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A REVIEW OF DIRECT SHEAR AND INCLINED PLANE TESTS RESULTS FOR DIFFERENT INTERFACES IN LANDFILL CAPPING

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ABSTRACT

One of the crucial aspects in design of a landfill capping is the interface behavior between the different layers of the cover system, from levelling layer above waste up to the topsoil. Design guidelines and international codes require a geotechnical stability analysis to be performed along every interface. The critical interface is the one which gives the minimum shear resistance, in terms of friction angle and adhesion. Evaluation of the correct values to be used is then essential. Shear resistance at the interface between different geosynthetics or between a geosynthetic and a soil can be measured through laboratory tests. Testing methods are EN ISO 12957-1 and ASTM D5321 (for direct shear test) and EN ISO 12957-2 (for inclined plane). The paper briefly describes direct shear and inclined plane testing methods and enhances pros and cons. In the last 25 years the authors have coordinated a great number of the above tests with different types of geosynthetics and soils (e.g., Cazzuffi and Recalcati, 2018). The main results of these tests are reported in the paper, summarizing the values obtained with contact interface between different products belonging to the same families. The purpose of this work is to validate the already big database of interface strength measured with direct shear tests (e.g., Koerner and Narejo, 2005) and to evaluate the differences with the results obtained for the different types of tests. This can give to designers the chance to have a critical approach toward the most suitable testing method to be used according to the specific needs of a project.

1. INTRODUCTION

Geosynthetics are increasingly used for a lot of engineering applications. In particular, they are becoming the consolidated solution in capping systems of a municipal solid waste (MSW) landfill because of various factors, as the economic benefits that come through increased void space, quicker construction times and correct compliance to the environmental regulatory requirements. They are typically used in conjunction with soil and also with other type of geosynthetics and could perform several functions, such as:

- drainage (e.g., geocomposites and geonets);
- separation and filter (e.g., geotextiles);
- hydraulic barriers (e.g., geosynthetic clay liners (GCLs) and/or geomembranes);
- erosion control (e.g., geomats and geocells);
- reinforcement (e.g., geogrids).

They are employed, in all range of the above functions, in the capping of a landfill waste containment system and in a slope side erosion control system.

As very well known, the main purpose of the capping system is to guarantee the following goals:

- insulate wastes from the external environment;
- control water from precipitations entering the landfill body;
- prevent surface water from entering the landfilled waste;
- avoid the risks of subsidence and sliding.

Traditional schemes for gas drainage layer, barrier mineral layer and liquid drainage layer usually foresee natural soils layers having a minimum thickness of 0.50 m.

Modern production technologies, together with the increasingly stringent quality control requirements guarantee that geosynthetics CE marked and supplied by a certified company can provide to customers and to designers a level of efficiency, durability and reliability higher than any natural material.

The reasons to replace natural material with geosynthetics are various.

The most relevant are technical reasons: the stratigraphy foreseen by the European Directive is sometimes

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not compatible with the geometry of the landfill bodies, particularly when the same have been designed and constructed well before the Directive was active. This type of problem is amplified whenever the landfill site is in a seismic area, and when the Eurocodes 7 and Eurocode 8 must be followed.

Another reason is economical: e.g., granular materials used to guarantee the proper drainage must be clean coarse sands or gravel. The need to bring on site huge quantities of a quite expensive material, and the difficulties to collect all the required material from the same quarry or source and then the difficulties to guarantee a proper quality control on site make this solution extremely expensive (Cazzuffi and Recalcati, 2018; Moraci et al., 2014).

Last, but not least, it is necessary to take into account the environmental impact (Grossule and Stegmann, 2020).

In particular areas, the use of natural materials (gravel) causes important costs from the environmental point of view. Quarries have to be excavated, and the material have to be moved to the site by means of huge trucks, causing problems in terms of traffic and pollution. To understand the size of this type of problem, it is possible to consider a small landfill (40.000 m^2, corresponding to a surface of 200 m x 200 m). To guarantee the drainage layers it is necessary to bring on site 40.000 m^3 gravel (about 4000 trucks each of 10 m^3).

From a technical point of view, the use of geosynthetics in a cover system, in combination with other geosynthetics or soil layers, introduces potential weakness surfaces or interfaces of low shear strength. The interface shear behavior greatly contributes to the response of the whole system and may control its performance; therefore, a requirement to assess the stability along interfaces between geosynthetic and geosynthetic and between soil and geosynthetic has to be considered.

Design guidelines and international codes require a geotechnical stability analysis to be performed along every interface. The critical interface is the one which gives the minimum shear resistance, in terms of friction angle and adhesion. Evaluation of the correct values to be used is, then, essential.

1.1 Acronyms

- HDPE-S: High Density Polyethylene - smooth surface Geomembrane
- HDPE-T: High Density Polyethylene - textured surface Geomembrane
- NW-NP GT: Nonwoven Needle Punched Geotextile
- GCL: Geosynthetic Clay Liner
- GCD: Drainage Geocomposite
- PVC: Poly Vinyl Chloride
- LLDPE: Linear Low Density Polyethylene
- EVA: Ethyl Vinyl Acetate

2. THEORETICAL SHEAR STRENGTH SIGNIFICANCE

A failure criterion is a definition of the conditions that determine the failure of a material. According to the characteristics of the material, this definition can be given in terms of stress or deformations.

In the case of soils, the failure criteria that have received the most credit are those that refer to a limit situation described in terms of stress. Therefore, shear strength of a soil mass is the internal resistance, per unit area, that the soil mass can offer to resist failure and sliding along any plane inside it.

In 1900, Mohr presented a theory for rupture in materials. According to this theory, failure along a plane in a material occurs by a critical combination of normal and shear stresses, and not by either maximum normal or shear stress alone. Thus, the functional relationship between the shear stress on a given failure plane was shown to be a function of the normal stress acting on that plane:

$$\tau = f(\sigma) \tag{1}$$

where: τ is the shear stress at failure and σ is the normal stress on the failure plane.

If a series of shear tests at different values of normal stress are performed, and the stress circles corresponding to failure are plotted for each test, at least one point on each circle must represent the normal and shear stress combination associated with failure. As the number of tests increases, a failure envelope (line tangent to the failure circles) for the material becomes evident.

In general, the failure envelope could be a curved line for many materials. This has been demonstrated experimentally in the laboratory/field for many soft/stiff soils, especially at low normal stress range (Penman,1953; Holtz and Gibbs, 1956; Bishop et al., 1965; Vesic and Clough, 1968; Marsland, 1971; Ponce and Bell, 1971; Lefebvre, 1981; Atkinson and Farrar, 1985; Day and Maksimovic,1994; Maksimovic, 1989).

This nonlinearity can result from various complex mechanisms such as particle crushing, particle reorientation, and stress history. The curvature of the failure envelope implies that the instantaneous friction angle reduces with increasing normal stress.

Stability analysis of slopes with shallow slip surfaces must account for the nonlinearity of the failure envelope.

