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Interface shear properties of geosynthetics and construction and demolition waste from large-scale direct shear tests

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ABSTRACT

In recent years environmental sustainability has demanded a progressive increase of waste recycling in general and waste value-added utilization in the construction industry in particular. As regards the application of Construction and Demolition Wastes (C&DW) in geotechnical works, it has been noticed that the use of recycled aggregates is found mostly in road construction. Value-added utilization of C&DW in geosynthetic reinforced structures is almost an unexplored field. This paper presents results of physical, mechanical and environmental characterization of recycled C&DW, as well as the direct shear behaviour of three recycled C&DW/geosynthetic interfaces. The C&DW material was collected from a recycling plant and came from the demolition of single-family houses and the cleaning of land with illegal deposits of C&DW. Two geogrids and one geocomposite reinforcement (high strength geotextile) were used to assess the behaviour of C&DW/geosynthetic interfaces. The environmental characterization of the C&DW, carried out through leaching tests, did not show environmental concerns. Direct shear test results have demonstrated that properly selected and compacted C&DW can exhibit shear strength similar to natural soils. The coefficients of interaction achieved for C&DW/geosynthetic interfaces compare well with those reported in the literature for soil/geosynthetic interfaces under similar conditions, which supports the feasibility of using these recycled materials as backfill in geosynthetic reinforced structures.

KEYWORDS: Geosynthetics; Environmental sustainability; Construction and Demolition Wastes; Interface shear strength; Large scale direct shear tests

1. Introduction

The reduction of non-renewable natural resource use is a constant concern in environmental preservation and encourages the use of alternative materials. About 50% of the materials extracted from the earth's crust are used in construction industry (European Commission, 2001). The significant consumption of minerals and ores, about 15ton/year per capita in the European Union (European Commission, 2007), makes it imperative to promote major changes in consumption patterns. The recycling of construction materials is an ancient practice, carried out by the Egyptians, Greeks and Romans. In the modern era it began to find expression in Europe after the Second World War with the use of crushed aggregates in the reconstruction of buildings.

Over recent years environmental sustainability has demanded a progressive increase of waste recycling in the construction industry. Several studies and applications of recycled Construction and Demolition Wastes (C&DW) have been performed, mainly related to the production of aggregates for use in concrete (Behera et al. 2014; Medina et al. 2014; Rao et al. 2007; Silva et al. 2014) and for use in base layers of transportation infrastructures (Agrela et al. 2012; Herrador et al. 2011; Jiménez et al. 2012; Poon and Chan, 2006). As regards the application of C&DW in geotechnical works, it has been noticed that waste reutilization is performed mainly in road construction, particularly in the base and sub-base layers of the infrastructures. Outside the scope of road infrastructures, there are not many references to C&DW applications in embankments. Apart from some recent studies (Arulrajah et al. 2014; Santos et al. 2013, 2014), the value-added utilization of recycled C&DW in geosynthetic reinforced structures is almost an unexplored field.

To broaden the potential application of C&DW in geotechnical works and, simultaneously, to assess the replacement of natural soils traditionally used in geosynthetic reinforced structures with recycled C&DW, a research project was carried out. The characterization of recycled C&DW/geosynthetic interfaces behaviour was one of the main goals of the project.

The interaction mechanism between the reinforcement and the fill material has the utmost importance in the design of geosynthetic reinforced structures. This mechanism depends on the fill properties, reinforcement characteristics and elements (fill and reinforcement) interaction.

Accurate identification of the interaction mechanism and the choice of the most suitable test for its characterization are important factors. The reinforcements tend to be pulled out in the upper part of the retained reinforced soil mass, so the soil-reinforcement interaction should be characterised by laboratory pullout tests. On the other hand, soil sliding is expected near the base of the slope and the interaction between the fill and the geosynthetic material is better characterised through direct shear tests (Vieira et al. 2013).

In recent decades many researchers have investigated the shear properties of soil-geosynthetic interfaces through direct shear tests (Liu and Martinez 2014; Esmaili et al. 2014; Khoury et al. 2011; Lee and Manjunath 2000; Liu et al. 2009a; Nakamura et al. 1999; Vieira et al. 2013). More recently, Santos and Vilar (2008) carried out direct shear and pullout tests to characterize the behaviour of C&DW/geogrid interfaces. To assess the viability of using geogrid-reinforced construction and demolition materials as alternative construction materials, Arulrajah et al. (2014) conducted direct shear tests using biaxial and triaxial geogrids as reinforcement elements.

This paper presents the results of direct shear tests carried out on a large scale apparatus to characterize the shear strength of recycled C&DW, with grain size distribution similar to the natural soils traditionally used in Portugal as backfill material of geosynthetic reinforced structures, as well as to assess the interaction mechanism between recycled C&DW and three distinct geosynthetics (a uniaxial HDPE geogrid, a uniaxial PET geogrid and a high strength geotextile). Physical and environmental characterization of the recycled C&DW are also presented.

2. Materials and methods

A fine grain recycled C&DW, coming mainly from the demolition of single-family houses and the cleaning of land with illegal deposits of C&DW (Figure 1) was used in this study.

The constituents of the C&DW can be found in Appendix A. The predominant materials of this recycled C&DW are concrete, mortar and unbound aggregates. A significant amount of soil was also identified.

The gradation of the material was determined according to the standard ISO/TS 17892-4 (2004). The particle size distribution determined by sieving and sedimentation is represented in Figure 2. The grading ranges recommended by the *Federal Highway Administration* (FHWA 2010) for Reinforced Soil Slopes (RSS) and Mechanically Stabilized Earth Walls (MSEW) and by the National Concrete Masonry Association – NCMA (NCMA 2010) for Segmental

Retaining Walls (SRW) are also shown in Figure 2. The recycled C&DW under analysis satisfies the guidelines for RSS structures and SRW structures according to FHWA and NCMA respectively, excepting a slight deviation between 0.5 mm and 3 mm.

