition, the specimens were sheared while submerged in water. For the dry condition, the tests were conducted without submergence in water. The interface peak friction angle in the wet condition was found to be 22% lower than that obtained in the dry condition.

Fleming *et al.* (2006) carried out a series of direct shear tests on non-textured geomembrane-soil interfaces, using a miniature pore pressure transducer embedded in saturated and unsaturated sandy soils. The authors performed a parametric study to investigate the influence of soil moisture content, soil dry density and shearing rate on the interface shear strength parameters. Test results indicated an increase in the interface friction angle with increasing soil density and decreasing moisture content. The influence of the shearing rate was found to be almost negligible.

Abu-Farsakh *et al*. (2007) analysed the effect of soil moisture content and dry density on the direct shear behaviour of cohesive soil-geosynthetic interfaces through large-scale direct shear tests. Soil samples were compacted at their optimum moisture content and at the dry and wet sides of their optimum condition. The authors identified a considerable reduction in the interface strength with the increase in soil moisture content and/or decrease in soil density. However, the degree of reduction was found to be dependent on the soil and geosynthetic types. Based on the obtained results, the authors suggested the use of interface shear parameters of soils at

95% maximum dry density and moisture content 2% above optimum value in the design of GRSS.

Khoury *et al.* (2011) presented results of suction-controlled laboratory direct shear tests on an unsaturated soil-geotextile interface. The interface shear strength was characterised through a 3D failure envelope. The authors observed that the increase in matric suction in the soil-geotextile direct shear tests resulted in an increase in the interface peak shear strength and apparent cohesion. However, the rate of increase in the interface apparent cohesion was found to be non-linear with suction. Furthermore, the increase in matric suction led to a slight reduction in the value of displacement for which the peak shear strength was achieved, and to a more pronounced strain softening behaviour. Test results also showed that the residual shear strength was not significantly affected by the soil matric suction.

Zhang *et al.* (2012) conducted a series of large-scale direct shear tests on soil-geogrid interfaces. Among other factors, the influence of soil moisture content on the interfaces strength properties was investigated. The authors concluded that the increment in the clayey soil moisture content resulted in a significant decrease in the interface cohesion. However, the interface friction angle remained similar for different soil moisture contents.

Esmaili *et al.* (2014) evaluated the strength properties of a clay-geotextile interface through a series of small-scale interface shear tests. The soil specimens were compacted at the optimum moisture content, 2% dry and 2% wet of the optimum moisture content. The authors observed a consistent increase in the interface shear strength with the overburden pressure and the soil matric suction. Based on the results of multi-scale pullout and interface shear tests, the authors developed a moisture reduction factor for the pullout resistance of geotextile reinforcement for the design of reinforced soil structures with marginal soils.

Taking into account the scarcity of studies on the direct shear behaviour of residual soil-geosynthetic interfaces under distinct conditions of soil density and moisture content and involving different geosynthetic types, over 100 large-scale interface direct shear tests were performed using a granite residual soil (typical soil from northern Portugal) and four geosynthetics (two geogrids, one geocomposite reinforcement and one geotextile). To analyse the influence of soil moisture content on the interfaces strength properties, the soil was tested in the air-dried moisture condition and at three different moisture contents. The effect of soil density was evaluated by preparing the soil samples to two distinct values of dry unit weight. The soil shear strength for various conditions of moisture content and density was also characterised from the results of about 20 large-scale direct shear tests. Direct shear test results for the soil-geosynthetic interfaces were compared with the direct shear test results for the soil. In the following sections, the experimental research is described and the obtained results are presented and discussed.

2 EXPERIMENTAL RESEARCH

2.1 Direct shear test apparatus

The large-scale direct shear test apparatus used in this study (Figure 2) was developed at the University of Porto in the scope of previous research (Vieira 2008). The device allows the analysis of the direct shear behaviour of soils, soil-geosynthetic and geosynthetic-geosynthetic interfaces under monotonic and cyclic loading conditions.

The direct shear test apparatus consists of a shear box, divided into upper and lower boxes, a support structure, five hydraulic actuators and respective fluid power unit, an electric cabinet and several internal and external transducers. The inner length, width and thickness of the upper and lower boxes are 600 mm × 300 mm × 150 mm and 800 mm × 340 mm × 100 mm, respectively. The upper box is fixed in the horizontal direction and vertically moveable through hydraulic actuators installed on its edges. The lower box is rigidly fixed to a mobile platform running on low-friction linear guides and its horizontal displacement is controlled by a hydraulic actuator of adjustable pressure. A rigid ring or a rigid base can be inserted in the lower box. When the rigid base is used, the apparatus is able to perform direct shear tests with constant contact area. If the rigid ring is inserted in the lower box, a reduced-contact-area shear box with equally sized (600 mm × 300 mm) upper and lower halves is materialised. The geosynthetic specimens are fixed to the lower box through rigid bars with bolts, positioned outside of the shear area, avoiding any relative displacement between the specimen and the support. The normal stress is applied on the top of the soil placed inside the upper box by a rigid plate with pressure-controlled double acting linear actuators. The value of the vertical force applied on the rigid plate is obtained through a pressure transducer. The shear force applied in the lower box is measured by a load cell and its horizontal movement is recorded by an internal displacement transducer. Vertical and horizontal displacements can also be measured with several external displacement transducers (LVDT). More details about the test apparatus can be found in Vieira *et al*. (2013).

2.2 Test procedures

According to the European Standard EN ISO 12957-1:2005, direct shear tests to characterise soil-geosynthetic interfaces shall be performed by fixing the geosynthetic specimen to a rigid, horizontal support in the lower part of the shear box. However, for geogrids with large apertures (> 15 mm) and a high percentage of openings (> 50% of the overall surface of the specimen) a soil support may alternatively be used. In fact, the mobilisation of the internal soil strength along the geogrid apertures, which contributes for a high percentage of the interface shear strength, is not modeled using the former test procedure (Lopes 2012).

In this research, for the direct shear tests on soil and soil-geogrid interfaces, the rigid ring was inserted in the lower box, which was filled with soil. The direct shear tests on the soil-geocomposite and soil-geotextile interfaces were performed using the rigid base inside the lower box.

For the direct shear tests involving moist soil, the soil samples were initially prepared to the required moisture content. In the case of the direct shear tests on soil and soil-geogrid interfaces, the soil was levelled and compacted inside the lower box in four layers 25 mm thick, using a light compacting hammer. Then, the geosynthetic was fixed to the lower box, outside of the shear area. For the direct shear tests on the other interfaces, the procedures began with the fixation of the geosynthetic specimen. After that, the upper box was positioned over the geosynthetic with a 0.5 mm gap between its base and the specimen surface. Using similar procedures to those described for the lower box, two layers of soil with a compacted thickness of 25 mm were placed inside the upper box. The normal stress was applied and the external displacement transducers were positioned on the rigid plate to record its vertical displacements.

Prior to shearing, the normal stress was applied to the specimens during 1 h. The direct shear tests were conducted at a constant displacement rate of 1mm/min with continuous monitoring of the applied normal stress, displacement of the lower box, shear force mobilised at the interface and vertical displacements of the loading plate.

2.3 Materials

2.3.1 Soil

The soil used in this study was a granite residual soil, which is typically found in northern Portugal and widely used as backfill material for reinforced soil construction. According to the Unified Soil Classification System, this soil can be classified as SW-SM (well-graded sand with silt and gravel). The particle size distribution curve and the main physical properties of the soil are provided in Figure 3 and Table 1, respectively.

2.3.2 Geosynthetics

As previously mentioned, four different geosynthetics were used in this research (Figure 4): a uniaxial extruded geogrid (GGRU), a biaxial woven geogrid (GGRB), a uniaxial geocomposite reinforcement (GCR) and a nonwoven geotextile (GTX).

The uniaxial geogrid (Figure 4a) is manufactured from high density polyethylene (HDPE); the biaxial geogrid (Figure 4b) is composed of high modulus polyester (PET) yarns covered with a protective polymeric coating; the geocomposite reinforcement (Figure 4c) consists of high-modulus polyester yarns, attached to a continuous filament nonwoven geotextile of polypropylene (PP) and the geotextile (Figure 4d) is composed of mechanically bonded continuous filaments of polypropylene. Figure 5 presents the load-strain curves of the different geosynthetics, in the machine direction, obtained from tensile tests performed following the EN ISO 10319:2008. Table 2 lists the main physical and mechanical properties of these materials.

2.4 Test programme

The direct shear tests on soil-geosynthetic interfaces were generally conducted under normal stresses (σ) of 50 kPa, 100 kPa and 150 kPa. For the 100 kPa normal stress, the tests were carried out twice, as recommended by EN ISO 12957-1:2005. For interfaces with higher shear strength, particularly the interfaces between the dense soil (at different moisture conditions) and the biaxial geogrid, it was not possible to perform the test under normal stress of 150 kPa due to limitations of the fluid power unit of the test apparatus. For these interfaces, a lower value was adopted (120 kPa).

In order to analyse the influence of soil moisture content (w) on the interfaces shear behaviour, the soil was tested in its air-dried moisture condition and at three different moisture contents: half of optimum moisture content (0.5 w_{opt}), optimum moisture content (w_{opt}) and optimum plus half of optimum moisture content (1.5 w_{opt}), the latter evaluated only for one of the interfaces (soil-GGRU interface). The effect of soil density was investigated by preparing the soil samples to dry unit weights (γ_d) of 15.31 kN/m^3 and 17.30 kN/m^3.The direct shear behaviour of the soil was also evaluated through large-scale direct shear tests. Similarly to the soil-geosynthetic interface tests, soil samples were tested at dry unit weights of 15.31 kN/m^3 and 17.30 kN/m^3 and at the air-dried (hereinafter referred to as “dry” for simplification), half of optimum and optimum moisture conditions. Direct shear tests on looser soil samples were conducted under normal stresses ranging from 50 kPa to 150 kPa. For denser samples, the applied normal stresses varied from 25 kPa to 100 kPa because of the mentioned limitation of the test apparatus.

