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A numerical modelling technique for geosynthetics validated on a cavity model test

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ABSTRACT

Numerical modelling approaches can aid in designing geotechnical constructions involving geosynthetics. However, the reliability of numerical results depends on how the model is developed, the constitutive model, and the set of parameters used. By comparing the numerical results with experiment, the present work verifies a numerical modelling technique developed to model multilayered geosynthetic lining systems for landfills. The numerical modelling technique involves strain softening at interfaces and allows the axial stiffness of the geosynthetics to evolve as a function of strain. This work focuses on a two-dimensional finite-difference model, which is used to simulate three types of experimental tests: conventional uniaxial tensile tests, direct shear tests, and a large-scale test that was used to assess the overall mechanical behaviour of a reinforced geosynthetic system that spanned over a cavity. This reinforced geosynthetic system consisted of a 50 kN/m polyvinyl alcohol geogrid reinforcement embedded in a layer of sand, a geosynthetic clay liner, a high-density polyethylene geomembrane, and a non-woven needle-punched geotextile. The uniaxial tensile tests, direct shear tests, and the large-scale test were numerically modelled and the numerical results were compared with experimental results. The results of the numerical modelling technique presented very closely match the results of the three experimental tests, which indicates that the numerical model correctly predicted the measured data.

Keywords: Geosynthetics, numerical modelling, tensile test, direct shear test, large-scale test, strains, shear displacements

43 NOTATIONS

44	K_s	Interface shear stiffness
45	K_n	Interface normal stiffness
46	c	Cohesion
47	ϕ_{peak}	Peak friction angle
48	ϕ_{res}	Residual friction angle
49	γ	Sand unit weight
50	E	Sand elastic modulus
51	ν	Sand Poisson ratio
52	D_{min}	Minimum diameter of sand particles
53	D_{max}	Maximum diameter of sand particles

54

55 ABBREVIATIONS

56	GMB	Geomembrane
57	GGR	Geogrid
58	GTX	Geotextile
59	GCL	Geosynthetic clay liner
60	PVA	Polyvinyl alcohol
61	HDPE	High-density polyethylene
62	UTT	Uniaxial tensile test
63	DST	Direct shear test
64	LSTA	Large-scale test apparatus

65

1. INTRODUCTION

Geotechnical constructions that involve geosynthetics, such as landfills, are traditionally designed by using limit equilibrium methods (Giroud, 1989, Koerner and Hwu, 1991). However, these methods cannot be used to assess the integrity (e.g., strain or tensile forces) (Long et al., 1994) of the construction components and do not consider whether stresses are compatible with strains and displacements (Villard et al., 1999). As an alternative, such constructions may be designed by using numerical modelling methods (Fowmes et al., 2008); these methods not only account for the above-mentioned aspects but also account for the multiple interactions between geosynthetics.

Numerical modelling techniques are becoming ever more sophisticated because today's software allows designers to consider the key aspects of the mechanical characteristics of geosynthetics (e.g., the nonlinear stiffness) and of the interfaces (e.g., strain softening). However, the reliability of such numerical results depends on the numerical modelling technique used, which in turn rests upon how the model is developed, the constitutive model, and the set of parameters used.

Whichever numerical modelling technique is used, questions exist with respect to (i) the relevance of the numerical modelling technique and therefore (ii) the reliability of the numerical results. Consequently, to answer such questions, numerical results should be confirmed by comparing them with experimental data. In the context of landfills, and particularly for piggy-back landfill expansions where a new landfill is built over an older one, such verification is essential because of the interactions between the various materials, such as clay, sand, gravel, geosynthetic, and waste (Tano and Olivier, 2014).

Unfortunately, limited studies that addressed the comparison of the experimental behaviour of multilayered geosynthetic lining systems with that predicted by numerical models are available (Fowmes et al., 2008). To the best of our knowledge, only three studies (Villard et al. (1999), Fowmes et al. (2008) and Zamara et al. (2014)) compare experimental results of multilayered geosynthetics with numerical results of models of landfill lining systems.

This work aims to verify a numerical modelling technique by comparing it with three experimental tests: a tensile test, a direct-shear test, and a large-scale large test. These tests were developed to assess the mechanical behaviour of a reinforced geosynthetic lining system.