In the context of slope stability analysis with shallow slip, where the nonlinearity of the failure envelope has to be take in account, numerous researchers have shown that the failure envelope is actually curved and that a linear approximation can be used only if the range of stresses for which it was estimated is the range of stresses expected in the problem being analyzed (Terzaghi et al., 1996; Mesri and Shahien, 2003; Wright, 2005; Noor and Hadi, 2010; Duncan et al., 2011; Gamez and Stark, 2014).

Therefore, for most geotechnical engineering problems, the shear stress on the failure plane is approximated as a linear function of the normal stress within a selected normal stress range.

In 1776, Coulomb defined the function f(σ) as:

$$\tau = ca + \sigma \tan \delta \tag{2}$$

where:
τ = shear stress [kPa]
σ = normal stress [kPa]
δ = friction angle [°]; and ca = adhesion [kPa]

Coulomb equation (2) is generally referred to the Mohr–Coulomb failure criteria and this linear approximation is known as the Mohr-Coulomb shear strength envelope. The significance of the failure envelope can be explained as follows. Combinations of shear stress and normal stress that fall on the Mohr-Coulomb shear strength envelope indicate that a shear failure occurs. Combinations below the shear strength envelope represent a non-failure state of stress. A state of stress above the envelope cannot exist since shear failure would have already occurred.

The approach to use an equivalent linear envelope may be necessary when the method of slope stability being used requires that shear strengths are represented by a cohesion and friction angle values. Many of the equations used to carry out limit equilibrium stability analysis are based on interface shear strengths defined by a linear Mohr-Coulomb strength envelope and values for c and δ.

Although Test Method ASTM D5321/D5321M and EN ISO 12957-1 call for the testing laboratory to draw a best-fit line through the shear stress-normal stress data and determine c and δ, it is strongly recommended that the design engineer also evaluate the data to determine the appropriate strength parameters to be used in a slope stability analysis.

In the Direct Shear Tests, the shear resistance between different types of geosynthetics or between a geosynthetic and a soil is determined by placing the geosynthetic and one or more contact surfaces, such as soil, within a direct shear box. A constant normal stress representative of field stresses is applied to the specimen, and a tangential (shear) force is applied to the apparatus so that one section of the box moves in relation to the other section.

It is important to note that the reported Mohr-Coulomb parameters only define the shear strength envelope for the range of normal stresses tested. Extrapolation of both friction angle and adhesion outside the range of normal stresses tested may not be representative. For example, extrapolating the failure envelope below the lowest normal stress tested can overestimate shear strength, since the failure envelopes for many geosynthetics interfaces can curve sharply to the origin. Similarly, extrapolating the failure envelope above the highest normal stress tested can overestimate shear strength, since the failure envelope for many geosynthetic interfaces flatten at high loads (ASTM D7702/D7702M).

Therefore, to better fit the linear failure envelope, care must be exercised to estimate the maximum and minimum stresses involved in the analysis, which should be representative and relevant to the design problem analyzed.

Interesting is a discussion of considering an apparent adhesion value in design of structures that incorporate interfaces with a true strength at zero normal stress.

It is common practice in many applications involving soil to ignore cohesion or adhesion values in design. Cohesion values for sands, non-plastic silts, and normally consolidated clays are generally approximated as zero (Lancellotta, 1995; Das, 1990). Although over consolidated clays or cemented sands may exhibit cohesion, engineers often choose to ignore this term because it may not be reliable for long-term conditions. This approach is not recommended for geosynthetic interfaces (Dixon et al., 2006; Koerner et al. 2005; Koerner and Koerner, 2007). The interlocking between geosynthetic and geosynthetic, or geosynthetic and soil, under fixes confined pressure, provide an adhesion or cohesion. This physical interaction between textured surfaces has been justified by experimental evidence carried out by different research (Dixon et al., 2006; Koerner and Narejo, 2005; Koerner and Koerner, 2007). For example, at low normal stress (about <50 kPa), the interaction between nonwoven geotextiles and the textured geomembranes consists of two mechanisms: (i) one is the interlocking (hook and loop) between the superficial filaments of the geotextile and the asperities of the geomembrane, (ii) the other is the friction between the materials. Both take place on a superficial level at interface (Bacas et al. ,2015).

Specifically, those geosynthetic-geosynthetic interfaces and soil-geosynthetic interfaces that experimentally have been shown to exhibit cohesion or adhesion are (ASTM D7702/D7702M, 2021; Koerner and Narejo, 2005):

- textured polyethylene geomembranes (HDPE and LLDPE) vs. geotextiles and soils
- smooth geomembranes (LLDPE and PVC) vs. other geosynthetics and soils
- drainage geocomposites, where geotextiles are thermally bonded to geonets
- geosynthetic clay liner (GCL) internal shear strength, where needle punching provides internal reinforcement of the bentonite layer
- selected geosynthetic-soil interfaces (for example, cohesive soil vs. a nonwoven geotextile) where the interface friction between the two materials is high enough to force the failure plane into the soil

Therefore, if adhesion due to the intercept of least-squares "best fit" straight line representing the linear failure envelope is associated with one of the above interfaces, its use in a stability analysis can be justified (Koerner and Koerner, 2007).

3. A REVIEW OF DIRECT SHEAR AND INCLINED PLANE TESTS METODS

The shear strength of soil-geosynthetic interfaces and geosynthetic-geosynthetic interfaces is a critical design parameter for many civil engineering projects, including, but not limited to waste containment systems, mining applications, dam designs involving geosynthetics, mechanically stabilized earth structures, and reinforced soil slopes, and liquid impoundments.

Since geosynthetic interfaces can be a weak plane on which sliding may occur, shear strengths of these interfaces are needed to assess the stability of soil materials resting above, such as an ore body over a lining system or a final cover on a slope.