3 RESULTS AND DISCUSSION

3.1 Direct shear tests on soil

Figure 6 illustrates the evolution of the shear stress and the vertical displacement of the loading plate center, as function of the shear displacement, along direct shear tests on soil conducted under the normal stress of 100 kPa.

The shear stress versus shear displacement curves for the different conditions of soil density and moisture content (Figure 6a) indicate that, as expected, the soil shear strength and stiffness increased significantly with soil dry density. In addition, denser soil samples mobilised maximum shear strengths at smaller shear displacements, which is consistent with the typical shear stress-strain behaviour of granular materials in dense and loose states. It can also be observed that the maximum strengths decreased progressively with increasing soil moisture content. For instance, the peak shear strength of the denser soil reduced about 10% with the increase in moisture content. Over the range of tested normal stresses, the maximum soil strength reduced up to 25% as a result of the moisture content increase.

Concerning the vertical displacements of the loading plate center, shown in Figure 6b, distinct behaviours can be identified for looser and denser soil samples. When the soil was compacted to $\gamma_d = 17.30$ kN/m^3, the vertical contraction observed at the initial stage of the shear displacement was followed by a dilation phase until the strain softening behaviour was completed. At the end of the test, dry samples exhibited larger dilation than moist samples. When the soil was placed with $\gamma_d = 15.31$ kN/m^3, the vertical contraction was significantly higher, as expected, and increased continuously throughout the test in the case of the moist samples.

Figure 7 presents the peak shear strengths of the soil for the different normal stress values, as well as the corresponding linear best fits. Based on the Mohr-Coulomb failure criterion, the values of the soil internal friction angle (ϕ) and cohesion (c) were derived (Table 3). From Figure 7 it can be concluded that, regardless of the applied normal stress, the soil shear strength increased substantially with soil density. On the other hand, the soil shear strength was found to decrease as the soil moisture content was progressively increased. Thus, the highest shear strengths were obtained for the dry soil in the denser state, which may be characterised by $\phi = 46.6°$ and c = 29.5 kPa (Table 3). The results provided in Table 3 also evidence a relevant increment in the soil cohesion as a result of the soil density increase. On the other hand, while the soil internal friction angle was not significantly affected by the moisture content increase, the soil cohesion decreased considerably, which is in agreement with other research studies on the shear strength properties of unsaturated soils (Vanapalli *et al.* 1996; Lu and Likos 2006).

3.2 Soil-geosynthetic direct shear tests

3.2.1 Influence of soil moisture content

The shear stress-shear displacement behaviour of the soil-GGRU interface under different soil moisture conditions (dry, $w = 0.5\ w_{opt}$, $w = w_{opt}$ and $w = 1.5\ w_{opt}$) is illustrated in Figure 8 for two values of soil dry unit weight: $\gamma_d = 15.31$ kN/m^3 (Figure 8a) and $\gamma_d = 17.30$ kN/m^3 (Figure 8b). It is possible to observe that, regardless of the soil density, the interface shear strength tended to decrease with increasing soil moisture content. Results obtained with dry soil and with soil at $w = 0.5\ w_{opt}$ indicated that for some test conditions (i.e., normal stress and soil density), the increase in moisture content between the mentioned values caused a relevant drop in the interface strength (by 20%); however, for other test conditions, no significant influence was observed. When the soil at $w = w_{opt}$ was tested, the interface shear strength decreased considerably (up to 13%) in relation to the corresponding values at $w = 0.5\ w_{opt}$. Comparing the results obtained for $w = w_{opt}$ and $w = 1.5\ w_{opt}$, an important reduction (up to 27%) of shear strength with increasing moisture content can be identified, particularly in the case of the denser soil (Figure 8b). The reduction in shear strength with increasing moisture content observed for unsaturated soil-geosynthetic interfaces may be attributed to the development of positive pore-water pressures and the loss of soil matric suction (Khoury *et al.* 2011; Hatami *et al.* 2013; Hatami *et al.* 2014; Esmaili *et al.* 2014).

The influence of soil moisture content on the shear strength of the soil-GGRB, soil-GCR and soil-GTX interfaces can be examined from the results presented in Figures 9-11, respectively. Similarly to what was observed for the soil-GGRU interface, the interfaces shear strength tendentially decreased with increasing moisture content. The increase in moisture content from the dry condition to $w = 0.5\ w_{opt}$ caused a maximum drop in the interfaces strength of 17%. In turn, when the soil moisture content varied from $w = 0.5\ w_{opt}$ to $w = w_{opt}$, the interfaces shear strength decreased up to 22%. However, it should be noted that the degree of reduction in the interfaces strength with the moisture content increase was dependent on the remaining test conditions. This finding is in agreement with the results reported by Abu-Farsakh *et al.* (2007) concerning the influence of normal stress and geosynthetic type on the degree of reduction in the soil-geosynthetic interface shear strength with increasing soil moisture content. Furthermore, results of direct shear tests performed with denser soil showed well-defined peak shear strengths for the interfaces involving dry soil, while for the interfaces with moist soils the strain softening was generally less pronounced, indicating a more ductile behaviour.

The results shown in Figures 8-11 support the idea that the compaction of the backfill material at the dry side of the optimum moisture content leads generally to a considerable improvement in the soil-geosynthetic interface shear strength, in comparison to that achieved when the optimum value is adopted. However, it is important to highlight that this observation is based on the assumption that the soil dry density is maintained, which for moisture contents below the optimum value implies the use of a greater compactive effort.

Figure 12 compares the evolution of the vertical displacement of the loading plate center along soil-geosynthetic direct shear tests conducted with different soil moisture contents, under a normal stress of 100 kPa. Results for different interfaces are plotted in different graphs, each one including the displacements recorded for both values of soil dry unit weight. In the case of the looser soil samples, the vertical contraction tended to increase with soil moisture content, regardless of geosynthetic type. Indeed, the vertical deformation of the dry soil was consistently lower than that of the moist soils. Similar trend was also observed in the direct shear tests of the soil alone (Figure 6b). This evidence is justified by the fact that the presence of water in the soil facilitates the rearrangement of the soil particles during shearing, owing to the increased lubrication, and causes the weakening of the particle lumps, resulting in a higher compressibility of the soil. With respect to the vertical deformation of the denser soil samples under different moisture conditions, the results presented in Figure 12 indicate that the dry soil tended to exhibit a more pronounced expansive behaviour in comparison to that of the moist soils. In general, soil dilation at the end of the tests decreased with the moisture content increase. When the wet soil (w = 1.5 w_{opt}) was tested (Figure 12a), no dilation was observed even for the denser condition (γ_d = 17.30 kN/m^3).

3.2.2 Influence of soil density

Figure 13 shows the effect of soil density on the shear stress-shear displacement behaviour of the soil-GGRU interface for different soil moisture conditions: dry (Figure 13a), $w = 0.5\ w_{opt}$ (Figure 13b), $w = w_{opt}$ (Figure 13c) and $w = 1.5\ w_{opt}$ (Figure 13d). As expected, the initial shear stiffness and the maximum shear strength of the interface increased substantially with soil dry unit weight, regardless of the moisture content and normal stress. Furthermore, when the soil was tested at the denser state, the maximum shear strength of the interface was mobilised for smaller shear displacements, in comparison to those corresponding to the looser soil.

The influence of soil density on the vertical deformation of the GGRU-reinforced soil subjected to normal stresses between 50 kPa and 150 kPa is shown in Figure 14 for different soil moisture conditions. The results of the direct shear tests performed with denser soil samples reveal an initial contractile behaviour which was generally followed by a significant dilation, with the exception of the wet soil (Figure 14d). For the remaining moisture contents, soil dilation was found to be more pronounced at lower normal stress values (Figure 14a, 14b, 14c). Similar trend was also reported by Khoury *et al.* (2011) based on the results of suction-controlled direct shear tests on unsaturated soil-geotextile interfaces. On the other hand, in the tests performed with looser soil samples, soil settlement consistently increased as the normal stress was progressively increased, although for the wet soil this increase has been of little significance (Figure 14d). For the soil-GGRB interface, similar conclusions concerning the effect of soil density on the interface behaviour during shearing were drawn.

The soil-GCR interface shear strength for different soil densities is presented in Figure 15 for the dry condition (Figure 15a) and for the soil half of optimum and optimum moisture contents (Figure 15b and 15c, respectively). As previously observed for the soil-GGRU interface, soil density is positively correlated with the soil-GCR interface maximum shear strength, regardless of soil moisture content. However, the influence of soil density on the soil-GCR interface shear strength was not as significant as that observed when the geogrid was used, which may be associated with the different interaction mechanisms mobilised at the interfaces during the shear process. According to the results presented in Table 3, when the soil was tested in the denser state, the shear strength parameters (in particular, the cohesion) increased significantly. In the case of the geogrid interface, the internal soil strength could be mobilised in the geogrid apertures, contributing for a relevant increment in the interface strength when the denser soil was used, due to the greater grain interlocking and soil cohesion. In the case of the geocomposite interface, the increase in soil density did not induce such a significant increase in shear strength since the only interaction mechanism mobilised during shearing was the skin friction along the reinforcement.

Figure 16 compares the deformative behaviour of the soil along the soil-GCR direct shear tests for different values of dry unit weight. As expected, the vertical contraction exhibited by the looser soil was substantially higher than that of the denser soil. It can also be observed that the contraction of the looser soil samples generally increased with the applied normal stress, which is consistent with the results presented for the soil-GGRU interface (Figure 14). Yet, the dilatancy of the denser soil was found to be relatively insignificant as compared to that observed for the GGRU-reinforced soil, particularly when the moist samples were tested (Figure 16b, 16c). Similar response was also observed along the direct shear tests on the soil-GTX interface with regard to the expansive behaviour of the denser soil samples (Figure 12d). The fact that the soil dilation along the direct shear tests on the soil-GCR and soil-GTX interfaces is lower than that along the tests on the soil-geogrid interfaces may be partly justified by the use of the rigid support for the geotextiles, which may restrain soil dilatancy to some extent.