Prior to discussing the details of the verification process, the previous studies of Villard et al. (1999), Fowmes et al. (2008) and Zamara et al. (2014) are further discussed in the following section. The benefits and limitations of these studies provide a framework for the present study and lead us to develop a new modelling technique.

2. BACKGROUND

Villard et al. (1999) applied finite-element modelling to describe a veneer cover of a landfill and to better understand the distribution of forces and strains within a geotextile (GTX) and geomembrane (GMB) placed at the bottom and on side slopes of the landfill. The forces within the GTX were measured by force sensors positioned at the top of the slope. A cable-type displacement (extensometer wires) was used to measure the geosynthetic displacements and then the strains were calculated from the differential displacements obtained between two consecutive measurement marks on the geosynthetic. The numerical model used the Mohr–Coulomb failure criterion with single shear strength to model the interfaces and assumed a linear elastic behaviour

for the various geosynthetics. The numerical results (displacements, strains and tensions) were then compared with the measured results obtained at the instrumented site.

Although Villard et al. (1999) considered that their model produced results that were, on the whole, satisfactorily consistent with experiment, discrepancies appeared between trends and between the strains, forces, and displacements obtained numerically and by experiment. As an example, the strains calculated by applying the numerical model to the section of slope where the GTX was not covered by the granular layer were constant whereas the experimentally determined strains increased along this slope. The authors attribute these discrepancies to differences in displacement measurements. Moreover, at the top of the geosynthetic, the simulated displacements differed from the measured displacements. Villard et al. (1999) concluded that these differences were probably due to the system that fastened the geosynthetic to the top of the slope interfering with the measurements. In addition, the calculated and measured strains at the nonloaded part of the slope differed significantly, which the authors attributed to faulty operation of the measuring devices or to insufficient stabilization time.

Fowmes et al. (2008) used a finite-difference method to numerically model the mechanical behaviour of a two-layered geosynthetic (GTX and GMB) lining system subjected to down-drag forces from synthetic waste in a large-scale test. The relative displacement of the GMB and of the GTX was measured by using extensometer wires, and the forces within the GMB were measured by using tensile load cells. The numerical model used the Mohr–Coulomb failure criterion with a displacement-dependent limiting shear strength (i.e., strain softening) to model the behaviour of the interfaces between the geosynthetics. Fowmes et al. (2008) used a constant 2% strain secant modulus to model the various geosynthetics. However, they noticed that the use of a 2% modulus

in the design could lead to an overestimate of the material stiffness for strains in excess of 2%. To verify the numerical modelling technique, the results of the numerical calculations (strains and relative shear displacements at interfaces) were compared with measurements.

Fowmes et al. (2008) concluded that some discrepancies existed between the measured and the numerically modelled results, which they attributed to simplifications in modelling the axial behaviour of the geosynthetics and in the constitutive model used to represent the synthetic waste. In fact, the displacements of the GMB were underestimated by the numerical model, as revealed by comparison with the large-scale test. This underestimation thus led to the underestimation of the extension of the GMB. They attributed this discrepancy to the fact that the 2% strain secant modulus used underestimated the tensile strength at the relevant strains in their investigation. Overall, Fowmes et al. (2008) concluded that their numerical model (strain-softening characteristics of interfaces) and application methodology were valid.

More recently, Zamara et al. (2014) used the numerical model developed by Fowmes et al. (2008) for strain-softening behaviour at interfaces to investigate the mechanical performance of a multilayer-lining (GTX and GMB) system on a landfill slope. This site was monitored, which allowed the measured displacements and strains of the geosynthetics to be compared with the numerical results.

Zamara et al. (2014) claim that their model replicates the behaviour of the GTX at the lower part of the slope. However, the correlation between (i) the measured displacements and strains and (ii) the modelled displacements and strains was faulty. For example, the numerical model gave compressive strains for the toe section throughout the cell-filling stages, whereas tension in this section was measured on site. The authors concluded that this discrepancy is related to significant displacement of the GMB at the toe of the slope due to the presence of a compressible shredded-

tire basal drainage layer at the toe of the slope. Zamara et al. (2014) also numerically modeled the lining system of the landfill slope by using a basic approach that does not account for the strain-softening behaviour of interfaces. The results were inconsistent with the measurements, especially for the GTX displacements. They also reported that the geotextile did not deform in a manner modelled. Overall, Zamara et al. (2014) recognized that the verification of the numerical modelling technique that they used would benefit from more monitoring data.