Additional properties of the recycled C&DW and the standardised procedures used to estimate them are provided in Appendix A.

Three commercially available geosynthetics were used in this study: an extruded uniaxial high density polyethylene (HDPE) geogrid, a laid uniaxial geogrid made of extruded polyester (PET) bars with welded rigid junctions, and a high-strength composite geotextile consisting of polypropylene continuous-filament needle-punched nonwoven and high-strength polyester yarns (unidirectional reinforcement). Table 1 summarizes the main properties of the geosynthetics.

The direct shear tests were performed on a large scale direct shear device. The shear box comprises an upper container, fixed horizontally, with dimensions of 300 mm × 600 mm in width and length and 150 mm high, and a lower container 340 mm × 800 mm in width and length and 100 mm high, rigidly fixed to a mobile platform running on low friction linear guides. A rigid base or a rigid ring (reduction box) can be inserted into the lower container. More details about this prototype can be found in Vieira et al. (2013).

Reduced contact area direct shear tests were carried out to characterize the shear strength of recycled C&DW and C&DW/geogrid interfaces. Following the recommendations of EN ISO 12957-1 (2005), direct shear tests with the rigid base placed inside the lower box (constant contact area shear test) were performed to characterize the C&DW/geocomposite interface.

The C&DW was put into the shear boxes, at its air-dried water content, with relative density (I_D) of 70%. For reduced contact area direct shear tests, each container was filled with four layers of thickness equal to 25 mm to the target unit weight. Geogrid specimens were held with screws at the front edge of the lower box outside the shear area.

To prevent relative displacement between the geosynthetic and the rigid support, an aluminium oxide abrasive sheet (P80 type) was glued to the support and the geocomposite specimen was held with screws at the two edges of the lower box. The C&DW was placed inside the upper shear box and compacted under similar conditions as those described for reduced contact area direct shear tests.

All direct shear tests were carried out with a constant displacement rate of 1 mm/min at normal stresses of 50, 100 and 150 kPa. To evaluate the shear strength of the recycled C&DW,

direct shear tests at normal stress of 25 kPa were also carried out. For assessing the variability of the results, each test was performed three times under similar conditions.

Prior to shearing, the normal stress was applied to the specimens for one hour. After this period of time, the settlement of the C&DW under the pre-established normal stress was stabilised in all specimens. Vertical displacements of the loading plate before and during shear were recorded with a linear variable displacement transducer (LVDT). The tests were stopped once the horizontal shear displacement reached approximately 60 mm.

3. RESULTS AND DISCUSSION

3.1. Mechanical and environmental characterization of the recycled C&DW

Regardless of the fact that the use of C&DW in geosynthetic reinforced structures can attain a good mechanical performance, environmental concerns regarding the potential contamination of groundwater impose an assessment of the release of dangerous substances through the leaching behaviour of these recycled materials. Thus, laboratory leaching tests were carried out in accordance with European Standard EN 12457-4 (2002).

Table 2 presents the leaching test results, as well as the acceptance criteria of maximum leached concentration for inert landfill, as defined by the European Council Decision 2003/33/EC.

It can be concluded from the analysis of the results presented in Table 2 that only sulphate exceeded the maximum limit established by European and Portuguese legislation. However, Directive 2003/33/EC states that "*if the waste does not meet these values for sulphate, it may still be considered as complying with the acceptance criteria if the leaching does not exceed 6000 mg/kg at L/S = 10 l/kg, determined either by a batch leaching test or by a percolation test under conditions approaching local equilibrium*."

The shear strength of the recycled C&DW was evaluated through direct shear tests carried out under confining pressures of 25, 50, 100 and 150 kPa. Shear stress-shear displacement curves of direct shear tests conducted under normal stress of 50 kPa are illustrated in Figure 3. Results of direct shear tests carried out under similar conditions (I_D = 70% and σ = 50 kPa) with a fine grain sand (D_{50} = 0.45 mm; C_u = 1.9) and a coarse grain sand (D_{50} = 1.32 mm; C_u = 3.6) are also represented in Figure 3.

Figure 3 shows low variability in the results, as the maximum difference among the values reached in three tests for peak shear strength and large displacement shear strength is lower than 12% and 4%, respectively. Under similar conditions, the recycled C&DW had higher shear strength than that of natural soils, although the peak shear strength tends to be reached with larger shear displacements.

The evolution of shear stress as a function of shear displacement for direct shear tests carried out under discrete confining pressures (25, 50, 100 and 150 kPa) is illustrated in Figure 4. In general, the shear stress–shear displacement curves show a well-defined peak shear strength for shear displacements that increased with normal stress. With the exception of direct shear tests carried out under normal stress of 25 kPa, for which the maximum difference among the values of peak shear strength reached in the three tests is approximately 30%, the variability of the results is not significant.

The difficulties associated with direct shear tests under lower confining pressures are well known. Therefore, the highest variability recorded in the direct shear tests at 25 kPa is not surprising.

Several experimental studies have suggested that the failure criteria of many soils are not linear, particularly in the range of small normal stresses (Bishop et al. 1965; Maksimovic 1989; Baker 2004). The cohesion of a soil, defined as the shear strength when the normal stress is null, can be true or apparent. True cohesion appears in cemented soils and in overconsolidated soils. Apparent cohesion can result from soil matric suction in unsaturated soils, as well as from the intercept in the shear stress axis when a linear failure envelope is adjusted to a curved failure envelope.

Among the nonlinear shear strength envelopes, the most commonly used relation between the normal stress, σ, and the shear stress, τ, has the general form $\tau = A\sigma^b$, where A and b are constants with no physical meaning and depending on the units (Eid 2010).