To analyse the influence of the type of support on the direct shear behaviour of the soil-GTX interface, additional direct shear tests were carried out with the geotextile specimens placed over a soil support. The soil was tested at the air-dried moisture content and at different values of dry unit weight. Test results demonstrated that, when the soil was tested in the looser condition (γ_d = 15.31 kN/m^3), the influence of the type of support on the interface shear strength was almost negligible. However, soil compression was more pronounced when the soil support was used, as expected. Regarding the tests performed with denser soil samples (γ_d = 17.30 kN/m^3), the use of soil in the lower box resulted in a small increase in the interface peak shear strength (by 6%) and soil dilation. However, more general conclusions about the influence of the type of support on the direct shear behaviour of soil-geotextile interfaces require further investigation.

3.2.3 Influence of geosynthetic type

Figure 17 presents the peak strength envelopes from the soil-geosynthetic direct shear tests conducted with looser soil samples under different moisture conditions: dry (Figure 17a), $w = 0.5\ w_{opt}$ (Figure 17b) and $w = w_{opt}$ (Figure 17c). In turn, the peak strength envelopes for the interfaces with denser soil are plotted in Figure 18. For comparison purposes, the peak strength envelopes of the soil for identical test conditions (i.e., moisture content and dry density) are also superimposed in Figures 17 and 18. The values of the soil-geosynthetic interface shear strength parameters, namely interface friction angle (δ) and apparent cohesion (c_a), are provided in Table 4.

As shown in Figure 17, the geogrid interfaces were generally more effective than the interfaces with the geotextiles regarding mobilisation of peak shear strength. This conclusion became even more evident when the denser soil was used (Figure 18). In addition, between the two geogrids tested, the biaxial geogrid was found to be the most efficient reinforcement for this particular type of soil, regarding the direct shear mechanism (Figures 17 and 18). This may be attributed to the fact that the biaxial geogrid has higher percent open area than the uniaxial geogrid (Table 2), in which the internal shear strength of the soil is mobilised. It is widely accepted that, when the reinforcement is a geogrid, the development of the internal soil strength in the geogrid apertures contributes for a high percentage of the overall interface strength in direct shear mode.

It should also be pointed out that, for most test conditions, soil-geosynthetic interface peak shear strength was considerably lower than that of the soil alone (Figures 17 and 18), which suggests that soil-geosynthetic interfaces are potential sliding surfaces when the direct shear mechanism is of concern. The relationship between the peak shear strength of the soil and the peak shear strength of the different soil-geosynthetic interfaces under identical test conditions is quantitatively evaluated in the following section through the coefficient of interaction.

3.3 Coefficient of interaction

The coefficient of interaction or friction ratio (EN ISO 12957-1:2005), c_i, is defined as the ratio of the maximum shear stress in a soil-geosynthetic direct shear test, $\tau_{soil/geo}^{max}\ (\sigma)$, to the maximum shear stress in a direct shear test on soil, $\tau_{soil}^{max}\ (\sigma)$, under the same normal stress, σ:

$$c_i = \frac{\tau_{soil/geo}^{max}\ (\sigma)}{\tau_{soil}^{max}\ (\sigma)} = \frac{c_a + \sigma \tan\delta}{c + \sigma \tan\phi} \qquad (1)$$

In this study, the values of the coefficient of interaction for the different soil-geosynthetic interfaces were determined from Equation (1), using the results from the soil-geosynthetic direct shear tests and those obtained from the direct shear tests on soil under the same conditions of moisture content and dry density.

Table 5 summarises the values of the coefficient of interaction (also called by other authors "interface shear strength coefficient" or "interface efficiency") of the different interfaces for normal stresses of 50 kPa and 100 kPa. For the interfaces with geogrids, the coefficients of interaction range from 0.71 to 0.99 (0.71-0.90 for the soil-GGRU interface and 0.80-0.99 for the soil-GGRB interface). These values are generally consistent with those reported by other researchers for soil-geogrid interfaces. Cazzuffi *et al.* (1993) reported coefficients of interaction of about 0.83-1.04 for different soil-HDPE geogrid interfaces, while Liu *et al.* (2009) presented values of interface shear strength coefficient ranging from 0.89 to 1.01 for a variety of soil/PET-yarn geogrid interfaces. However, in contrast to what was observed in the aforementioned studies, any coefficient of interaction determined in this study was greater than unity, indicating that the soil-geogrid interface strength was consistently lower than the internal shear strength of the soil under identical test conditions.

In general, the values of the coefficient of interaction obtained for the soil-GTX and soil-GCR interfaces were lower than those corresponding to the geogrid interfaces. For the soil-GCR interface, the coefficients of interaction for the different test conditions range from 0.54 to 0.81. In the case of the soil-GTX interface, the values are comprised between 0.57 and 0.85 (Table 5). This finding is in agreement with the results of the research study by Liu *et al.* (2009), in which the interface shear strength coefficients obtained for the soil-geotextile interface were lower than those achieved for the interfaces with geogrids. Similarly, Abu-Farsakh *et al.* (2007) reported values of interface efficiency of about 0.64-0.81 for a soil-geotextile interface and higher values (0.66-1.05) for PET and PP geogrid interfaces at the optimum compaction condition. The reason why higher coefficients of interaction are obtained for interfaces with geogrids is usually attributed to the relevance of the mobilisation of internal soil strength in the geogrid apertures. However, some authors consider that the bearing resistance provided by the geogrid apertures (Bergado *et al.* 1993) or transverse ribs (Liu *et al.* 2009) may also be able to contribute to the overall shear strength of the soil-geogrid interface in direct shear mode.

As shown in Table 5, the coefficient of interaction for the soil-GCR and soil-GTX interfaces is negatively correlated with the soil dry unit weight. This indicates that, when the soil dry unit weight increased from 15.31 kN/m^3 to 17.30 kN/m^3, the percentage increase in the internal soil strength exceeded the percentage increase in the soil-GCR and soil-GTX interfaces shear strength. This finding is comparable to the previous observation that the increment in the soil-GGRU interface shear strength due to the soil density increase was more significant than that for the soil-GCR interface (section 3.2.2). These observations support the idea that soil density does not induce such a significant increase in the interface shear strength when the only interaction mechanism developed during shearing is the skin friction along the reinforcement (in comparison to the cases where the internal soil strength is mobilised).

Regarding the soil-geogrid interfaces, the influence of soil density on the coefficient of interaction was found to be dependent on the soil moisture content and applied normal stress: for the normal stress of 100 kPa, the coefficient of interaction consistently increased with soil density; for the 50 kPa normal stress, the coefficient of interaction increased with soil density for the dry soil but decreased for the moist soils.

The results presented in Table 5 suggest that the influence of soil moisture content on the coefficient of interaction of the soil-geosynthetic interface is dependent on the type of geosynthetic used. For instance, while for the soil-GGRB interface higher coefficients of interaction were generally obtained for the soil optimum moisture content, for the soil-GCR interface the values were generally higher for the dry soil. However, it is worth noting that the coefficient of interaction as determined in this study is a measure of the soil-geosynthetic interface efficiency as compared to the soil shear strength (when subjected to the same conditions of moisture content and dry density) and should not be directly associated with the interface shear strength. In other words, an interface with higher coefficient of interaction is not necessarily an interface with higher strength but it is an interface at which the mobilised shear strength is closer to the shear strength of the soil under identical test conditions.

4. CONCLUSIONS

This paper presents an extensive direct shear testing programme involving four geosynthetics and a locally available granite residual soil, which was tested under different conditions of moisture content and dry density. Some of the conclusions of this study are summarised below.

- Direct shear tests on soil demonstrated that the placement dry density has a marked influence on the internal soil strength, regardless of moisture content. The soil internal friction angle and, particularly, the soil cohesion increased with soil density.

- The internal soil strength decreased progressively with increasing moisture content. From the air-dried condition to the optimum moisture content, the soil shear strength reduced up to 25%. The soil internal friction angle remained similar for the different moisture contents evaluated. However, the soil cohesion decreased considerably with the soil moisture content increase.

- Soil density is positively correlated with the soil-geosynthetic interface shear strength. However, the increase in the interfaces strength with respect to soil density was more pronounced for the soil-geogrid interfaces, in comparison to that for the geotextile interfaces. When the only interaction mechanism mobilised during shearing is the skin friction along the reinforcement, the influence of soil density on the soil-geosynthetic interface shear strength is not as significant as when the internal soil strength is mobilised (i.e., when geogrids are used).

- The increase in soil moisture content can measurably reduce the soil-geosynthetic interface shear strength. Results obtained using dry soil and moist soil at $w = 0.5\ w_{opt}$ showed a maximum drop in the interfaces shear strength of 20%. When the moisture content increased from $w = 0.5\ w_{opt}$ to $w = w_{opt}$, the interfaces shear strength decreased up to 22%. In turn, when the moisture content increased from $w = w_{opt}$ to w = 1.5 w_{opt}, the reduction in the soil-GGRU interface shear strength reached 27%. However, the level of influence of the soil moisture content on the soil-geosynthetic interfaces shear strength was found to depend on the remaining test conditions (i.e., geosynthetic type, soil density and normal stress).

- In the direct shear tests with looser soil samples, the vertical contraction of the soil tended to increase with applied normal stress and soil moisture content. In the direct shear tests with denser soil samples, the soil dilation tended to increase with decreasing normal stress and moisture content.