None of the previous studies consider the nonlinear axial stiffness of the geosynthetics in modelling the interactions between these geosynthetics in the lining system. However, the stress-strain response of geosynthetics is typically nonlinear (Giroud, 1994, Wesseloo et al., 2004, Tano et al., 2016a), which means that the axial stiffness (e.g., the modulus) of the geosynthetic evolves as a function of strain. Thus, neglecting the true characteristics of the geosynthetics could lead to inaccuracies and discrepancies, as exemplified in the studies just mentioned.

For a more rigorous numerical analysis, Tano et al. (2016a) proposed a numerical modelling technique based on the following five criteria: all geosynthetic interfaces in the lining system are modelled [criterion 1 (CR1)], the staged construction or evolution of the waste properties depends on depth or stress (CR2), the strain softening occurs at interfaces between geosynthetics (CR3), geosynthetic compressive characteristics differ from the tensile characteristics (CR4), and the geosynthetic has nonlinear axial stiffness (CR5). However, Tano et al. (2016a) did not compare their numerical results with experimental results.

To correct this shortcoming, the present work verifies this numerical modelling technique by comparing its results with experiment.

These experimental tests include tensile tests and direct-shear tests, which are treated as index tests throughout this paper. The numerical results are also compared with the results of a large-scale (2 m × 1 m × 0.5 m) test that was developed to assess the mechanical behaviour of a multilayered-lining system that spans over a cavity. This lining system includes a polyvinyl alcohol geogrid (PVA GGR) reinforcement embedded in a sand layer, a geosynthetic clay liner, a high density polyethylene (HDPE) GMB, and a nonwoven needle-punched GTX. For numerical modelling, a two-dimensional (2D) finite-difference program was used; however, CR2 was not addressed in this study because the experiments did not include waste.

In the following sections, the three testing devices (tensile test, direct-shear test, and large-scale test) and the various materials used and their properties are presented. Next, the numerical modelling technique is fully described, following which the experimental results are compared with numerical results.

3. TESTING DEVICES

3.1 Uniaxial tensile test device

The UTT is a simple test used to assess tensile behaviour of a material. Geosynthetic manufacturers generally systematically subject all products to UTTs to determine their quality. However, complete tensile curves are rarely provided. Therefore, the authors performed additional tensile tests to obtain the complete tensile curves. In these tests, a geosynthetic specimen is subjected to a constant tensile deformation rate along a given axis and the resulting forces and strains within the geosynthetic are recorded. Prior to the test, the specimens prepared

according to the standard NF EN ISO 9862 (AFNOR, 2005a) were clamped in the tension machine grips, as shown in Figure 1. For reliable measurements, five specimens of each geosynthetic were subjected to uniaxial tests. The tensile tests used a strain rate of 10%/min., and the strains and forces were measured by using a video-camera extensometer and a force sensor within the upper tension jaw, respectively.

All tests on GMB specimens were performed according to the standard NF EN ISO 10319 (AFNOR, 2008) and the tests on GTX, GCL, and GGR specimens were performed according to the standard NF EN 12311-2 (AFNOR, 2013).

3.2 Direct shear test device

The DST is a common laboratory test used to determine the shear-strength parameters of interfaces and materials. The DST device measures the shear strength under high confining stress, typically up to 150 kPa. In the present study, the authors used a 0,3 m × 0,3 m test device (Figure 1b). Three specimens of each material were tested under three normal stresses (50, 100, and 150 kPa). To assess the shear strength of interfaces between geosynthetics, the first geosynthetic sample was fixed to a horizontal plate, and the second geosynthetic sample was mounted on the lower face of the test box filled with sand. Because the GGR is meshed, the procedure as previously described may lead to unreliable measurements if used to assess the shear characteristics of the contact between GGRs and mineral materials (e.g., sand). In fact, the shear strength of the GGR-sand interface is due to several addition mechanisms, such as the frictional resistance at the ribs and nodes, the bearing resistance at transverse ribs, and the interlocking of aggregates (Jewell et al., 1984).