Shear resistance at the interface between different ge-

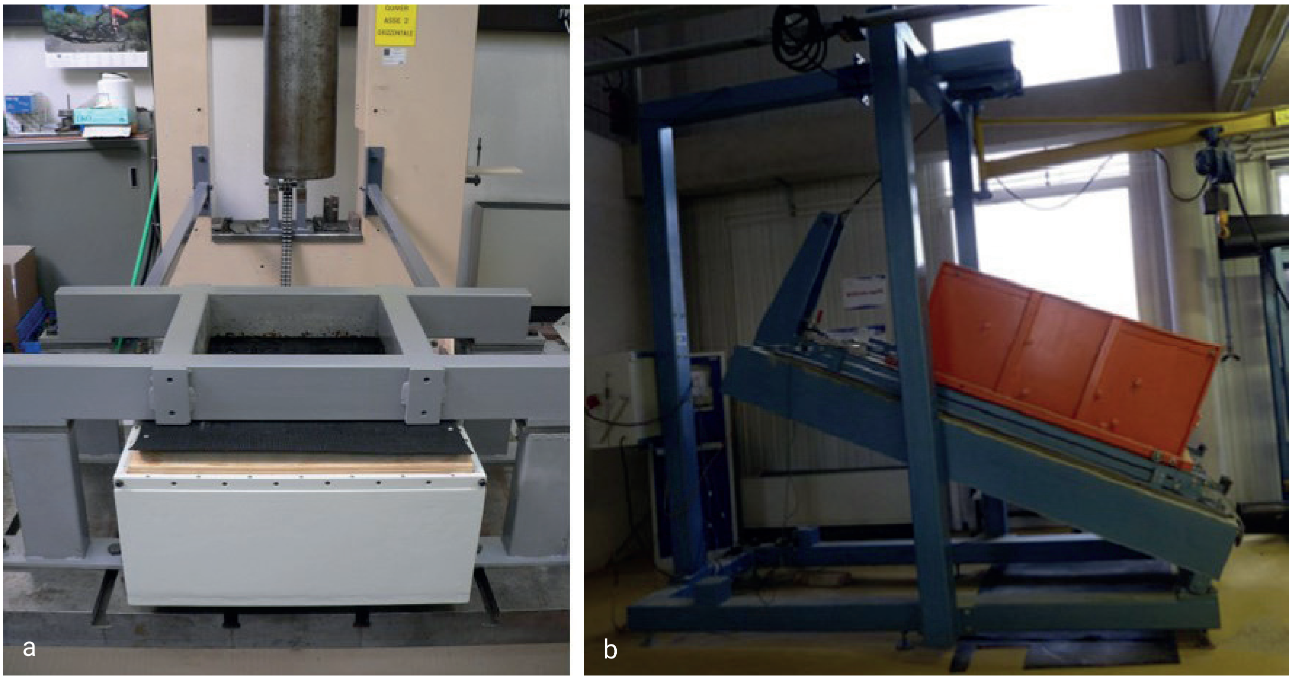

FIGURE 1: Test apparatus: (a) Direct Shear Test; (b) Inclined Plane.

osynthetics or between a geosynthetic and a soil can be measured through laboratory tests. There are three standards, in common use, that provide guidance on testing methods such us EN ISO 12957-1 and ASTM D5321 (direct shear test, Figure 1a) and EN ISO 12957-2 (inclined plane, Figure 1b).

In Table 1 and Table 2, in detail, a summary of the key elements of the above testing methods is reported.

TABLE 1: Key elements of laboratory tests method used to measure interface shear strength - Part 1.

Standard	ASTM D5321/D5321M:2017	EN ISO 12957-1:2018	EN ISO 12957-2:2005
Scope	This test method covers a procedure for determining the shear resistance of a geosynthetic against soil, or a geosynthetic against another geosynthetic, under a constant rate of deformation.	This test method determines the friction characteristics of geosynthetics in contact with a standard sand, or any type of soil or with another geosynthetic under a normal stress and at a constant rate of displacement, using a direct shear apparatus	This European Standard describes a method to determine the friction characteristic in contact with soils, at low normal stress, using an inclining plane apparatus.
Test Apparatus	Square or rectangular containers; minimum dimension 300 mm; 15× the D_{85} of the coarser soil used; minimum of 5× the maximum geosynthetic opening size (in plan); minimum depth of each container 50 mm or 6× the maximum particle size of the coarser soil tested	Minimum internal dimensions of upper box 300 mm × 300 mm; minimum width of upper and lower 50% of their length; both boxes sufficiently deep to accommodate the sand layer and the loading system, or a rigid support to which the upper geosynthetic has to be fixed. For the testing of geogrids at least two full longitudinal ribs and three transverse bars must be contained within the length of both the upper and lower boxes throughout the test.	Rigid base apparatus: Upper soil box: - length ≥ 300 mm; - width ≥ 300 mm; - depth > 7 x D'_{max} > 50mm. Soil filled base apparatus: Upper soil box: -length≥ 300 mm; -width≥ 300 mm; Lower soil box: - length≥400 mm; - width> 325 mm; - depth of both upper and lower box > 7 x D'_{max} > 50mm.
Specific requirement: normal force loading device	Weights, pneumatic or hydraulic bellows or piston, capable of applying and maintaining a constant uniform normal stress for duration of test with accuracy of+2%	A Fluid filled soft membrane or rigid plate ensuring that the normal force is applied uniformly over the whole area of the specimen with an uncertainty of ± 2 %.	A rigid steel plate or a fluid filled soft membrane capable to ensure an even pressure distribution (5 ± 0,1 kPa) with a precision of ± 2%.
Specific requirement: shearing rate	A maximum displacement rate of 5 mm/min for tests on geosynthetics without soil use; A constant rate of shear displacement over a range of at least 6.35 mm/min to 0.025 mm /min, with accuracy of ± 10 %, when soil is included in the test specimen.	A constant rate of share displacement of 1 ± 0,2 mm/min with an uncertainty of ± 2 %. For low permeability soils (D_{10} < 0.0075 mm), shear rates between 0.005 and 1.0 mm/min to ensure drained conditions. The precision must remain ±20% of the selected value. Measurements of the shear force continuously or at intervals of 0,2 mm or 12 s.	Rigid base apparatus: The apparatus fitted with a mechanism which allows the plane to be raised smoothly at a rate of (3 ± 0,5) degrees per minute;

TABLE 2: Key elements of laboratory tests method used to measure interface shear strength - Part 2.

Standard	ASTM D5321/D5321M:2017	EN ISO 12957-1:2018	EN ISO 12957-2:2005
Specific requirement: displacement measurement	LVDTs capable of measuring a displacement of at least 75 mm for shear displacement and 25 mm for vertical displacement with a sensitivity respectively of 0.02 mm and 0.002 mm.	Transducers or dial gauges capable measuring relative displacement shall be measured to a precision of ±0,02 mm. The actual relative displacements continuously or at intervals of 0,2 mm or 12 s.	Measurement of the displacement of the upper box with a precision of ± 0,05 mm. Displacement readings at intervals not exceeding 30 seconds. Measurement of the inclination angle of the table to the horizontal with a precision of ± 0,5 degrees
Number of tests conducted	Minimum of 3 σ_n selected by the user.	Normal pressures: 50 kPa, 100 kPa or 150 kPa Twice test at σ_n = 100 kPa	Normal stress of 5 kPa.
Material conditioning	Temperature of 21 ±2 °C; about humidity control is normally not required for tests on geosynthetics without soil; when soil is included in the test specimen, at a relative humidity between 50 ÷ 70%	In the standard atmosphere for testing: temperature of (20 ± 2 °C and a relative humidity of 65 ±2 %	In the standard atmosphere for testing: temperature of (20 ± 2 °C and a relative humidity of 65 ±2 %
Geosynthetics clamping	Outside the shar are by flat or jaw-like clamping devices.	Geosynthetic clamped at the front part outside the shear area or inside the friction area by gluing or with a standard friction support, e.g., an aluminum oxide abrasive sheet	Geosynthetic fixed to the inclined plane apparatus by stitching or gluing; use of a rough high friction support; anchoring the geosynthetic outside the contact area.
Test geosynthetics specimens	Three specimens in MD and in TD if required. Note: direction and side of the geosynthetic that matches the installation	Four specimens for each direction and for each face to be tested.	Three specimens for each direction and for each face to be tested.
Derivation of shear strength parameters	Mohr-Coulomb failure envelopes defined by best fit straight lines to obtain strength parameters: - δ, the friction angle for peak strength between the two materials - c_a, the adhesion intercept Additionally, shear strength parameters may be calculated at some post-peak condition.	"Best fit regression straight line", through the plot of maximum shear stress to obtain: • ϕ_{sg}, the peak angle of friction between geosynthetic and sand or specific soil, or ϕ_{gg}, the peak angle of friction between geosynthetic and geosynthetic; • c_{sg} (apparent cohesion), the intercept of the line for a geosynthetic–sand test or geosynthetic soil test with the vertical axis or $a_{sg,}$ (apparent adhesion for a geosynthetic –geosynthetic test	The angle of friction for the soil/geosynthetic system is determined by measuring the angle at which a soil filled box (with possible additional weights) slides when the base supporting the geosynthetic is inclined at a constant speed.