- The geogrid interfaces were generally more efficient than the interfaces with the geotextiles with respect to the mobilisation of direct shear strength. Among the different geosynthetics tested, the biaxial geogrid was found to be the most effective reinforcement for this particular type of soil, concerning the direct shear mechanism.

- Regardless of the test conditions, the soil-geosynthetic interface shear strength was consistently lower than the internal shear strength of comparable soil. For soil-geogrid interfaces, the coefficients of interaction ranged from 0.71 to 0.99. For soil-geotextile interfaces, the coefficients of interaction varied from 0.54 to 0.85.

ACKNOWLEDGMENTS

The authors gratefully acknowledge the financial support from the Portuguese Foundation for Science and Technology (FCT) under Grant SFRH/BD/72886/2010 and Research Projects FCOMP–01–0124–FEDER-009750-PTDC/ECM/100975/2008 and FCOMP-01-0124-FEDER-028842-PTDC/ECM-GEO/0622/2012.

This manuscript is the accepted version of the paper:

Direct shear behaviour of residual soil-geosynthetic interfaces - influence of soil moisture content, soil density and geosynthetic type, Geosynthetics International, Vol. 22, Issue 3, pp. 257-272, https://doi.org/10.1680/gein.15.00011

NOTATION

Basic SI units are given in parentheses.

c – soil cohesion (Pa)

c_a – soil-geosynthetic interface apparent cohesion (Pa)

C_c – soil curvature coefficient (dimensionless)

c_i – coefficient of interaction (dimensionless)

C_u – soil uniformity coefficient (dimensionless)

D_{10} – diameter corresponding to 10% passing of soil (m)

D_{30} – diameter corresponding to 30% passing of soil (m)

D_{50} – diameter corresponding to 50% passing of soil (m)

e_{max} – maximum void ratio of soil (dimensionless)

e_{min} – minimum void ratio of soil (dimensionless)

G – specific gravity of soil particles (dimensionless)

w – soil moisture content (dimensionless)

w_{opt} – soil optimum moisture content from Modified Proctor test (dimensionless)

γ_d – soil dry unit weight (N/m^3)

γ_{dmax} – soil maximum dry unit weight from Modified Proctor Test (N/m^3)

δ – soil-geosynthetic interface friction angle (degrees)

σ – normal stress (Pa)

τ – shear stress (Pa)

$\tau_{soil}^{max}(\sigma)$ – maximum shear stress in a direct shear tests on soil (Pa)

$\tau_{soil/geo}^{max}(\sigma)$ – maximum shear stress in a soil-geosynthetic direct shear test (Pa)

ϕ – soil internal friction angle (degrees)

ABBREVIATIONS

GCR – geocomposite reinforcement

GGRB – biaxial geogrid

GGRU – uniaxial geogrid

GRSS – geosynthetic-reinforced soil structure

GTX – geotextile

HDPE – high density polyethylene

LVDT – linear variable displacement transducer

PET – polyester

PP – polypropylene

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This manuscript is the accepted version of the paper:

Direct shear behaviour of residual soil-geosynthetic interfaces - influence of soil moisture content, soil density and geosynthetic type, Geosynthetics International, Vol. 22, Issue 3, pp. 257-272, https://doi.org/10.1680/gein.15.00011

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TABLES

Table 1. Soil physical properties.

Property	Value
D_{10} (mm)	0.09
D_{30} (mm)	0.35
D_{50} (mm)	1.00
C_U	16.90
C_C	1.00
G	2.73
e_{min}	0.476
e_{max}	0.998
γ_{dmax} (kN/m^3)[1]	18.93
w_{opt} (%)[1]	11.45

[1] Obtained from the Modified Proctor Test (as per BS 1377-4:1990).

Table 2. Physical and mechanical properties of the geosynthetics.

Property	Geosynthetics			
	GGRU	GGRB	GCR	GTX
Raw material	HDPE	PET	PET/PP	PP
Mass per unit area (g/m^2)	450	380	310	1000
Thickness – 2 kPa (mm)	-	-	2.3	7.2
Thickness of longitudinal ribs (mm)	1.1	1.6	-	-
Thickness of transverse ribs (mm)	2.7	1.6	-	-
Mean grid size (mm)	22×235	25×25	-	-
Percent open area (%)	59	68	-	-
Short term tensile strength[1] (kN/m)	68	58	58	55
Strain at maximum load[1] (%)	11.0	10.5	11.5	105.0
Short term tensile strength[2] (kN/m)	52.2	43.9	54.6	69.5
Strain at maximum load[2] (%)	12.4	7.9	10.6	100.9
Secant stiffness at 5% strain[2] (kN/m)	509.8	401.6	600.9	156.3

[1] Provided by the manufacturer (machine direction).

[2] Obtained from tensile tests in the machine direction (as per EN ISO 10319:2008).

Table 3. Soil shear strength parameters.

Soil condition		ϕ (°)	c (kPa)
w	γ_d (kN/m^3)		
Dry	15.31	44.7	7.8
Dry	17.30	46.6	29.5
0.5 w_{opt}	15.31	44.3	1.4
0.5 w_{opt}	17.30	46.9	21.7
w_{opt}	15.31	42.6	0.0
w_{opt}	17.30	46.6	15.8

Table 4. Soil-geosynthetic interface shear strength parameters.

Soil condition		Soil-GGRU interface		Soil-GGRB interface		Soil-GCR interface		Soil-GTX interface	
w	γ_d (kN/m^3)	δ (°)	c_a (kPa)	δ (°)	c_a (kPa)	δ (°)	c_a (kPa)	δ (°)	c_a (kPa)
Dry	15.31	38.1	2.6	42.9	0.0	38.0	5.8	34.4	5.0
Dry	17.30	37.6	33.0	42.1	31.4	38.5	14.1	42.4	0.0
0.5 w_{opt}	15.31	32.3	12.0	38.2	2.8	35.1	2.6	33.2	10.1
0.5 w_{opt}	17.30	42.8	12.6	45.8	14.4	33.0	17.4	37.4	12.0
w_{opt}	15.31	32.7	7.5	34.5	11.4	32.5	3.7	32.9	1.6
w_{opt}	17.30	37.5	14.8	45.6	8.8	36.1	1.5	31.7	12.0

Table 5. Coefficients of interaction of soil-geosynthetic interfaces.

Soil condition		Soil-GGRU interface		Soil-GGRB interface		Soil-GCR interface		Soil-GTX interface	
w	γ_d (kN/m^3)	Normal stress (kPa)							
		50	100	50	100	50	100	50	100
Dry	15.31	0.78	0.74	0.81	0.86	0.80	0.78	0.70	0.68
Dry	17.30	0.86	0.82	0.93	0.90	0.68	0.68	0.57	0.66
0.5 w_{opt}	15.31	0.86	0.77	0.87	0.80	0.76	0.73	0.85	0.77
0.5 w_{opt}	17.30	0.71	0.85	0.83	0.92	0.61	0.66	0.64	0.69
w_{opt}	15.31	0.90	0.77	0.99	0.88	0.81	0.73	0.77	0.72
w_{opt}	17.30	0.74	0.78	0.84	0.93	0.54	0.62	0.62	0.61

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FIGURES

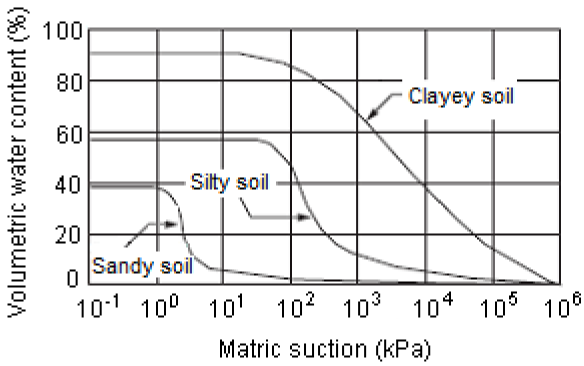

Figure 1. Typical soil-water characteristic curves for different soil types (modified from Fredlund and Xing 1994).

Figure 2. Direct shear test apparatus.

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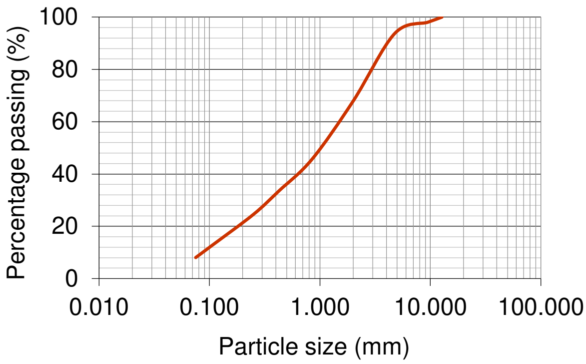
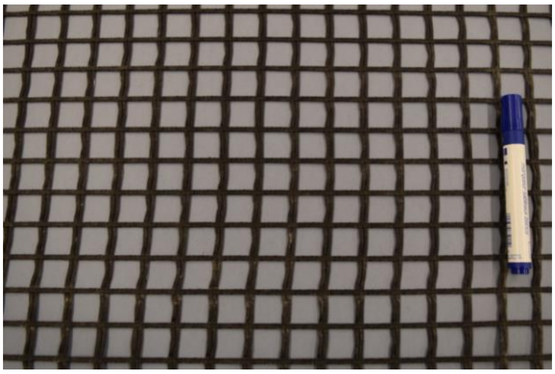
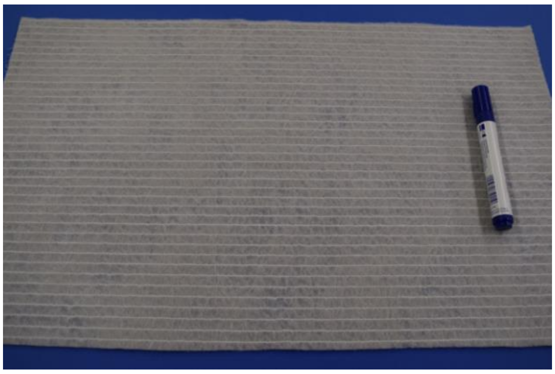


Figure 3. Soil particle size distribution curve.