Here, the procedure used for the GGR was modified to consider such mechanisms. A three-dimensional geocomposite filled with sand was added at the horizontal plate, so the transverse ribs should provide bearing resistance and the aggregated should interlock. Finally, a shear rate of 1 mm/min. was imposed on the sample during the tests by using the jack, as shown in Figure 2.

3.3 Large-scale test apparatus to simulate cavity

This work used a LSTA developed to simulate the mechanical characteristics of a reinforced geosynthetic system that spans over a cavity. The LSTA is parallelepiped in shape (Figure 3) and has internal dimensions of 2.0 m $\times$ 1.0 m $\times$ 0.5 m. The principle of this large-scale test consists of applying a spatially uniform pressure over a reinforced geosynthetic system that bridges a 0.5-m-wide rectangular cavity.

The reinforced geosynthetic system is considered to reasonably represent the piggyback landfill-lining system. This lining system often includes, from top to bottom, a GTX, a GMB, a GCL, and a GGR reinforcement (Tano et al., 2015) embedded in a dense material. Sand was also added within the LSTA as a subgrade for the lining system and to simulate the mineral drainage layer generally installed above the GTX. The full properties of these geosynthetics and materials are presented in the following section.

A vertical load was applied by filling a flexible bladder with water. The pressure applied at the top of the LSTA via the bladder was controlled by a microprocessor-controlled pressure gauge. Foil microstrain gauges (MSGs) with a linear pattern (EP-8-40CBY-120) were also used to measure strains within the GGR and the GMB. The effect of the adhesive on the tensile strength

of these geosynthetics was assessed through a rigorous calibration procedure, and all raw MSG measurements were corrected for strains without the influence of the adhesive.

4. MATERIAL, GEOSYNTHETIC AND INTERFACE PROPERTIES

This study considers a lining system made of a GTX, a GMB, a geosynthetic clay liner (GCL), and a GGR reinforcement embedded in a dense material. Sand was used to simulate this dense material. The full properties of these geosynthetics, materials, and of the interfaces between the various materials are presented below.

4.1 Sand

Hostun sand (Girout et al., 2014, Lajevardi et al., 2014) was used as a subgrade for the lining system and to simulate the drainage layer above the GTX in the large-scale test apparatus (LSTA). This sand layer also ensured that the load was uniformly distributed at the top surface of the LSTA. The sand was dried and sieved to obtain a particle-size distribution ranging from 0.6 to 1.6 mm. Direct shear tests (DSTs; see Sec. 3.2) performed on the sand under normal stresses of 50, 100, and 150 kPa, reveal a peak (residual) friction angle of 35.2° (33.8°) and zero cohesion. The properties of the Hostun sand used in the LSTA are grouped in These properties are the unit weight γ , the elastic modulus E , the Poisson ratio ν , the cohesion c , the peak friction angle ϕ_{peak} , the residual friction angle ϕ_{res} , and the minimum and maximum particle diameters (D_{min} and D_{max}). In situ density tests were also conducted within the LSTA. This led to an in-situ mean unit weight of 14 kN/m^3 . As the elastic modulus of uniform medium sand can vary from 30 to 50

MPa (Kezdi, 1974, Prat et al., 1995, Obrzud and Truty, 2012), a low value of 30 MPa was considered in this study since the sand was laid without specific compaction. As regards the Poisson ration ν , Hicher (1998) shows that a value of 0.23 could be considered for a well graded Hostun sand. In this study, a value of 0.25 close to 0.23 was considered. Finally $D_{\min}$ and $D_{\max}$ was determined using the standard NF P94-056 (AFNOR, 1996).

4.2 Geosynthetics

For this study, four different geosynthetics were used for the experiments. These are, from top to bottom, a nonwoven polypropylene GTX, a HDPE GMB, an un-hydrated sodium bentonite GCL, and a PVA GGR.

Five uniaxial tensile tests (UTTs; see Sec. 3.1 and Figure 1a) were performed on each of the various geosynthetics. All these tests were performed on-air without confining loads and at a constant temperature of 22°C in a temperature controlled room (laboratory) as required by the existing standards. Rectangular specimens (0.05 m × 0.2 m) were used for the GTX and GCL according to the international standard NF EN ISO 10319 (AFNOR, 2008). Larger specimens (0.2 m × 0.1 m) were used for the GGR because of their ribs and nodes. For the GMB, the standard NF EN 12311-2 (AFNOR, 2013) was used to assess the uniaxial tensile behaviour. The full tensile curves of the four geosynthetics are given in Figure 4a (GMB and GGR) and Figure 4b (GCL and GTX). In Figure 4, data relating to the numerical simulations will be further discussed in section 6.1.