Two failure envelopes were fitted to the peak shear strengths recorded in the tests carried out to characterise the recycled C&DW (Figure 5): the best fit straight line (Mohr-Coulomb failure criterion) and the power-law relation that best represents the results.

Figure 5 shows that both failure envelopes fit the results with similar correlation coefficients, R^2. For normal stresses higher than 40 kPa the shear strength failure envelopes are very close. As expected, the main difference occurs for low normal stresses.

Due to the composition of this C&D material, some cohesion is expected. The nonlinear failure envelope plotted in Figure 5 also suggests this. Even so, the estimated cohesion value based on Mohr-Coulomb failure criterion may have an apparent component resulting from the adjustment of a linear failure envelope for low normal stress.

The failure envelopes presented in Figure 5 point out the importance of limiting the range of validity of the estimated shear strength parameters based on linear failure envelopes. Following the Coulomb failure criterion, the cohesion and the friction angle of the recycled C&DW is 20.9 kPa and 43.1º, respectively.

Figure 6 compares peak and large displacement shear strength, as well as the corresponding linear best fit straight lines. The large displacement shear strengths were estimated as the mean values of the shear stresses recorded for displacements greater than 50mm (Figure 4). Following the Coulomb failure criterion, the large displacement shear strength can be defined by a friction angle of 40.9º and cohesion of 4.0 kPa. Comparing these parameters with those for peak shear strength (peak friction angle = 43.1° and cohesion = 20.9 kPa), a slight decrease in the friction angle but a large reduction (higher than 80%) of the cohesive term is noticed.

Even if most of the guidelines from different countries explicitly indicate the use of peak friction angles in the design (FHWA 2010; BS 8006 2010; NCMA 2010; EBGEO 2011), the use of peak shear strength parameters instead of residual shear strength parameters in the design of geosynthetic reinforced soil structures is not consensual. Jewell (1996) suggests the selection of a design value for the soil shearing resistance equal to the critical state shear resistance, and Leshchinsky (2001) proposes a hybrid approach where the critical slip surface is determined based on peak shear strength and the required long-term reinforcement strength is estimated using the residual shear strength parameters.

Regarding the cohesion of the fill material, earlier versions of some guidelines have precluded the cohesive component of the shear strength, but recent editions (AASHTO 2012; BS 8006: 2010) allow the use of cohesive fills. Moreover, based on the evidence that there are many reinforced soil walls that have been built with granular backfills having significant fines content and that in general these walls have performed very well, Miyata and Bathrust (2007) extended the K-Stiffness method, developed to estimate the maximum reinforcement loads in reinforcement layers of reinforced soil walls, to cohesive backfill materials.

The shear strength achieved for this recycled C&DW is encouraging, since it compares with the values for natural soils. Nevertheless, the cohesion should be used with great caution in design, due to its significant reduction for large strain conditions.

3.2. *Characterization of Recycled C&DW/geosynthetic interfaces*

The evolution of the shear stresses with the imposed shear displacement recorded in direct shear tests carried out to characterize C&DW/geosynthetic interfaces under distinct values of confining pressure is illustrated in Figure 7. Figure 7(a) refers to the interface between the recycled C&DW and the HDPE geogrid (GGR1), Figure 7(b) presents the results related to the PET geogrid (GGR2) and Figure 7(c) shows the curves for the interface between the recycled C&DW and the high-strength composite geotextile (GCR).

In general and as observed for the recycled C&DW, the shear stress–shear displacement curves show a well-defined peak shear strength recorded for shear displacements that increased with the confining pressure, followed by a reduction of shear stress and an almost constant shear stress stage (residual shear strength). The results presented in Figure 7 provide evidence of reduced variability of the results.

Figure 8 presents peak and large displacement shear strengths achieved in direct shear tests carried out to characterize the interfaces under analysis and the corresponding linear best fit straight lines. Shear strength parameters of the interfaces are summarized in Table 3.

Similar to what was observed in the characterization of recycled C&DW, the recycled C&DW/geogrid interfaces underwent a slight decrease in the friction angle and a significant reduction of the cohesive component for large shear displacements. For the C&DW/geotextile (GCR) interface the decrease in the friction angle for large shear displacements was more pronounced than that recorded for C&DW/geogrid interfaces. This probably results from the mobilisation of the internal C&DW strength along the geogrid apertures, which does not exist in the high strength geotextile (GCR).

The interface between the recycled C&DW and the PET geogrid (GGR2) showed higher shear strength parameters, with a particularly high peak adhesion. This evidence could be explained by the large openings of the PET geogrid which allows a significant area where the internal shear strength of the C&DW is mobilised.

3.3. Coefficients of interaction

The coefficient of interaction between the backfill and the reinforcement is one of the key parameters in the design of geosynthetic reinforced structures. When the interaction between both materials is characterized through direct shear tests, the coefficient of interaction, f_g, can be defined as the ratio of the maximum shear stress in a C&DW/geosynthetic direct shear test, to the maximum shear stress in a direct shear test on C&DW, under the same normal stress, σ:

$$f_g = \frac{\tau^{max}_{C\&DW/geo}(\sigma)}{\tau^{max}_{C\&DW}(\sigma)} \qquad (1)$$

As each direct shear test was carried out three times under similar conditions, the coefficient of interaction should be estimated by the ratio of the mean value of maximum shear stresses recorded in the C&DW/geosynthetic direct shear tests, to the mean value of maximum shear stresses reached in shear tests on C&DW, under the same normal stress σ:

$$f_g^m=\frac{\text{average}\left\{\tau_{C\&DW/geo}^{max,1}(\sigma);\ \tau_{C\&DW/geo}^{max,2}(\sigma);\ \tau_{C\&DW/geo}^{max,3}(\sigma)\right\}}{\text{average}\left\{\tau_{C\&DW}^{max,1}(\sigma);\ \tau_{C\&DW}^{max,2}(\sigma);\ \tau_{C\&DW}^{max,3}(\sigma)\right\}} \qquad (2)$$

Figure 9 presents the coefficients of interaction, as a function of the normal stress, for the interfaces under analysis. For C&DW/geogrid interfaces the coefficients of interaction increased with the confining pressure, the highest values being recorded for the interface with PET geogrid (GGR2). An increase trend of the coefficient of interaction is also noted for C&DW/geotextile (GCR) interface. Notwithstanding, the values for normal stress of 50 kPa and 100 kPa are quite similar.