(a)



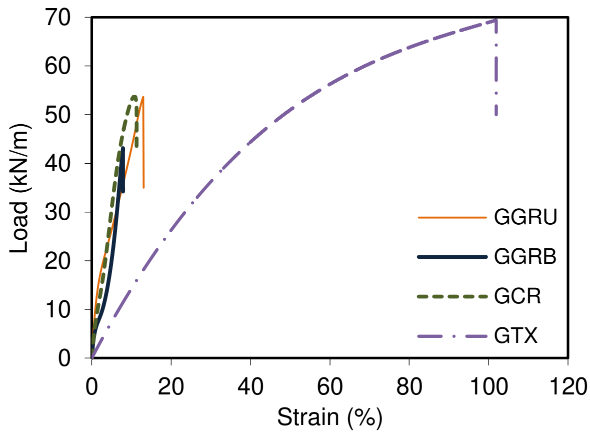
(b)



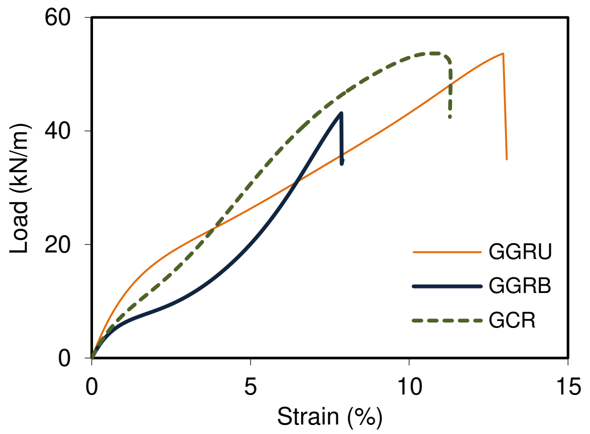
(c)

(d)

Figure 4. Geosynthetics used: a) GGRU; b) GGRB; c) GCR; d) GTX.

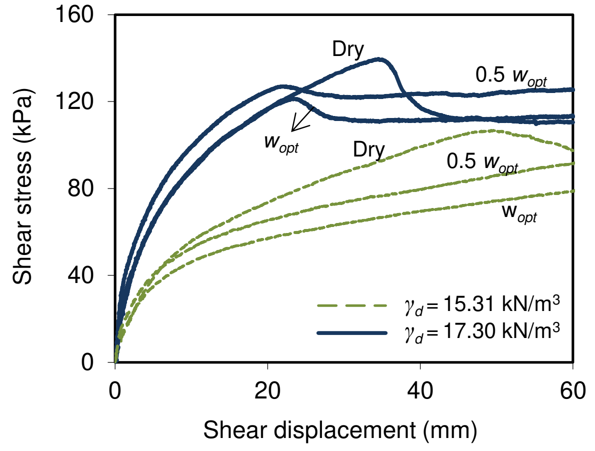


(a)

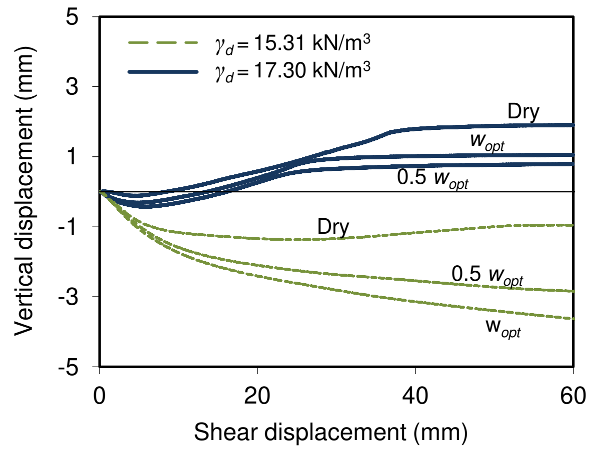


(b)

Figure 5. Load-strain curves of the geosynthetics in the machine direction: a) GGRU, GGRB, GCR and GTX; b) detail for GGRU, GGRB and GCR.



(a)



(b)

Figure 6. Results of direct shear tests on soil for σ = 100 kPa: a) shear stress vs shear displacement; b) vertical displacement of the loading plate center vs shear displacement.

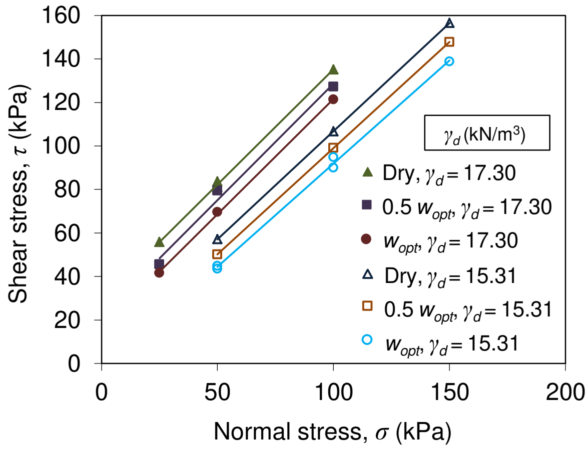
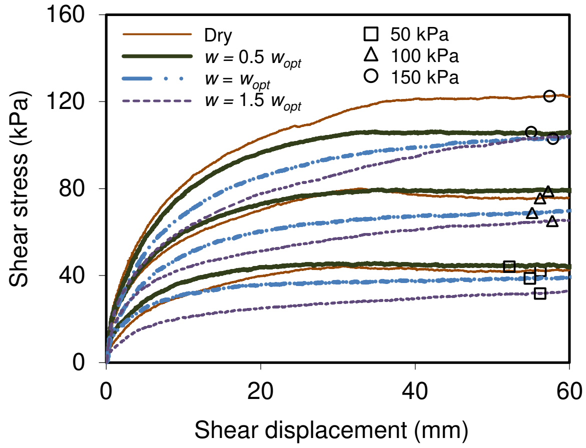
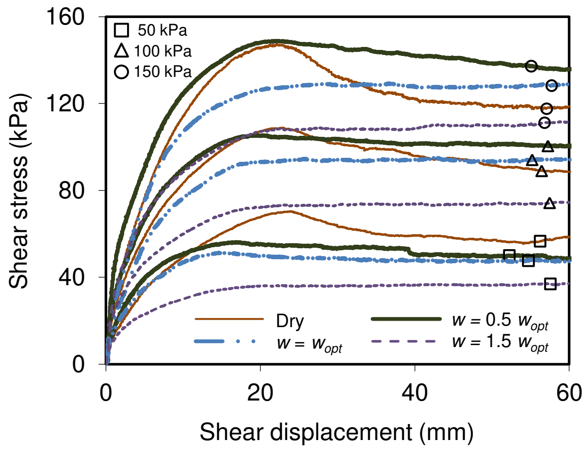


Figure 7. Peak strength envelopes of the soil.

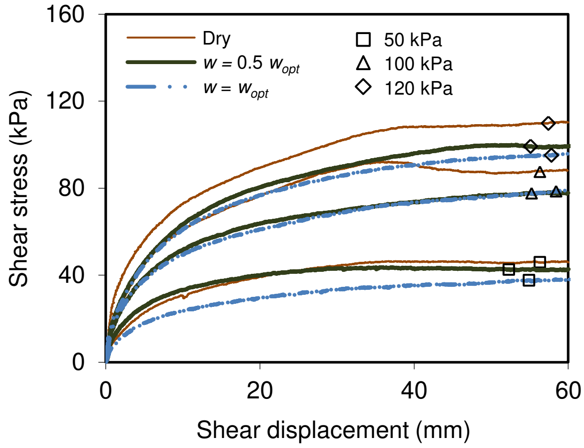


(a)

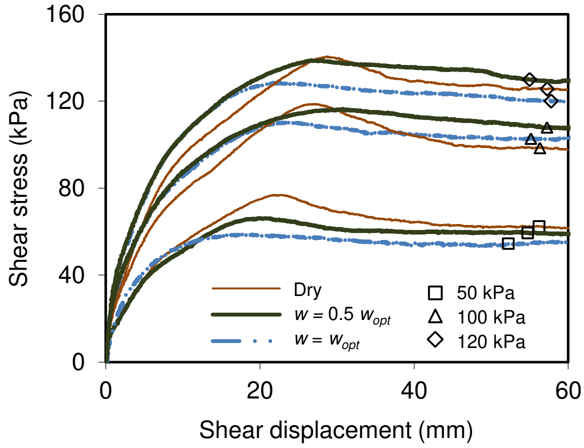


(b)

Figure 8. Influence of soil moisture content on the shear strength of soil-GGRU interface: a) $\gamma_d = 15.31$ kN/m^3; b) $\gamma_d = 17.30$ kN/m^3.