To take into account for the non-linear stress-strain behaviour of the geosynthetics, a user-coded piecewise “modulus-strain” relationship was used so that the modulus of the geosynthetic can

vary with strain. These tensile axial moduli derived from the full tensile curves of the various geosynthetics are shown in Figure 4. These sets of parameter were defined based on the tangent modulus at each strain level. In addition, Table 1 summarizes the main properties of these geosynthetics. These properties concern the thickness, the mass per unit area, the machine tensile strength, the strain as a function of tensile strength, and the secant modulus at 1% strain. The thickness was determined using the standard NF EN 1849-1/2 (AFNOR, 2010) for the GMB and the standard NF EN ISO 9863-1 (AFNOR, 2016a) for the GTX and other geosynthetics. As regards the mass per unit area, the standards NF EN 1849-1/2 (AFNOR, 2010), NF EN 14196 (AFNOR, 2016b) and NF EN ISO 9864 (AFNOR, 2005b) were used for the GMB, the GCL and the GTX and the GGR respectively.

4.3 Interfaces

Because the LSTA included four geosynthetics, interaction between these geosynthetics occurred across the following six interfaces:

- Interface I1: between the sand layer at the top of the LSTA and the underlying GTX;
- Interface I2: between the GTX and the underlying GMB;
- Interface I3: between the GMB and the underlying GCL;
- Interface I4: between the overlying GCL and the sand layer at bottom of the LSTA, which is assumed to be a subgrade layer;
- Interfaces I5 and I6: the upper and lower interface, respectively, between the GGR and the sand layer in which it was located.

Furthermore, the shear strengths of these interfaces were assessed by using DSTs (see Sec. 3.2 and Figure 2). As previously discussed, the DST procedure used for the GGR-sand interfaces was modified to account for the bearing resistance at the transverse ribs and the interlocking of aggregates.

Figure 6 shows the shear stress versus displacement curves obtained at 50, 100 and 150 kPa of normal stresses for the six interfaces I1, I2, I3, I4, I5 and I6. In Figure 6, data relating to the numerical simulations will be further discussed in section 6.2.

Table 1 summarizes all the properties of the interfaces derived from these the shear stress versus displacement curves. These properties are shear stiffness K_s , normal stiffness K_n , cohesion c , peak ϕ_{peak} , and residual friction angle ϕ_{res} . K_s was calculated as the average of the three slopes of the initial part of the three shear stress versus displacement curves at 50, 100 and 150 kPa of normal stresses. The initial part of the shear stress versus displacement curves concerned the displacements between 0 mm and the peak value. A constant value of K_s was used for each interface. Before the peak, the shear displacements are directly controlled by K_s and the incremental relative shear stresses. Then the Mohr-Coulomb shear-strength criterion limits the shear stress applied to each interface.

Concerning K_n , as recommended by Itasca (2005) and Fowmes et al. (2008), an arbitrarily large value was considered to avoid node interpenetration during computation. A value of 100 MPa was considered following previous studies (Tano et al., 2016a, Tano et al., 2016b). Moreover, zero cohesion was assigned to all interfaces because the geosynthetic interfaces are assumed to have zero shear resistance when there is no confining pressure.

Figure 7a (I4, I5, and I6) and Figure 7b (I2 and I3) show the evolution of the mobilized friction angle (i.e., shear strength) that was considered for the

numerical modelling. The strain softening behaviour of interfaces was simulated using displacement-dependant limiting shear strength values. During computation, the shear strength was selected from piecewise tabulated relationships (e.g. friction angle values versus displacements relationship plotted in Figure 7 c and Figure 7d). The peak and residual values of the friction angles are given in Table 1.

The following section gives the details of the numerical modelling technique used in this study.

5. NUMERICAL MODELLING TECHNIQUE

As mentioned in section 2, Tano et al. (2016a) proposed a numerical model technique to simulate the behaviour of a multilayered geosynthetic lining system in the context of piggy-back landfill expansions. This technique was based on five criteria (CR1 to CR5). In the present study, the authors use this same numerical modelling technique (Tano et al., 2016a) to model the three experimental tests (UTT, DST, and LSTA). However, CR2 is not considered herein because none of the three experimental tests included waste. Moreover, even were CR3 considered, it is of no importance in this study because all the three experimental tests led to an overall tension (not compression) in the geosynthetics.