4. DIRECT SHEAR TEST APPARATUS AND TESTING PROGRAM

In the last 25 years the authors have coordinated a great number of direct shear tests with different types of geosynthetics and soils (e.g., Cazzuffi and Recalcati, 2018). The test procedure has followed the guideline documents for these testing programs ASTM D5321 and EN ISO 12957-1 using a constant area contact surface.

Tests have been conducted by CESI SpA and by TENAX SpA in their laboratories in Italy, sometimes in a direct partnership, by the mean of a two-axis servo-hydraulic actuator devices.

The two-axis (vertical and horizontal) were applying respectively the vertical (normal) constant stress and the horizontal displacement. The maximum applicable vertical stress is equal to 180 kN in static conditions and 150 kN in dynamic conditions (maximum frequency 5 Hz), while the horizontal axis can apply a maximum force of 62.5 kN and 50 kN, respectively in static and dynamic conditions.

This device is controlled by a digital multi-axis closed loop controller that is both generating the testing waveform and sampling the test results with a frequency of 2 kHz (2000 data/sec).

Typically, every 0.5/1000 sec of testing, the following information have been recorded:

- applied normal load (kN);
- vertical displacement (mm, positive numbers mean dilatancy effects);
- resulting shear load (kN);
- horizontal displacement (mm).

The direct shear apparatus is made of an upper steel frame having inner dimensions of 316.2 x 316.2 mm (0.10 m^2), 100 mm deep, fixed, and a lower box, free to move.

The lower part of the shear apparatus contains the support of the specimen and clamping arrangements to prevent the specimen from slipping during the test. It is sufficiently long and wide to maintain full contact between specimen and the open area of the upper part during the whole duration of the test.

The normal load is applied in the center of the specimen and the horizontal displacement is applied along the interface shear plane.

The normal and the shear stresses are controlled and measured with an accuracy of ±0.1 kN (i.e., 1 kPa).

The horizontal and vertical displacement are recorded with an accuracy of ±0.01 mm and the maximum horizontal displacements can be 50 or 75 mm.

Loading device can apply a horizontal shear force to the shear apparatus at a constant rate of displacement of 1 (± 0,2) mm/min. When the test involves soils with a low permeability (D10 < 0.0075 mm), the shear rate must be determined to ensure the test will be conducted in drained conditions. This may require the use of shear rates much smaller than 1.0 mm/min, i.e. between 0.005 and 1.0 mm/min. When using such shear rate, the precision must remain ±20% of the selected value.

Peak value and the minimum or the steady value after the peak are recorded for every test.

The interface shear strength between six different types of geosynthetics such as geomembranes, geotextiles, geonets, geosynthetic clay liners and geocomposites, and the interface shear strength between themselves in contact with a natural soil (granular and cohesive soils) have been investigated. According to the above testing programs, only the static condition was investigated.

Constant normal stresses usually varying from 10 kPa to 50 kPa were applied to the specimen.

This range of vertical stress have been chosen because representative of field stresses acting on capping landfill lining systems.

5. RESULTS AND DISCUSSION

5.1 Interpretation of direct shear test data

This section presents the testing results of the various interfaces that were described in the previous sections. In all cases, the data is plotted on Mohr-Coulomb stress space, which represents normal stress versus shear strength, resulting from the measured data.

The discussion for using peak, residual, or a combination of these shear strengths for the analysis of geosynthetic-lined slopes and design recommendations for landfill liner and cover systems was presented by Stark and Choi (2003). The authors recommend the stability of landfill cover systems to be analyzed using peak shear strengths with a proper factor of safety, because of the absence of large detrimental shear displacement along with the weakest interface. This recommendation was also confirmed by two and three-dimensional back-analyses of cover failure studies by Stark and Choi (2003) that show that peak interface strengths are mobilized throughout a cover system. This consideration derives from several reasons, including the presence of low shear stresses, low normal stresses (which limit detrimental effects, i.e., damage-inducing, shear displacements to a geosynthetic interface), smaller shear displacements required for stress transfer in soil cover than in MSW, and smaller settlements of the compacted veneer soil compared to MSW. According to the above considerations, the results of direct shear tests has been conducted in term of peak conditions.

For each interface investigated and from the plotting of these individual peak data points, a linear least-squares best-fit response is obtained. This identifies the respective peak values for friction angle and adhesion. It has been possible also to determine the statistical R2-value for every test results.

R2 is the square of the correlation coefficient proportion of the variability of the linear regression model. R2 values range between one and zero. The closer the R2-value is to one, the more accurate is the correlation of the variables involved. An R2-value below 0.3 has no statistical significance.

5.2 Presentation, comparison and discussion of direct shear test data

The main results of a great number of the direct sliding tests coordinated by the authors on different types of geosynthetic and soils, performed according to ASTM D5321 or EN ISO 12957-1, are reported in this paper. The purpose and the focus of this work is to validate and complete the already big database of interface strength measured with direct shear tests by Koerner and Narejo (2005), performed according to ASTM D5321, and to evaluate the differences with the results obtained. This can give to designer the chance to have a critical approach toward the most suitable testing method to be used according to the specific need of a project.

As aforementioned, each interface evaluated were sheared under three different normal stresses, typically varying from 10 kPa to 50 kPa and the corresponding peak shear stresses were obtained. The variation of shear stress versus normal stress for the whole test carried out on the same interface represents a cloud point data in a Mohr-Coulomb stress space. Therefore, the least-squares "best-fit" straight line of the cloud point data, representing the peak failure envelope, can be plotted. Each graph, drawn for each interface investigated, also shows a box where, as additional information, the least-squares "best-fit" straight line equation, the corresponding R2-value, the peak interface friction angle (δ) and the peak apparent adhesion (ca), are reported.