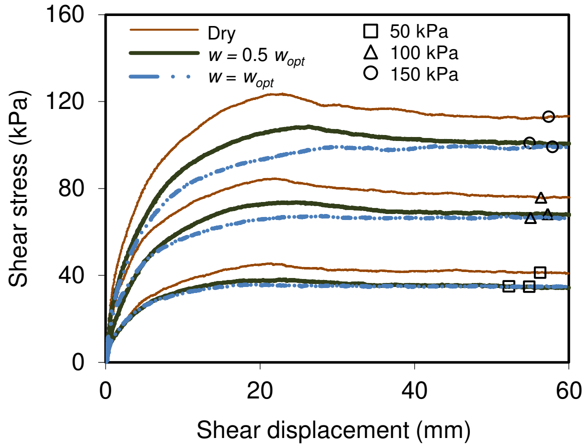


(a)

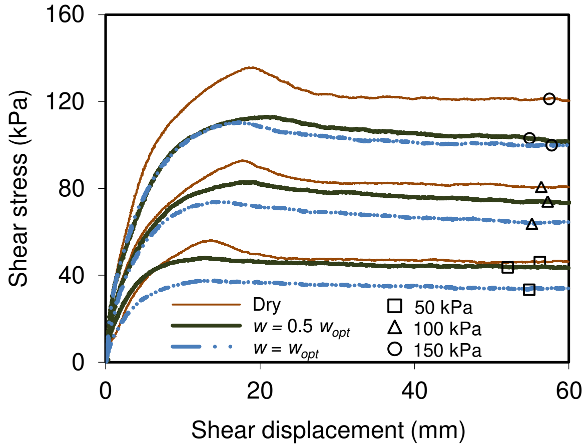


(b)

Figure 9. Influence of soil moisture content on the shear strength of soil-GGRB interface: a) $\gamma_d = 15.31$ kN/m^3; b) $\gamma_d = 17.30$ kN/m^3.

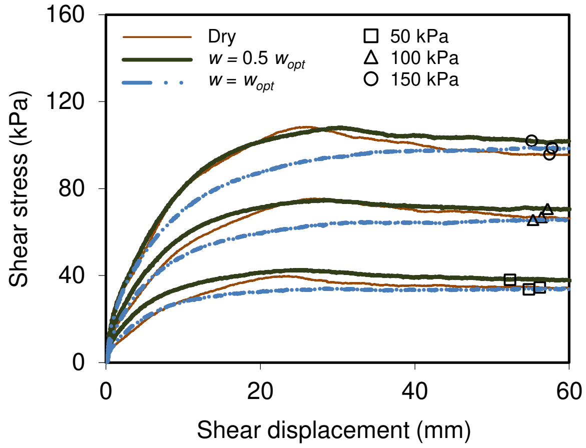


(a)

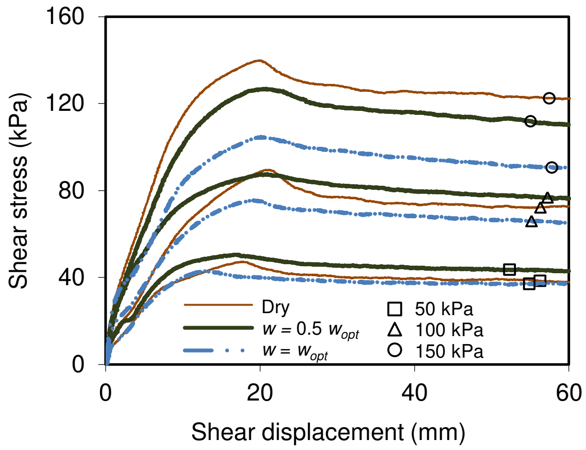


(b)

Figure 10. Influence of soil moisture content on the shear strength of soil-GCR interface: a) $\gamma_d = 15.31$ kN/m^3; b) $\gamma_d = 17.30$ kN/m^3.

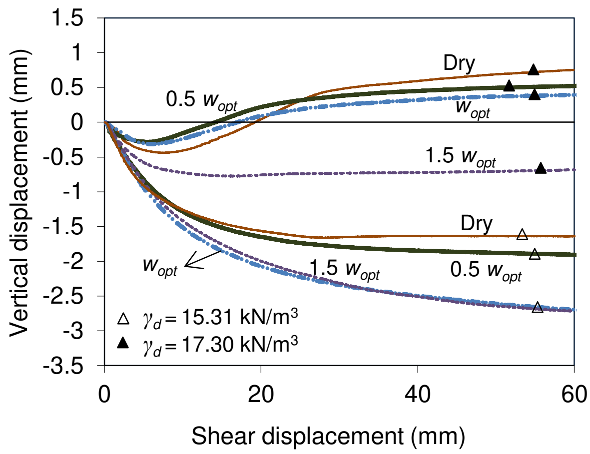


(a)

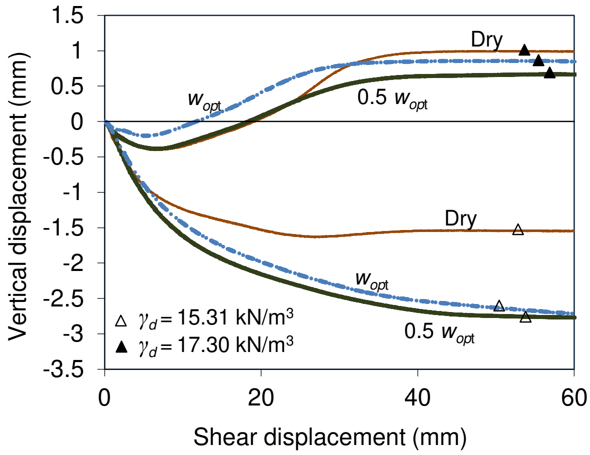


(b)

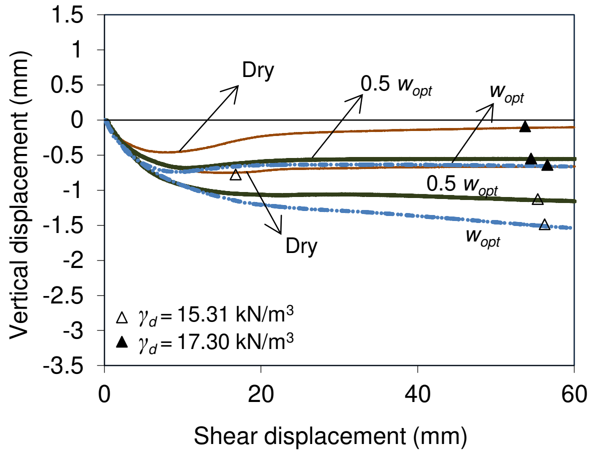
Figure 11. Influence of soil moisture content on the shear strength of soil-GTX interface: a) $\gamma_d = 15.31$ kN/m^3; b) $\gamma_d = 17.30$ kN/m^3.


Vertical displacement (mm)
Shear displacement (mm)
Dry
0.5 w_{opt}
w_{opt}
1.5 w_{opt}
Dry
w_{opt}
1.5 w_{opt}
0.5 w_{opt}
$\gamma_d = 15.31$ kN/m³
$\gamma_d = 17.30$ kN/m³

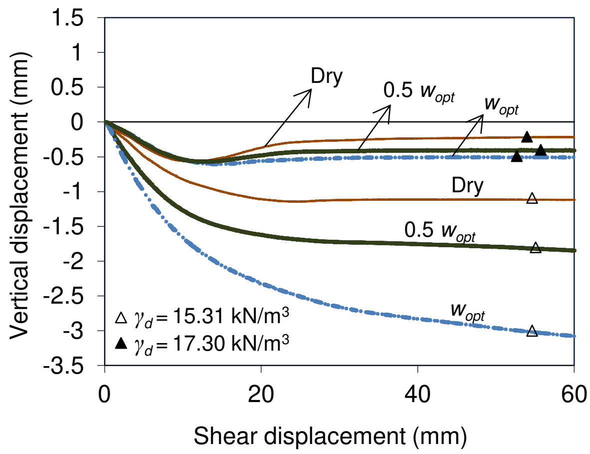
(a)


Vertical displacement (mm)
Shear displacement (mm)
Dry
w_{opt}
0.5 w_{opt}
Dry
w_{opt}
0.5 w_{opt}
$\gamma_d = 15.31$ kN/m³
$\gamma_d = 17.30$ kN/m³

(b)

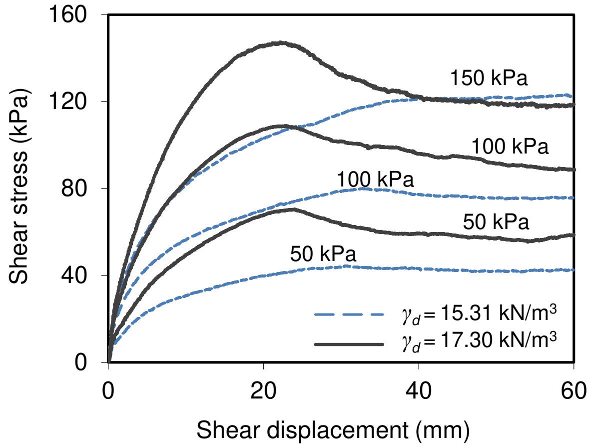

Vertical displacement (mm)
Shear displacement (mm)
Dry
0.5 w_{opt}
w_{opt}
0.5 w_{opt}
Dry
w_{opt}
$\gamma_d = 15.31$ kN/m³
$\gamma_d = 17.30$ kN/m³

(c)

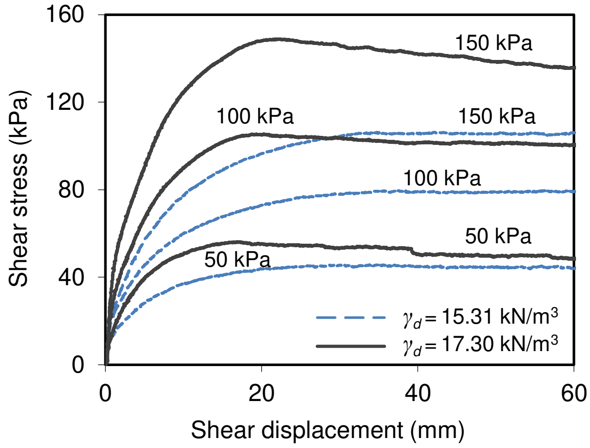


(d)

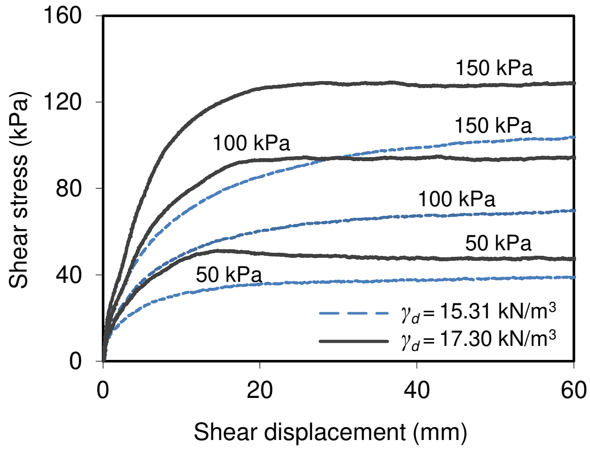
Figure 12. Influence of soil moisture content on the vertical displacement of the loading plate center for σ = 100 kPa: a) soil-GGRU; b) soil-GGRB; c) soil-GCR; d) soil-GTX.