The numerical modelling of these three experimental tests was implemented by using the 2D finite-difference program Fast Lagrangian Analysis of Continua (FLAC 2D V.7). This program allows users to consider multilayered geosynthetic interactions while modelling structural elements and interfaces with a nonlinear stress-strain law (Tano et al., 2016a). Several authors [e.g., Byrne (1994), Connell (2002), Jones and Dixon (2005), Fowmes et al. (2005b), Zhu et al.

(2009), Chen et al. (2009b), Zamara et al. (2014)] have used this program to model complex geosynthetic interactions.

The procedure used to consider such particularities (e.g. the strain softening behaviour of interfaces and the nonlinear axial stiffness of geosynthetics) and to verify the numerical model by comparison with experimental tests is outlined in the flowchart of Figure 8. Prior to the calculations, four specific subroutines codes were developed by using the *FISH* programming language, which is embedded in the FLAC software. The first FISH code modeled each geosynthetic as a concatenation of several structural elements to allow the properties of each individual structural element to be varied independently of the other parts of the geosynthetic and as a function of strain. A similar FISH code was developed to model the various interfaces to allow strength at each interface to soften independently. The two other FISH codes were developed and used within the program to implement strain softening at the interfaces and to update the geosynthetic axial stiffness as a function of strain. Each FISH code has already been used in a previous study (Tano et al., 2016a), but the results were not verify by comparison with experimental tests.

The experimental verification of the numerical modelling technique involved the following three steps (Figure 8):

- Step 1: The four UTTs conducted on the GMB, GGR, GCL, and GTX were first numerically modeled based on the axial modulus vs strain curves shown in **Erreur ! Source du renvoi introuvable.a** and **Erreur ! Source du renvoi introuvable.b**. The

tensile forces and strains obtained from the numerical model were compared with those obtained by experiment for each of the four geosynthetics.

- Step 2: The DSTs conducted on the six interfaces (I1–I6) were also numerically modelled based on the dependence of the friction angle on relative shear displacement (see **Erreur ! Source du renvoi introuvable.c** and **Erreur ! Source du renvoi introuvable.d**). In this step, the shear-stress–displacement curves measured by the DST are compared with the results of the numerical simulation of the six interfaces.
- Step 3: Once steps 1 and 2 are completed for the index tests, the accuracy of the numerical modelling technique was determined by using the LSTA. This large-scale test, which involved multiple interactions between the four geosynthetics and a mechanical load above a cavity, was also numerical modelled. Here, the sand within the LSTA was modelled by using a rectangular mesh that becomes finer as it nears the geosynthetic contacts. The mesh consists of 2500 volume elements (zones), each having a size between $0.02\text{ m} \times 0.025\text{ m}$ and $0.02\text{ m} \times 0.01\text{ m}$. At the lower side of the LSTA, fixed nodal displacements were imposed. At the sides of the model, the horizontal displacements were fixed. Then, a pressure was applied to the top of the LSTA. Furthermore, all the geosynthetic layers in the model were represented by structural beam elements, which can reproduce the membrane effect with zero inertia. These structural elements are the only structural elements in FLAC 2D that allow direct interaction between two geosynthetics. Next, the overall deformed shape and mechanical behaviour of the reinforced geosynthetic system observed in the LSTA were compared with the results of the numerical simulation. Finally, the simulated strains within the GGR and GMB were compared with the strains measured with the MSGs.

All of the comparisons in steps 1 to 3 are fully discussed in the following sections.

6. NUMERICAL MODELLING VERSUS EXPERIMENTAL RESULTS

This section compares the results of numerical modelling with experimental results and discusses these comparisons for the three experimental tests (UTTs, DSTs, and LSTA).

6.1 Uniaxial tensile tests

Figure 4 compares the tensile curves measured by UTTs and with curves obtained by using the numerical modelling technique. The tensile curves for the GGR and the GMB appear in Figure 4a and those of the GTX and GCL appear in Figure 4b. The experimental and numerical results are given in Table 2 for 2% and 4% strain.