The coefficients of interaction are in the ranges 0.64-0.74, 0.76-0.83 and 0.67-0.74 for the interfaces C&DW/GGR1, C&DW/GGR2 and C&DW/GCR, respectively.

The coefficients of interaction achieved for the C&DW/GGR2 (PET geogrid) interface are similar to the values reported by Abu-Farsakh et al. (2007) for clay/geogrid interfaces (see Appendix B). The values of f_g achieved in the present study for C&DW/geogrid interfaces are lower than those reported by Liu et al. (2009b) for sand/PET geogrid interfaces and by Ferreira et al. (2013) for residual soil/biaxial geogrid interface. However, they are higher than those presented by Arulrajah et al. (2014) for interfaces between a polypropylene biaxial geogrid and recycled concrete aggregate or crushed bricks.

For the interface between the C&DW and the high strength geotextile (GCR) the coefficients of interaction are lower than those reported by Vieira et al. (2013) for an interface between a similar high strength geotextile and a poorly graded sand. Nevertheless, they are within the same order of magnitude as those presented by Ferreira et al. (2013) for an interface between a residual soil from granite and a geotextile, and those reported by Liu et al. (2009b) for sand/geotextile interfaces.

Taking into account the significant reduction of the cohesive component of shear strength for large shear conditions either for the C&DW or for C&DW/geosynthetic interfaces, coefficients of interaction for residual shear strength were also estimated. Their values were determined by equation (2) replacing the maximum shear strengths by residual shear strengths.

The coefficients of interaction for residual shear strengths are in the ranges 0.64-0.77, 0.83-0.90 and 0.61-0.75 for the interfaces C&DW/GGR1, C&DW/GGR2 and C&DW/GCR,

respectively. Except for the interface C&DW/GGR2, for which the coefficients of interaction for residual shear strength were higher than those for peak shear strength, the coefficients of interaction for residual shear strengths are in similar ranges to those presented for peak shear strength.

4. Conclusions

The interaction mechanism between recycled C&DW, with grain size distribution similar to the natural soils traditionally used as backfill material of geosynthetic reinforced structures in Portugal, and three geosynthetics (a uniaxial HDPE geogrid, a uniaxial PET geogrid and a high strength geotextile) was studied through large scale direct shear tests. Physical, mechanical and environmental characterization of the recycled C&DW was also reported.

The environmental characterization of the C&DW, carried out through leaching tests, has shown that this C&DW meets the acceptance criteria for inert landfill. No environmental concerns were identified.

Based on the analysis and interpretation of direct shear tests results, the following conclusions can be drawn:

- properly selected and compacted C&DW can exhibit shear strength similar to (or even higher) the backfill materials commonly used in the construction of geosynthetic reinforced structures;
- comparing peak and residual shear strength parameters, a slight decrease in the friction angle but a large reduction of the cohesive term for large shear displacement conditions was noticed. This finding is valid either for C&DW shear strength or for C&DW/geosynthetic interface shear strength;
- the cohesion estimated based on Mohr-Coulomb failure criterion may have an apparent component resulting from the adjustment of a linear shear strength failure envelope for low normal stresses. The validity of the shear strength parameters is limited to the range of tested normal stresses;
- in general the coefficients of interaction achieved for C&DW/geosynthetic interfaces compare well with those reported in the literature for soil/geosynthetic interfaces under similar conditions.

The shear strength achieved for this recycled C&DW is encouraging, since it compares with the values for natural soils. Nevertheless, the cohesion should be used with great caution in design, due to the significant reduction for large strain conditions.

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NOTATIONS

Basic SI units are given in parentheses.

c - soil cohesion (Pa)

c_a - adhesion (Pa)

$c_{a,cv}$ - adhesion corresponding to large displacement shear strength (Pa)

$c_{a,p}$ - adhesion corresponding to the peak shear strength (Pa)

C_u - soil uniformity coefficient (dimensionless)

D_i - diameter corresponding to i% passing (m)

f_g - coefficient of interaction (dimensionless)

f_g^m - coefficient of interaction estimated as a mean value (dimensionless)

f_g^p - coefficient of interaction estimated by shear strength parameters (dimensionless)

I_D - relative density or density index (dimensionless)

w_{opt} - soil optimum moisture content (dimensionless)

δ - interface friction angle (º)

δ_{cv} - interface friction angle for large relative displacements (degrees)

δ_p - interface peak friction angle (degrees)

ϕ - soil internal friction angle (degrees)

γ_{dmax} - soil maximum dry unit weight (N/m^3)

σ - normal stress (Pa)

τ - shear stress (Pa)

$\tau_{C\&DW}^{max}(\sigma)$ - maximum shear stress in direct shear test on C&DW under normal stress σ (Pa)

$\tau_{C\&DW/geo}^{max}(\sigma)$ - maximum shear stress in C&DW/geosynthetic direct shear test under normal stress σ (Pa)

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Figure 1 - Visual aspect of the recycled C&DW (ruler in centimetres).