160
120
80
40
0
Shear stress (kPa)
0
20
40
60
Shear displacement (mm)
150 kPa
100 kPa
100 kPa
50 kPa
50 kPa
γ_d = 15.31 kN/m³
γ_d = 17.30 kN/m³

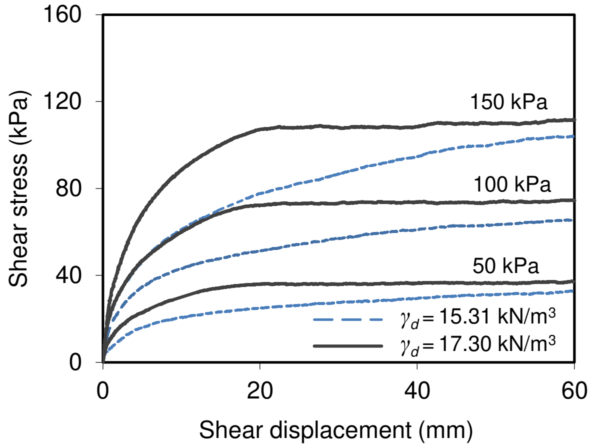
(a)


160
120
80
40
0
Shear stress (kPa)
0
20
40
60
Shear displacement (mm)
150 kPa
100 kPa
150 kPa
100 kPa
50 kPa
50 kPa
γ_d = 15.31 kN/m³
γ_d = 17.30 kN/m³

(b)

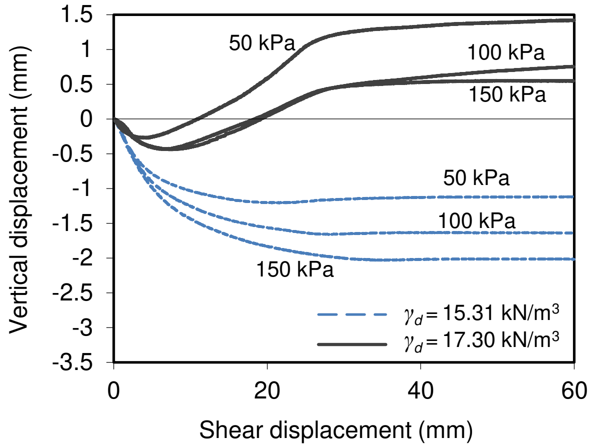

160
120
80
40
0
Shear stress (kPa)
0
20
40
60
Shear displacement (mm)
150 kPa
150 kPa
100 kPa
100 kPa
50 kPa
50 kPa
γ_d = 15.31 kN/m³
γ_d = 17.30 kN/m³

(c)

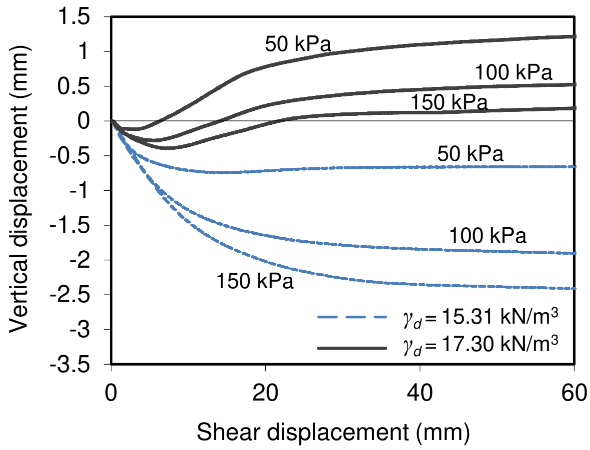


(d)

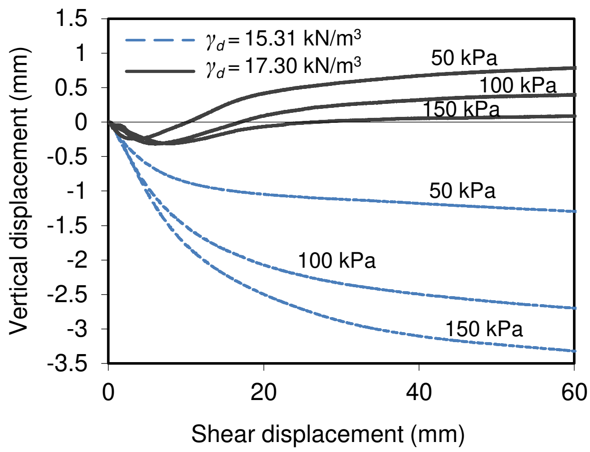
Figure 13. Influence of soil density on the shear strength of soil-GGRU interface: a) dry soil; b) $w = 0.5\ w_{opt}$; c) $w = w_{opt}$; d) $w = 1.5\ w_{opt}$.


50 kPa
100 kPa
150 kPa
50 kPa
100 kPa
150 kPa
γ_d = 15.31 kN/m³
γ_d = 17.30 kN/m³
Vertical displacement (mm)
Shear displacement (mm)

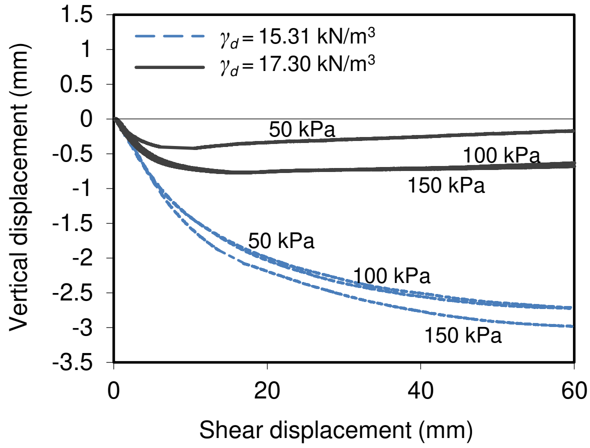
(a)


50 kPa
100 kPa
150 kPa
50 kPa
100 kPa
150 kPa
γ_d = 15.31 kN/m³
γ_d = 17.30 kN/m³
Vertical displacement (mm)
Shear displacement (mm)

(b)


γ_d = 15.31 kN/m³
γ_d = 17.30 kN/m³
50 kPa
100 kPa
150 kPa
50 kPa
100 kPa
150 kPa
Vertical displacement (mm)
Shear displacement (mm)

(c)



(d)

Figure 14. Influence of soil density on the vertical displacement of the loading plate center for soil-GGRU interface: a) dry soil; b) w = 0.5 w_{opt}; c) w = w_{opt}; d) w = 1.5 w_{opt}.

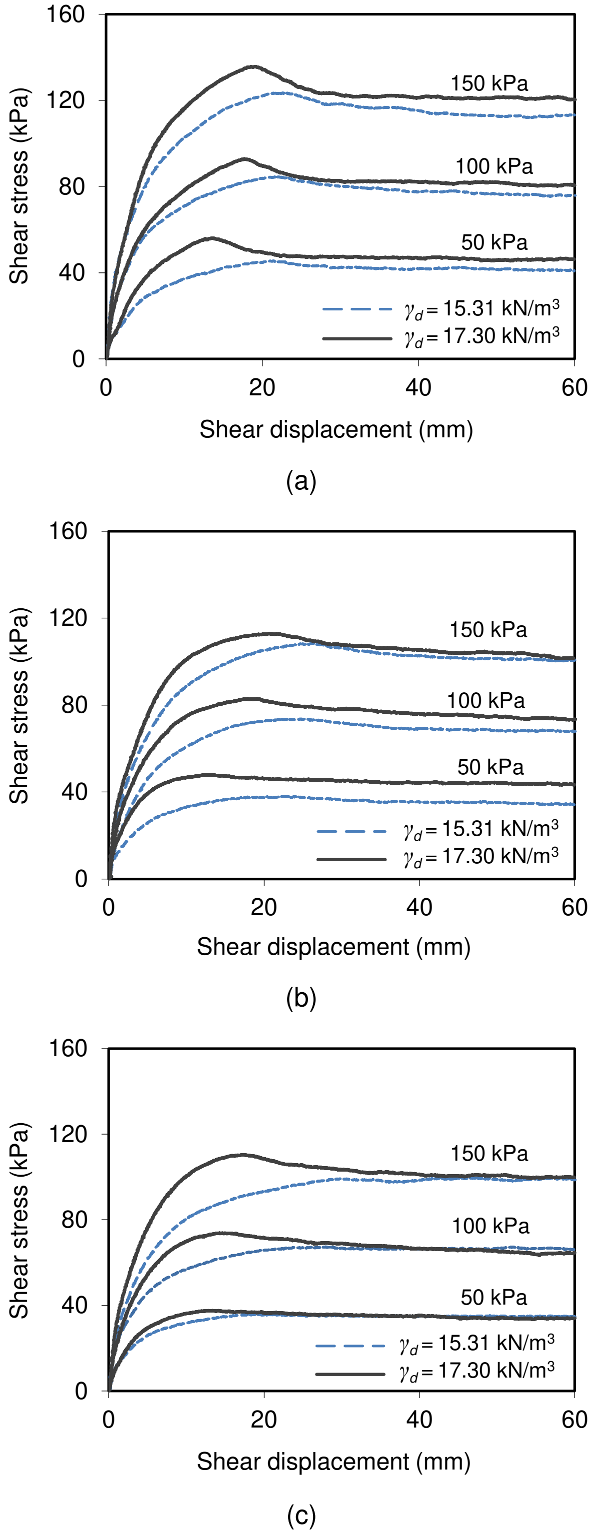


Figure 15. Influence of soil density on the shear strength of soil-GCR interface:

a) dry soil; b) $w = 0.5\ w_{opt}$; c) $w = w_{opt}$.