The results reported in Figure 4 and Table 2 provide clear evidence that the numerical model provides accurate results for all the geosynthetics. For the GMB, which exhibits the most nonlinear behaviour (i.e., significant curvature in the plot of tensile force vs strain), the strains and forces obtained by using the numerical model are very close to the measured values. For example, at 4% strain, the measured force within the GMB was 29.6 kN/m and the calculated force was 29.0 kN/m; a difference of only 0.6 kN/m (−2.0%). Again at 4% strain, the measured force within the GGR was 38.6 kN and the calculated force was 37.9 kN, a difference of only 0.7 kN/m (−1.8%). This excellent consistency between the measured and calculated forces and strains occurred for all four geosynthetics. These results demonstrate that the numerical

modelling technique the FISH codes that were developed to consider the evolution of the axial stiffness of geosynthetics in the numerical model, properly models the tensile behaviour of geosynthetics.

6.2 Direct shear tests

This section compares the shear-stress–displacements curves measured by the DSTs with the numerical results obtained by the numerical modelling technique. Figure 6 shows the shear-stress–displacement curves at normal stresses of 50, 100, and 150 kPa for the six interfaces I1–I6. Table 3 summarizes the difference between the DST results and the numerical results for 100 kPa normal stress and for two levels of relative shear displacements (10 mm and 40 mm).

The results reported in Figure 6 and Table 3 provide clear evidence that, for all six interfaces, the numerical model properly predicted shear behaviour. The numerical modelling results were comparable with the experimental results. In addition, this result holds for all levels of confining stress and for all relative shear displacements. Below we describe the results for the six interfaces obtained for 100 kPa normal stress and at 50 mm relative shear displacements.

Consider first interface I1 (sand-GTX) at 100 kPa normal stress and 50 mm relative shear displacement. The interface shear stress measured by using the DST was 61.8 kPa and the shear stress obtained by using the numerical model was 60.3 kPa; a difference of only 1.5 kPa (2.4%). For interface I2 (GTX-GMB) at a 100 kPa normal stress and 50 mm relative shear displacement, the interface shear stress measured by using the DST was 12.1 kPa and the shear stress obtained by using the numerical model was 13.7 kPa; a difference of only 1.6 kPa (13.2%). For interface

I3 (GMB-GCL) at 100 kPa normal stress and 50 mm relative shear displacement, the difference between the measured shear stress (16.0 kPa) and the calculated shear stress (15.8 kPa) was only 0.2 kPa; a difference of only 1.6 kPa (−1.3%). At interface I4 (GCL-sand), the difference between the measured shear stress (62.9 kPa) and the calculated shear stress (61.3 kPa) was 1.6 kPa (−2.5%). Finally, at interfaces I5 and I6 (GGR-sand), the shear stress measured by using the DST was 61.4 kPa and the shear stress obtained from the numerical model was 63.5 kPa; a difference of only 0.9 kPa (−1.5%). These results demonstrate that the numerical modelling technique used to simulate the strain-softening behaviour of interfaces accurately describes the shear behaviour of the six interfaces.

6.3 Large-scale test

Figure 9 shows the overall deformed shape of the reinforced lining system within the LSTA for 100 kPa confining pressure. Experiment and numerical model both exhibit a similar overall deformed shape, which is evidence that the results of the numerical model are reasonable and that it properly simulates the overall mechanical behaviour of the reinforced lining system. To deepen the analysis and show the relevance of the numerical model, the strains within the GGR and the GMB are analyzed.

To measure strain, a MSG was attached at the center of the cavity on both sides (lower and upper) of the GGR and of the GMB. The mean strain was then obtained and the corresponding error bar was determined. Figure 10a compares the strains measured within the GGR by using the MSGs (Experiment) and the strains calculated by using the numerical model. The same

comparison is made in Figure 10b for the GMB. Various values derived from these figures are grouped in Table 4.

For the GGR, the strains obtained by using the numerical model always fall within the experimental error bars (Figure 10a), which indicate the variability of the measured strains. The agreement is particularly good for strains below 1% (i.e., below 30 kPa) and above 2% (i.e., above 60 kPa). For example, at 10 kPa applied load, the strain measured within the GGR was 0.37% and the strain obtained by using the numerical method was 0.46%; a difference in strain of only -0.09%. At 100 kPa, the analogous strain difference was -0.04% (2.53% strain from experiment and 2.57% strain from numerical model). The maximum strain difference of 0.38% between the measured and calculated strains occurred at 40 kPa. These results demonstrate that the numerical model accurately predicts strains whatever the applied load.