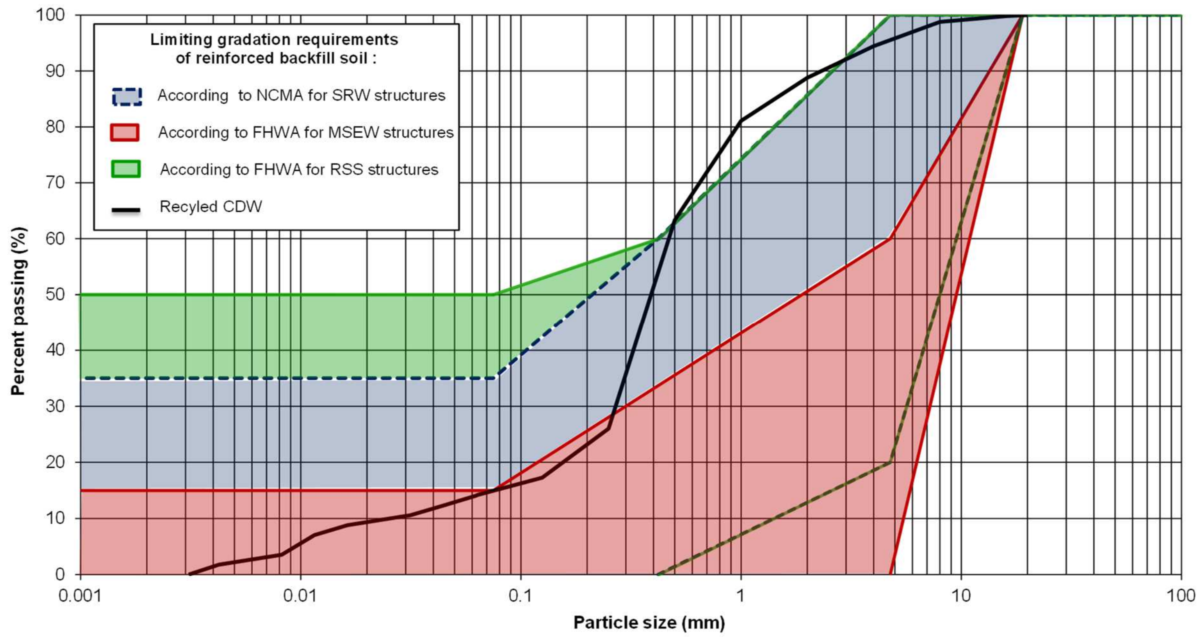


Figure 2 - Particle size distribution of the Recycled C&DW and gradation requirements of FHWA and NCMA.

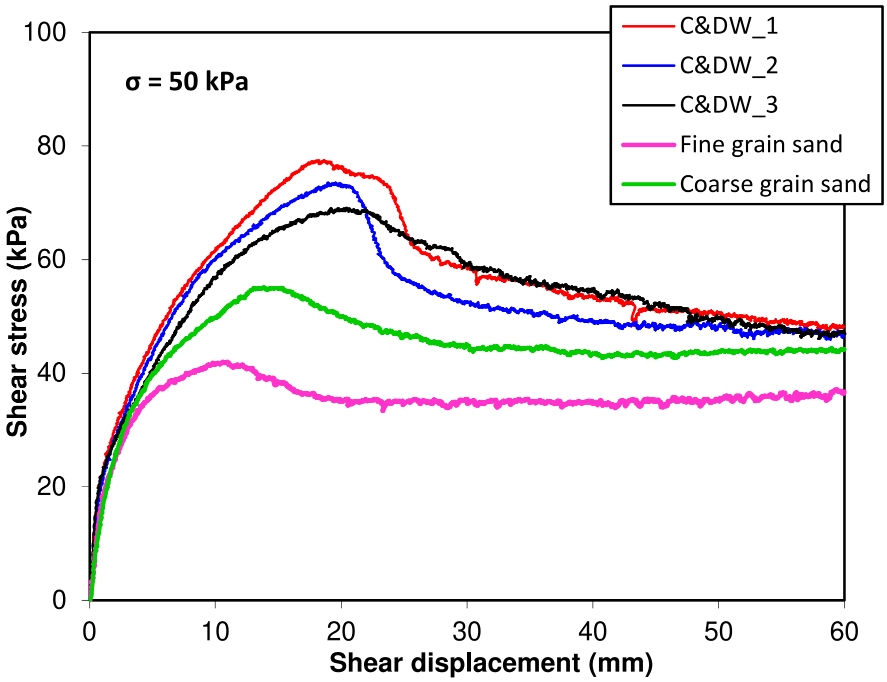


Figure 3 – Comparison of shear stress-shear displacement curves achieved for recycled C&DW and natural sands (σ = 50kPa).