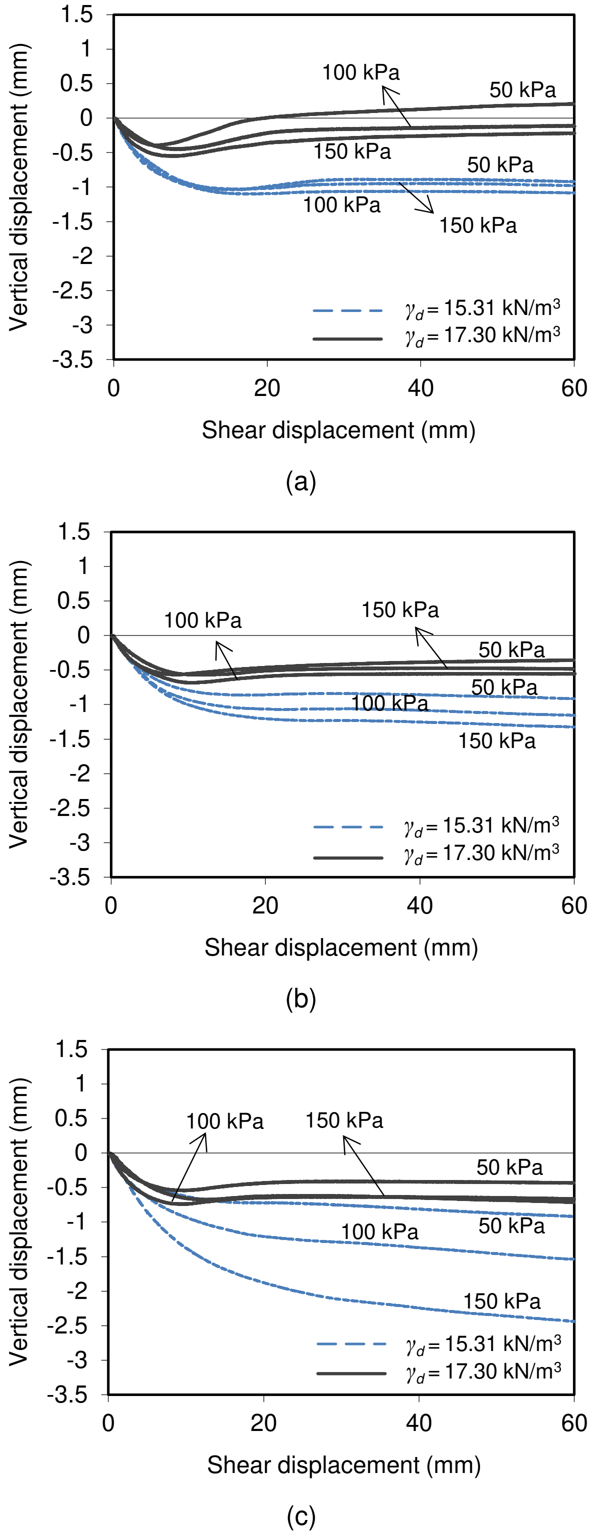
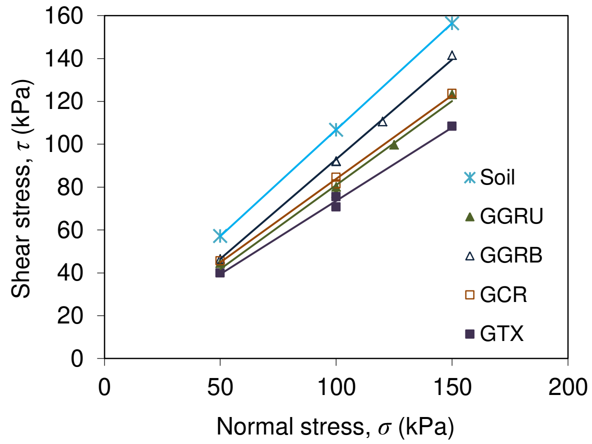
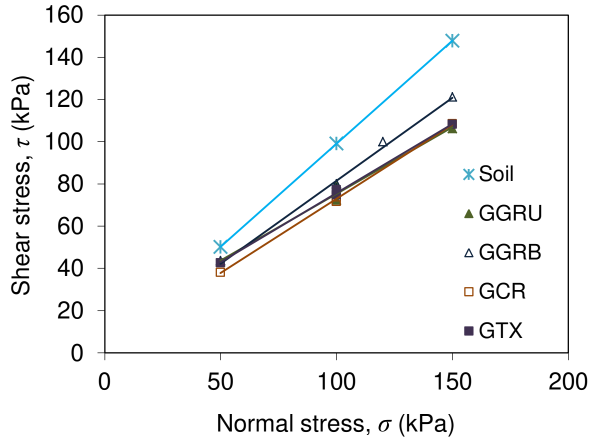


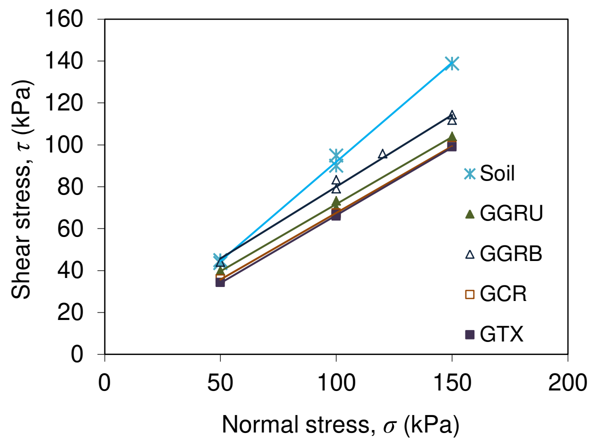
Figure 16. Influence of soil density on the vertical displacement of the loading plate center for soil-GCR interface: a) dry soil; b) $w = 0.5\ w_{opt}$; c) $w = w_{opt}$.



(a)

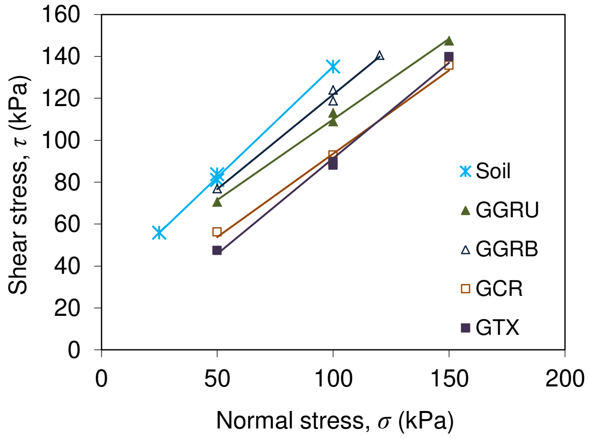


(b)

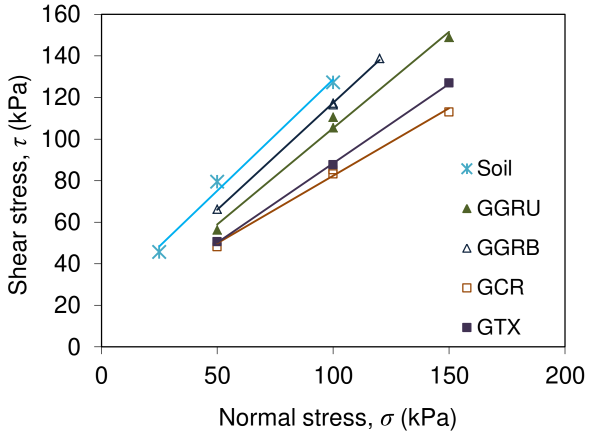


(c)

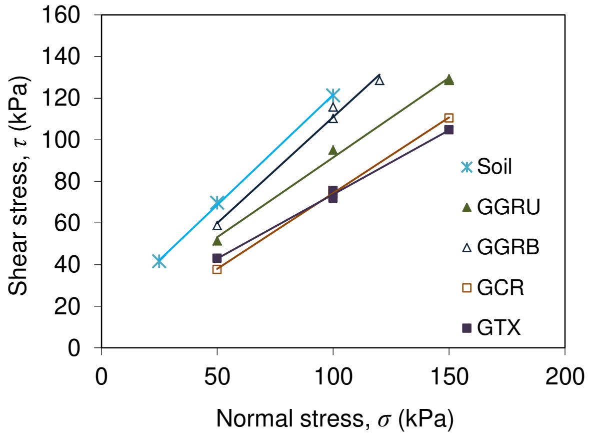
Figure 17. Influence of geosynthetic type on the peak shear strength of soil-geosynthetic interfaces for γ_d = 15.31 kN/m^3: a) dry soil; b) w = 0.5 w_{opt}; c) w = w_{opt}.



(a)



(b)



(c)

Figure 18. Influence of geosynthetic type on the peak shear strength of soil-geosynthetic interfaces for γ_d = 17.30 kN/m^3: a) dry soil; b) w = 0.5 w_{opt}; c) w = w_{opt}.