Finally, for some of those interfaces where the comparison between the authors interface strength data and the ones by Koerner and Narejo (2005) has been possible, both results in term of peak failure envelopes are plotted in the same graph (Figure 2 ÷ Figure 8).

Table 3 summaries the whole interface shear strengths results for both geosynthetic/geosynthetic and geosynthetic/soil interfaces studied by the authors and the comparison with the values obtained by Koerner and Narejo (2005).

Before comparing the results obtained by the authors of this paper and those provided by Koerner and Narejo (2005), it is necessary to make some considerations.

All the interfaces investigated by the authors, in this paper, were tested at a constant normal stress varying from 10 kPa to 50 kPa. This range of vertical stress was chosen because representative of field stresses acting on capping landfill lining systems. On the other hand, the data reported by Koerner and Narejo refer to interfaces studied for a much higher confined pressure (usually higher than 150÷300 kPa until 800 kPa, depending on the interface investigated and where itself could be placed (e.g. cover line system or base line system).

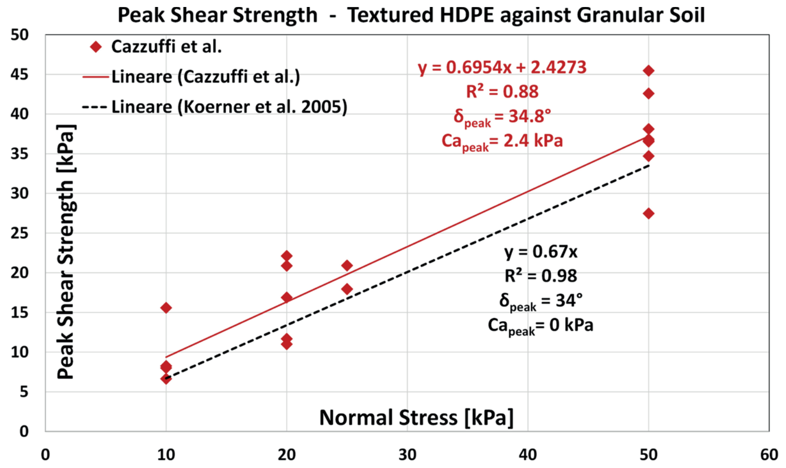

FIGURE 2: Direct Shear Test: Peak Shear Strength - Textured HDPE Geomembrane vs. Granular Soil.

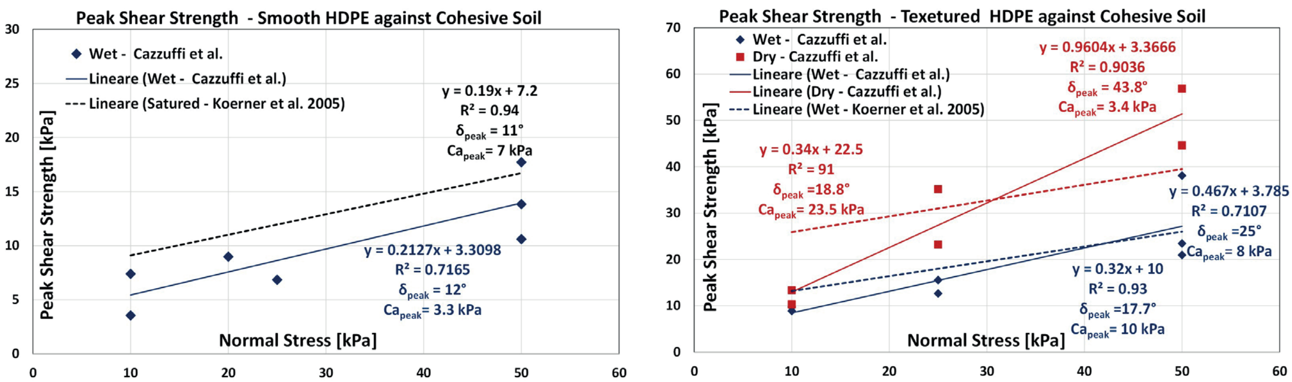

FIGURE 3: Direct Shear Test: Peak Shear Strength - Smooth HDPE Geomembrane and Textured HDPE Geomembrane vs. Cohesive Soil.

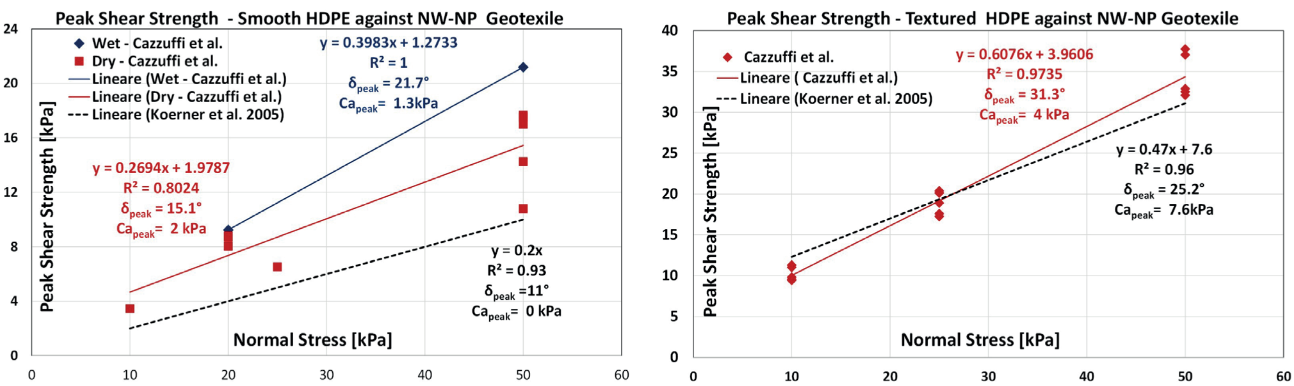

FIGURE 4: Direct Shear Test: Peak Shear Strength - Smooth HDPE Geomembrane and Textured HDPE Geomembrane vs. NW-NP Geotextile.

As far as the moisture test conditions, while the authors conducted tests, for different soil-geosynthetic and geosynthetic-geosynthetic interfaces, both in wet and dry conditions, Koerner and Narejo (2005) conducted the comparison of saturated versus unsaturated conditions only when a geosynthetic is in contact with a cohesive soil. In the other case the humidity conditions are not specified, moisture content is probably the natural one. Furthermore, differently from Koerner and Narejo (2005), the authors of this paper have extensively analyzed the shear strength in those interfaces where the GCD is in contact with soils and with other different geosynthetics.