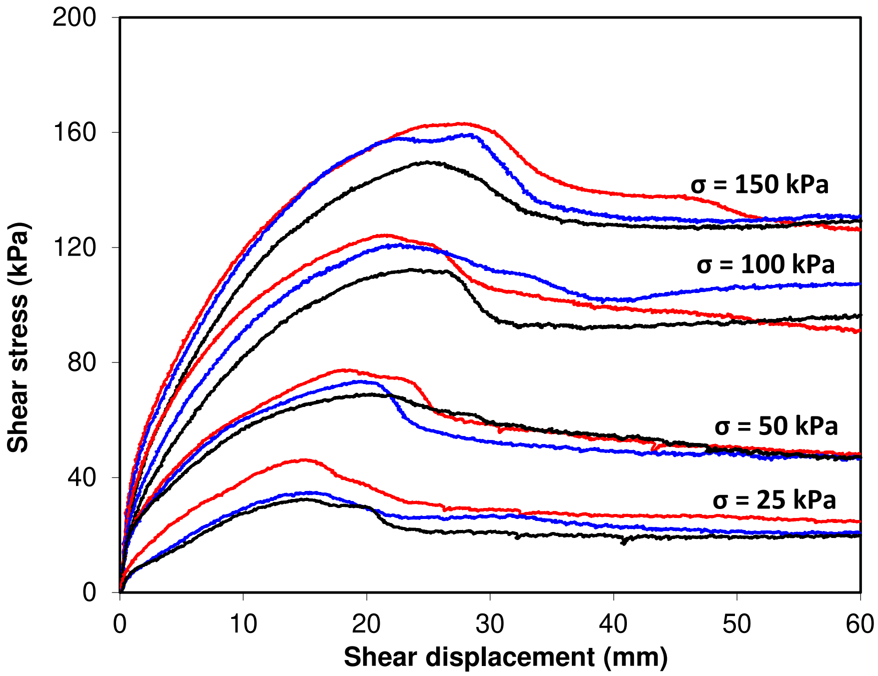


Figure 4 - Direct shear behaviour of recycled C&DW under distinct normal stress values.

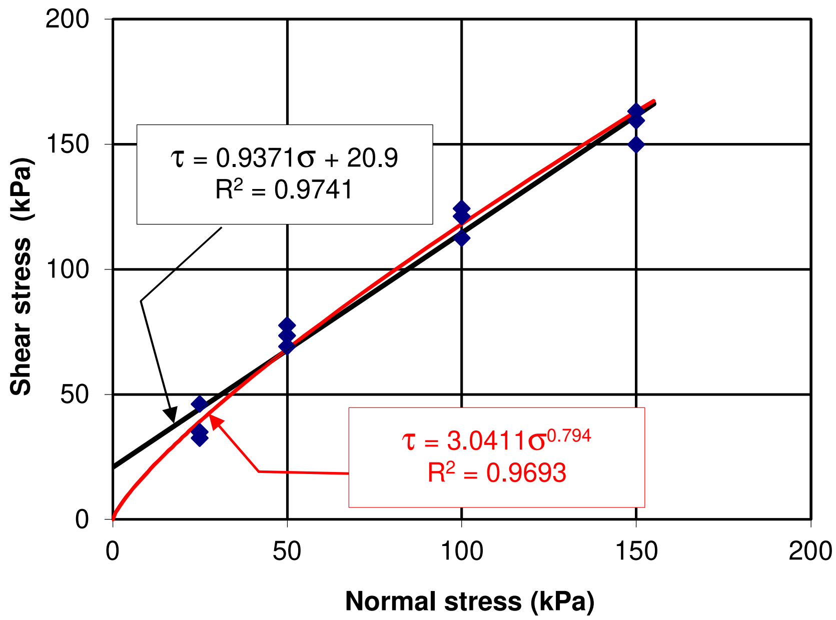


Figure 5 - Failure envelopes for C&DW peak shear strength.

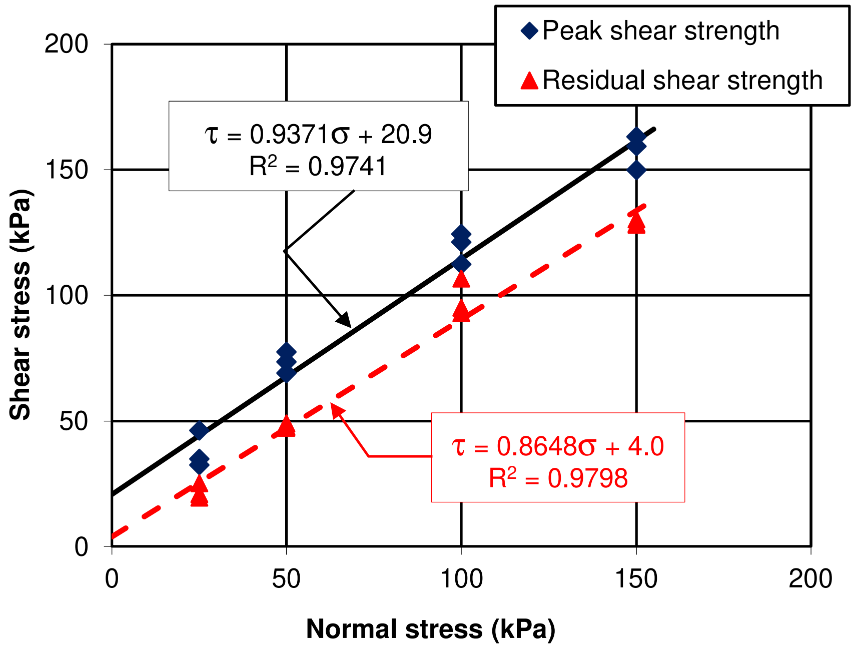
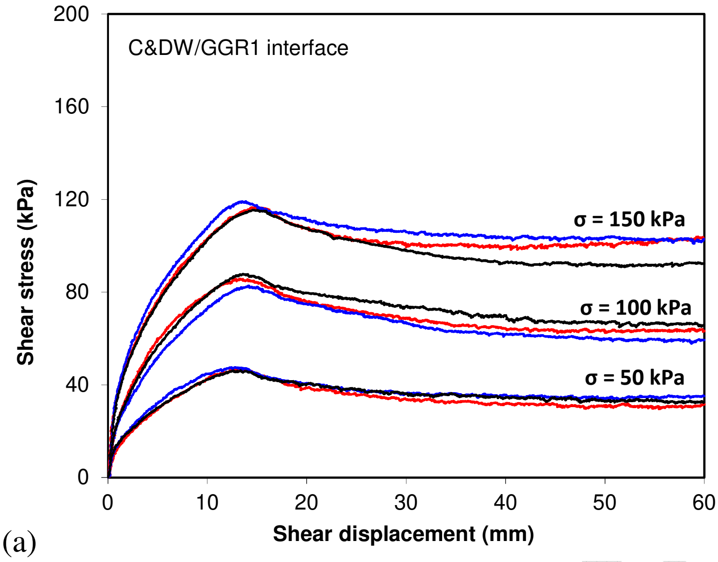
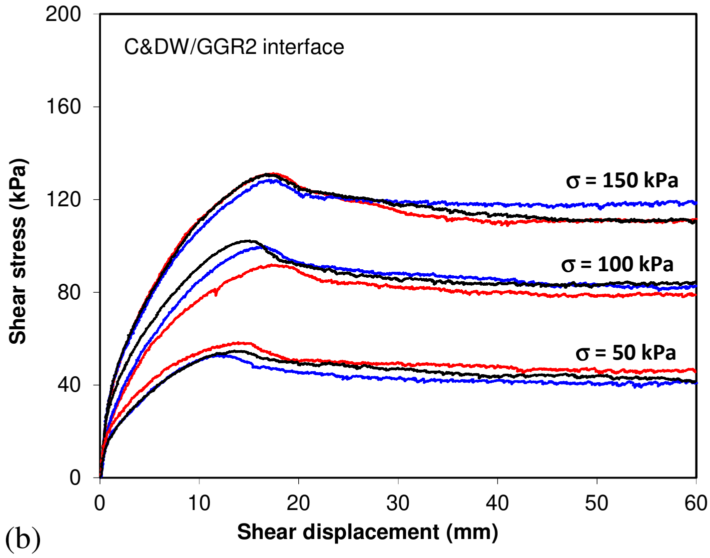


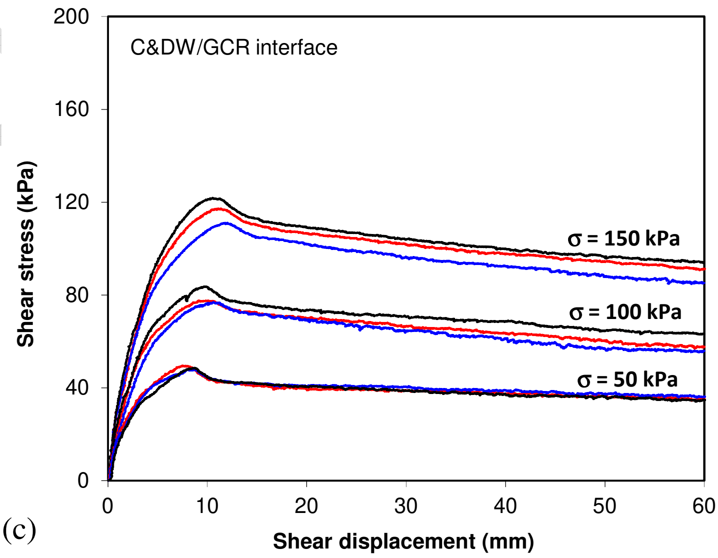
Figure 6 - Failure envelopes for C&DW peak shear strength and large displacement shear strength.


C&DW/GGR1 interface
σ = 150 kPa
σ = 100 kPa
σ = 50 kPa
Shear stress (kPa)
Shear displacement (mm)
0
40
80
120
160
200
0
10
20
30
40
50
60

(a)

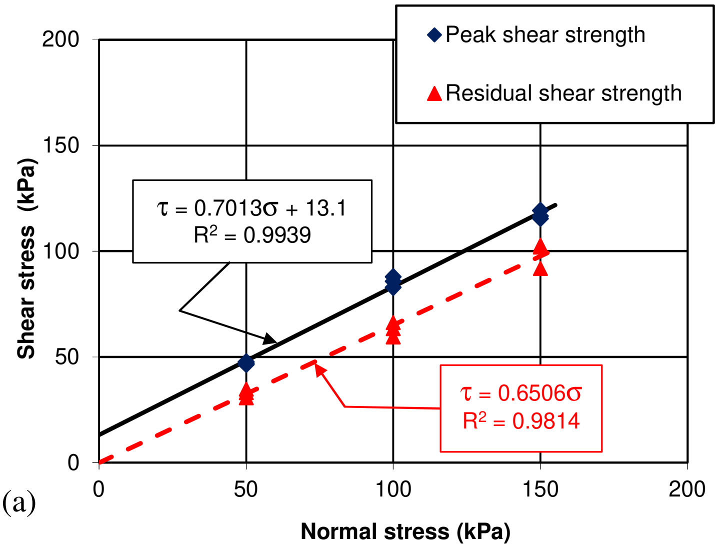

C&DW/GGR2 interface
σ = 150 kPa
σ = 100 kPa
σ = 50 kPa
Shear stress (kPa)
Shear displacement (mm)
0
40
80
120
160
200
0
10
20
30
40
50
60

(b)

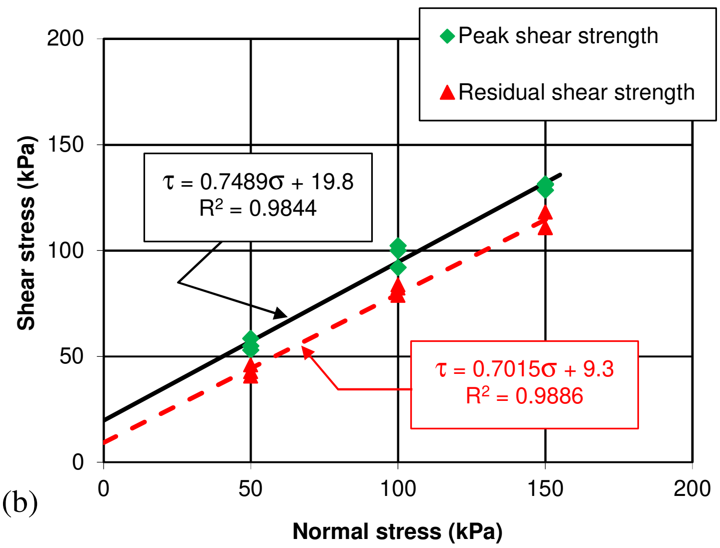

C&DW/GCR interface
σ = 150 kPa
σ = 100 kPa
σ = 50 kPa
Shear stress (kPa)
Shear displacement (mm)
0
40
80
120
160
200
0
10
20
30
40
50
60