In the range of normal stress applied (varying from 10 kPa to 50 kPa), from the comparison between interface strength response, the best-fit line through shear stress-normal stress data obtained by the authors (Table 3 and solid line in Figure 2 ÷Figure 8) generally give higher shear strength parameters, both in term of friction angle and adhesion, respect to those provided by Koerner and Narejo (2005, Table 3 and dashed line in Figure 2 ÷Figure

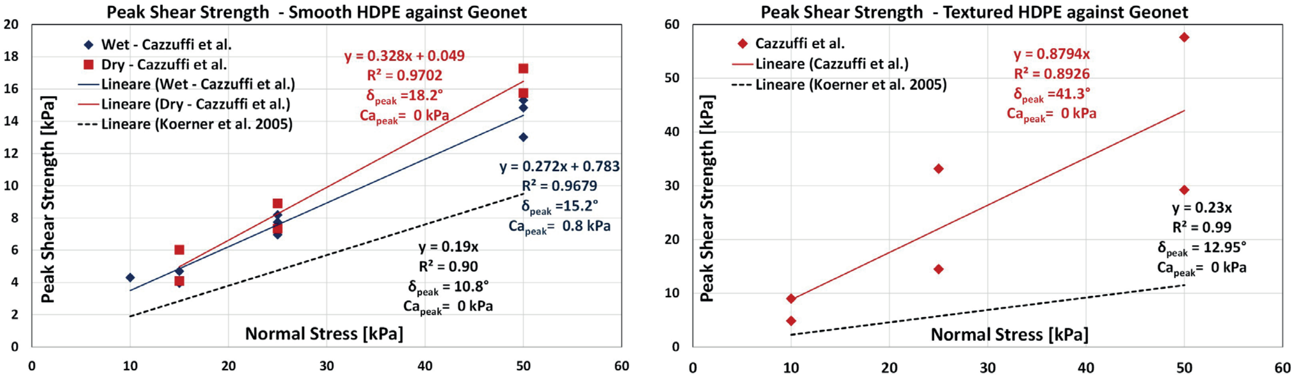

FIGURE 5: Direct Shear Test: Peak Shear Strength - Smooth HDPE Geomembrane and Textured HDPE Geomembrane vs. Geonet.

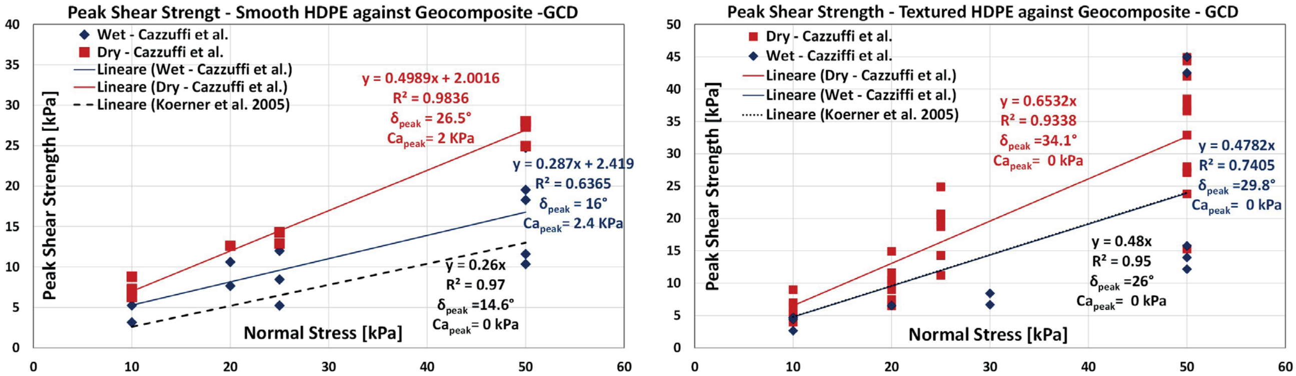

FIGURE 6: Direct Shear Test: Peak Shear Strength - Smooth HDPE Geomembrane and Textured HDPE Geomembrane vs. Drainage Geocomposite - GCD.

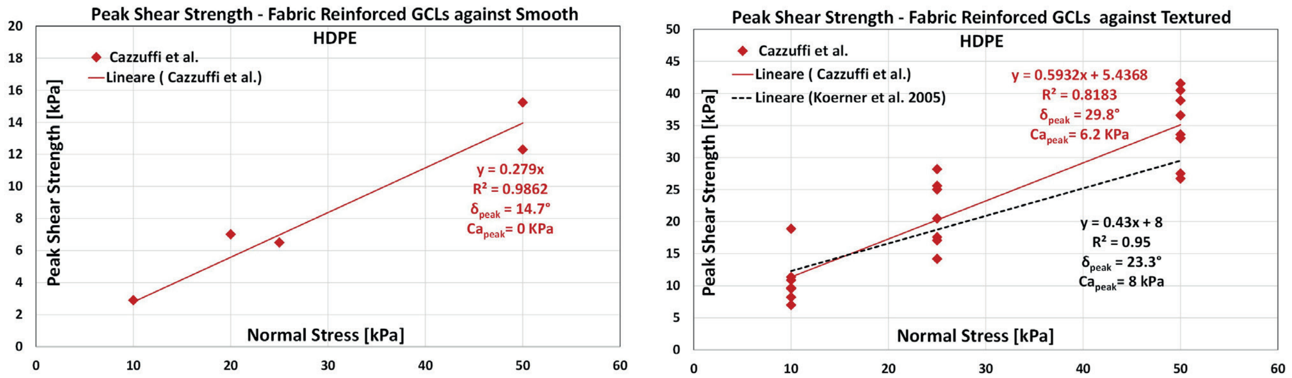

FIGURE 7: Direct Shear Test: Peak Shear Strength - Fabric Reinforced GCL vs. Smooth HDPE Geomembrane and Textured HDPE Geomembrane.

8). This result probably is due to the use by Koerner and Narejo (2005) of a cloud of data corresponding to applied normal stresses much higher than 50 kPa. Therefore, the interpolation of the data gives a reduced inclination of the failure. This flatten trend of the failure envelope for many geosynthetics interfaces (ASTM D7702/D7702M, 2021), at higher stresses, could be explained with particle breakage (when a soil is involved in the test), internal failure of geosynthetic (e.g. GCL), deterioration of the interface (e.g. reduction or rupture in roughness or asperities of textured geosynthetic surface - polishing effect, scratches and even ploughing at interface, degradation of hook and loop mechanism) and reduction in thickness and interlocking capabilities of the geotextile and fiber geosynthetics.

When HDPE geomembrane were tested, it is evident that higher shear strength is observed in textured geomembrane if compared with smooth ones, likely for hook and loop affects due to the greater entanglement between the filaments and the irregular roughness at low normal stress (Bacas et al. ,2015).

For every interface investigated, the dry conditions (Table 3 and red solid line in Figure 2 ÷Figure 8) are more efficient than the wet conditions (Table 3 and blue solid line in Figure 2 ÷Figure 8).

TABLE 3: Direct Shear Tests: summary and comparison of interface shear strengths.