(c)

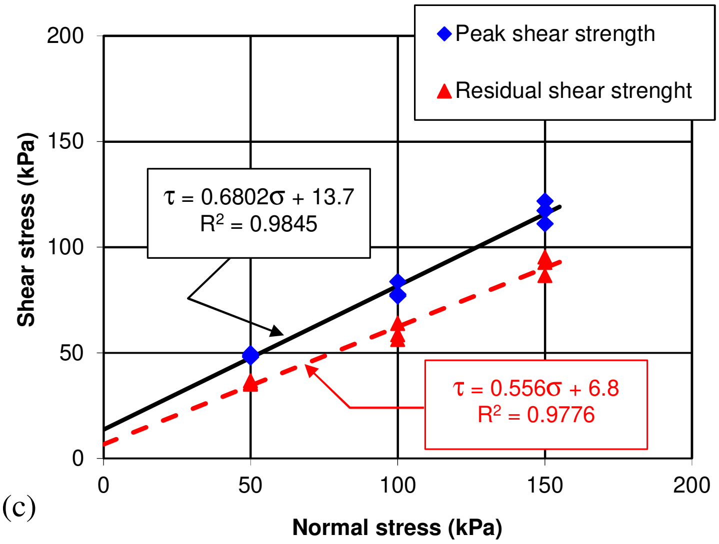
Figure 7 - Results of direct shear tests for: a) C&DW/GGR1 interface; b) C&DW/GGR2 interface; c) C&DW/GCR interface.



(a)



(b)



(c)

Figure 8 - Failure envelopes for peak and large displacement shear strength of: a) C&DW/GGR1 interface; b) C&DW/GGR2 interface; c) C&DW/GCR interface.

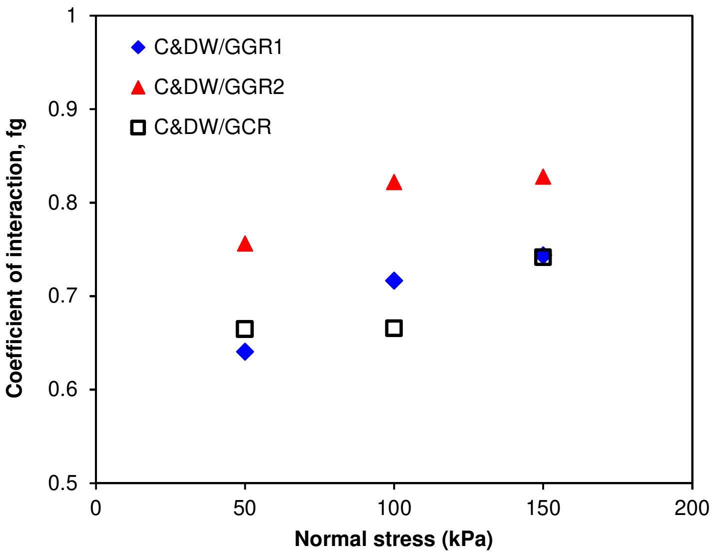


Figure 9 - Coefficients of interaction for peak shear strength against normal stress.

TABLES

Table 1 - Properties of the geosynthetics.

	GGR1	GGR2	GCR
Raw material	HDPE	PET	PP & PET
Mass per unit area (g/m^2)	450	380	340
Aperture dimensions (mm)	16×219	30×73	-
Mean value of the tensile strength (kN/m)	60	88	71
Elongation at maximum load, ε_{Tmax} (%)	10	9	10
Secant tensile stiffness at 2% strain (kN/m)	1085	1182	647
Secant tensile stiffness at 5% strain (kN/m)	718	928	577
Secant tensile stiffness at ε_{Tmax} (kN/m)	597	907	728

Table 2 - Leaching test results.

Parameter	Value (mg/kg)	Acceptance criteria for leached concentrations – Inert landfill
Arsenic, As	0.020	0.5
Lead, Pb	< 0.01	0.5
Cadmium, Cd	< 0.003	0.04
Chromium, Cr	0.015	0.5
Copper, Cu	0.12	2
Nickel, Ni	< 0.01	0.4
Mercury, Hg	< 0.002	0.01
Zinc, Zn	< 0.1	4
Barium, Ba	0.12	20
Molybdenum, Mo	0.027	0.5
Antimony, Sb	< 0.01	0.06
Selenium, Se	< 0.02	0.1
Chloride, Cl	130	800
Fluoride, F	2.7	10
Sulphate, SO_4	1900	1000
Dissolved Organic Carbon (DOC)	47	500
Dissolved Solids, DS (mg/kg)	2630	4000
pH	7.8	-

Table 3 - Summary of shear strength parameters for C&DW/geosynthetic interfaces.

	Peak shear strength		Residual shear strength	
Interface	δ_p (º)	$c_{a,p}$ (kPa)	δ_{cv} (º)	$c_{a,cv}$ (kPa)
C&DW/GGR1	35.0	13.1	33.0	0
C&DW/GGR2	36.8	19.8	35.0	9.3
C&DW/GCR	34.2	13.7	29.1	6.8