Interface 1	Interface 2	Cazzuffi et al.			Koerner and Narejo (2005)		
		Peak Strength			Peak Strength		
		δ	c_a	R^2	δ	c_a	R^2
		[°]	[kPa]	[-]	[°]	[kPa]	[-]
HDPE-S	Granular Soil				21	0	0,93
HDPE-S	Wet cohesive soil	12	3,3	0,72	11	7	0,94
HDPE-S	Dry cohesive soil				22	0	0,93
HDPE-S	NW-NP GT Wet	21,7	1,3	1,00	11*	0*	0,93*
HDPE-S	NW-NP GT Dry	15.1	2	0,80			
HDPE-S	Geonet						
HDPE-S	Geonet Wet	15,2	0,8	0,97	11*	0*	0,90*
HDPE-S	Geonet Dry	18,2	0	0,97			
HDPE-S	GCD						
HDPE-S	GCD Wet	16	2,4	0,64	15*	0*	0,97*
HDPE-S	GCD Dry	26,5	2	0,98			
HDPE-T	Granular Soil	34,8	2,4	0,88	34	0	0,98
HDPE-T	Cohesive Soil Wet	25	8	0,72	18	10	0,93
HDPE-T	Cohesive Soil Dry	43,8	3,4	0,91	19	23	0,91
HDPE-T	NW-NP GT	31,3	4	0,97	25	8	0,96
HDPE-T	Geonet	41,3	0	0,90	13	0	0,99
HDPE-T	GCD Wet	29,8	0	0,74	26*	0*	0,95*
HDPE-T	GCD Dry	34,1	0	0,93			
NW-NP GT	Granular Soil	44,5	0	0,88	33	0	0,97
GCL	Granular Soil	29,8	2,4	0,98			
GCL	Cohesive Soil	27,4	4	0,97			
GCL	HDPE-S	14,7	0	0,99			
GCL	HDPE-T	29,8	6,2	0,82	23	8	0,95
GCL	NW-NP GT	29,5	3,6	0,77			
GCL	Geonet	32,5	2,7	0,97			
GCL	GCD	20,7	3,1	0,78			
GCD	Granular Soil	34,1	0	0,99	27	14	0,86

The test humidity conditions are not specified

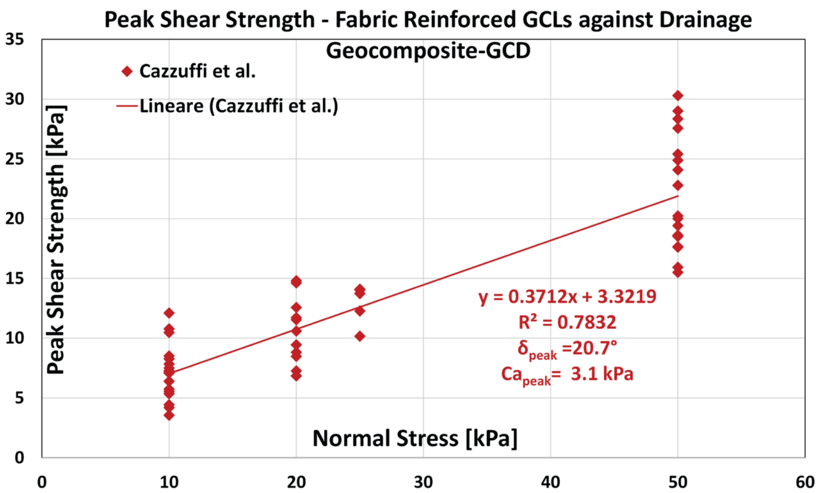

FIGURE 8: Direct Shear Test: Peak Shear Strength - Fabric Reinforced GCL vs. Drainage Geocomposite - GCD.

5.3 Comparison of shear tests data with the results obtained by tilting table tests.

As shown in Figure 2 ÷Figure 8, the use of a direct shear tests allows to define through a linear regression an envelope defining an interface friction angle and an adhesion which is not representing the real Mohr-Coulomb envelope that should be, as aforementioned, curved. The standard pressure used to carry out inclined plane tests according to EN ISO 12957-2 is 5 kPa. This confining pressure can hardly be applied with direct shear apparatus, as at low pressures it is difficult to control the state of stress while the lower box is moving.

The use of tilting table is not yet diffused due to the limitation in the pressure and to some concerns related to the clear identification of the angle that triggers the movement. As shown in Figure 9, depending on the type of materials, the movement (sliding) can activate during the change of the angle in a continuous way or can start in a sudden way.

However, when the limiting angle is well defined the test allows to have very useful information.

In order to evaluate interface tests performed under different states of stress, with different conditions (dry and wet) and with both the test methods, the results for specific interfaces have been assembled and summarized (Figure 10, Cazzuffi et Recalcati, 2018). The interface frictional properties of a traditional GCD and an innovative bi-component GCD in contact with a smooth and textured and a textured geomembranes are shown.

For each interface investigated it was possible to design a bilateral envelope with a clear change in the curve slope passing from the inclined plane test data (EN ISO 12957-2, plotted for confining pressures lower than 5 kPa) to the direct shear test data (EN ISO 12957-1 and ASTM D5321, plotted for confining pressure higher than 10 kPa). Such change in the slope (knee) is due the fact that the tilting table test measures only the limiting angle above which the sliding of the upper box starts the movement, while the shear box test measures, at the interface and for a fixed confine pressure, both the friction angle and the adhesion.

Regarding the tested interfaces, it is evident that the critical interface is, usually, between a smooth geomembrane and a traditional drainage geocomposite, therefore a valid alternative to solve stability problems would be the use of a textured geomembrane instead of the smooth one. This solution is effective but more expensive; furthermore, a textured geomembrane can create difficulties during welding of adjacent rolls in singular points and when the surface is not linear but curve.

Therefore, one of goals to improve the veneer stability consists in improving the interaction characteristics between the layers of the capping system, emphasizing the use of innovative products.

The improvements in extrusion technology in the last years have created more accurate and precise extrusion heads capable to control fluxes of different polymers and then producing geosynthetics with different polymers co-extruded. Bi-component innovative products, that are products with a geonet consisting in extrusion of two different polymers (EVA - Ethyl Vinyl Acetate - co-extruded on a HDPE geonet on the side in direct contact with the liner), are a good solution allowing to improve in a significant way the frictional properties (as is evident in Figure 10 the coupling between bi-component GCD - smooth geomembrane is, generally, more performing if compared with the one between a traditional GCD - textured geomembrane).

Finally, in Figure 10 the effect of water is emphasized. For both tests wet condition was obtained by spraying water over the geomembrane thus creating a thin slippery film reducing the shear resistance at the interface, particularly at low pressures. Under low normal stress (5 kPa) the package bi-component GCD - smooth membrane shows a shear resistance lower than traditional GCD - textured geomembrane. When the normal stress increases, the water under the geosynthetic ribs is squeezed apart, the contact between the geonet and the underlaying liner becomes dry and the shear resistance increases to a level higher than the one between traditional GCD - textured geomembrane.

The state of stress that shows the change in the curve is 10 kPa, corresponding to the weight of 0.50 m of topsoil. This result means that tilting table is particularly valid at low pressures, instead, for higher pressures, according to the European Directive that suggests the topsoil should be at least 1.00 m that corresponding to a surcharge between 15 and 20 kPa, the direct shear test is a suitable testing method.

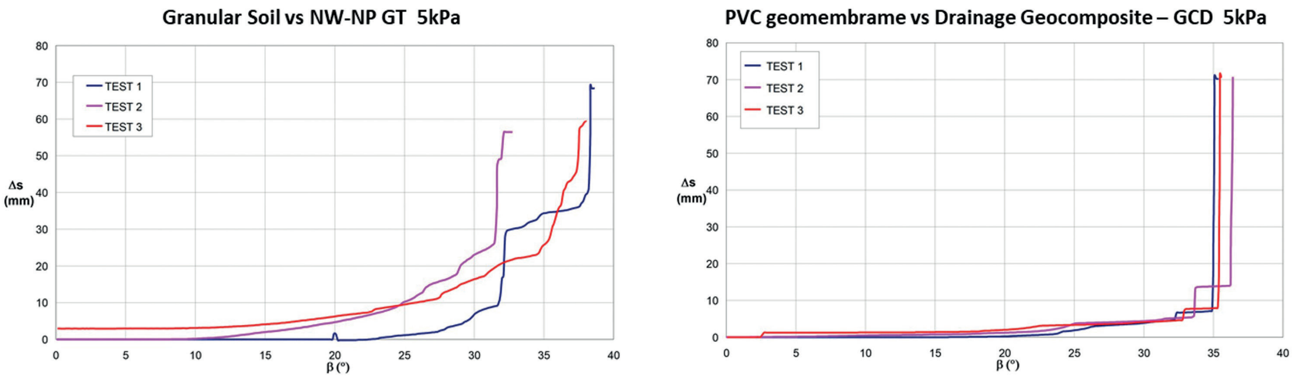

FIGURE 9: Inclined Plane Test – Example of test results at different interfaces: (a) Granular Soil vs NW-NP GT; (b) PVC geomembrane vs drainage geocomposite - GCD.

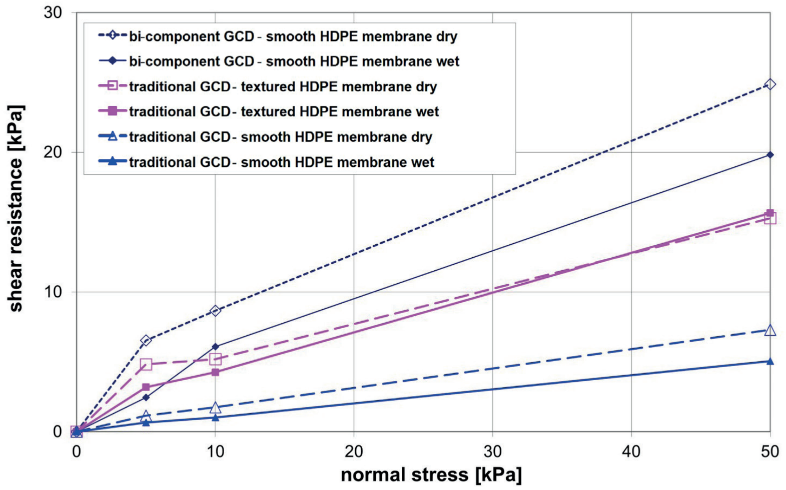

FIGURE 10: Interface tests between traditional and innovative bi-component GCDs with different geomembranes: for σ'_v = 5 kPa - inclined plane test results; for σ'_v > 10 kPa: shear direct test results.

6. CONCLUSIONS

Evaluation of the correct shear strength parameters among the different layers of a landfill capping (geosynthetics and soils) is essential for design, construction and maintenance of such structures. The available testing methods (i.e. direct shear and inclined plane) allow to estimate in a correct way those parameters, provided that the correct boundary conditions are taken into account. Whenever it is not possible to run a specific test analysis for a selected stratigraphy, the use of the results present in bibliography can be very useful for a preliminary design.

For each interface studied, boundary conditions (in term of normal stress and humidity) are the real discriminators: differences on interface strength parameters measured by the authors respect to those obtained by Koerner and Narejo (2005) can be found.

It is important to point out that a failure envelope representative of the real shear strength between different types of geosynthetics or between a geosynthetic and a soil, should be obtained only if the range of stresses, for which it was estimated, is the one expected in the problem being analyzed. For this reason, all interfaces investigated by the authors were tested under a confining pressure varying from 10 kPa to 50 kPa. This range of vertical stress was chosen because representative of field stresses acting on capping landfill lining systems.

In the range of normal stress applied (varying from 10 kPa to 50 kPa), the comparison between interface friction angle and adhesion/cohesion measured by the authors are, generally, higher than those provided by Koerner and Narejo (2005). The reason of this difference is probably the different range of stress for the cloud of data measured by Koerner and Narejo (2005), usually higher than 150-300 kPa; therefore, the interpolation of the data gives a reduced inclination of the failure envelope, at these higher confining pressures could be explained with particle breakage (when a soil is involved in the test), internal failure of geosynthetic (e.g. GCL), deterioration of the interface (e.g. reduction or rupture in roughness or asperities of textured geosynthetic surface - polishing effect, scratches and even ploughing at interface, degradation of hook and loop mechanism) and reduction in thickness and interlocking capabilities of the geotextile and fiber geosynthetics

Regarding to the moisture conditions, while the authors focus on considering all the possible moisture condition, wet or dry, in which all the linear layers of a capping system (soil-geosynthetic and geosynthetic-geosynthetic interfaces) can be found, Koerner and Narejo (2005) conducted the comparison of saturated versus unsaturated conditions only when a geosynthetic is in contact with a cohesive soil. In the other case the humidity conditions are not specified (moisture content is probably the natural one). It is important to emphasize how the presence of a thin film of water at geosynthetic-geosynthetic interfaces would lead to a significant reduction in the shear strength parameter due to its lubrification effect.

Furthermore, it is important to note that inclined plane test (that simulates in a proper way the geometry of the capping) has a limitation in the low normal stress applied (5 kPa).

The combination of laboratory tests conducted with different test method at different pressure allows to draw a bilateral envelope with a clear change in the curve slope passing from the inclined plane test data (EN ISO 12957-2, plotted for confining pressures lower than 5 kPa) to the direct shear test data (EN ISO 12957-1 and ASTM D5321, plotted for confining pressure higher than 10 kPa). Such change in the slope (knee) is due the fact that the tilting table test measures only the limiting angle above which the sliding of the upper box starts the movement, while the shear box test measures, at the interface and for a fixed confine pressure, both the friction angle and the adhesion.

Finally, this paper provides a useful and practical application for both researches and practitioners who use these materials in the field, helping them to make a decision about the type of geosynthetic to choose in a particular boundary conditions.

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