Although the agreement was less impressive for the GMB than for the GGR, it remains acceptable. The maximum strain difference between the measured and calculated strains was only 0.30% of strain (at 30 kPa). Note that the accuracy of the numerical result increased in particular for strains above 1% (above 50 kPa). For example, at 100 kPa, the measured strain was 1.74% and the calculated strain was 1.65%, a strain difference of only 0.09%.

These results show that the numerical modelling technique accurately predicts both the overall deformed shape of the reinforced lining system and the strains within both the GGR and the GMB.

7. CONCLUSIONS

Numerical modelling can be used to aid in the design of geotechnical constructions involving geosynthetics (geosynthetics), such as landfills. However, the reliability of the numerical results depends on how the model is developed and on the parameter set that is used.

The present work develops a procedure to analyze multilayered-lining systems by using a 2D finite-difference method. The accuracy of this numerical modelling technique is then verified by comparing its results with those obtained experimentally by using uniaxial tests, direct-shear tests and a specific large-scale test. The large-scale test involved a reinforced lining system spanning over a cavity and made of a 50 kN/m polyvinyl alcohol geogrid (GGR) reinforcement embedded in a sand layer, a geosynthetic clay liner (GCL), a high-density polyethylene geomembrane (GMB), and a nonwoven needle-punched geotextile (GTX). All three experimental tests were modelled by using the given numerical modelling technique and the numerical results were compared with the experimental results. These comparisons lead to the following conclusions:

(a) The tensile curves obtained from the uniaxial tensile tests closely match those obtained from the numerical model, and this good agreement occurred for the four geosynthetics (GGR, GCL, GMB, and GTX) and for all levels of strain. This verifies that the numerical modelling technique provides an accurate description of the evolution with strain of the stiffness within the geosynthetics.

(b) The shear behaviour of the interfaces obtained from the numerical model closely match those obtained experimentally by using direct shear tests, and this high accuracy occurred for all

520 interfaces, at all levels of the confining stress, and for all relative shear displacements at the given
521 interface. These results indicate that the numerical modelling technique considering the strain-
522 softening behaviour of interfaces accurately describes the shear behaviour of the geosynthetic
523 interfaces.

524
525 (c) Considering the multiple interactions between the four geosynthetics that spanned over a
526 cavity in the large-scale test apparatus, the experimentally observed overall deformation of the
527 reinforced lining system was successfully described by the numerical model. At the center of the
528 cavity, a close match also occurred between the measured and calculated strains within the GGR
529 and GMB. This result verifies that the numerical modelling technique that was developed in this
530 work gives accurate predictions.

531 This ensemble of results verifies the accuracy of this numerical modelling technique and allows it
532 to be used with confidence to predict the mechanical behaviour of multilayered geosynthetic
533 lining system over a cavity.

534
535 Finally, although a large-scale test and multiple interactions between geosynthetics were
536 considered in developing this numerical modelling technique, this technique could benefit from
537 further verification; namely, by comparing its results against those of full-scale measurements
538 particularly from geosynthetic lining system on slopes. Furthermore, it would also be interesting
539 to study whether the use of confined UTTs or multiaxial tensile tests can increase the reliability
540 of this numerical modelling technique developed in this study.

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Table 1. Summary of material, geosynthetic, and interface properties.

SAND						
γ (kN/m ³)	E (MPa)	ν	c (kPa)	ϕ_{peak} (°)	ϕ_{res} (°)	Particle size distribution ($D_{\text{min}} - D_{\text{max}}$ mm)
14	30	0.25	0	35	33	0.1 – 1.6
GEOSYNTHETICS						
Type	Nature	Thickness (mm)	Mass per unit area (g/m ²)	Machine tensile strength (kN/m)	Strain at tensile strength (%)	Secant modulus at 1% (MPa)
GTX	Non- woven, PP	7.5	1200	77	84	5.2
GMB	HDPE	2	9350	35	13	763
GCL	Sodium bentonite	10	5000	10	12.5	7.7
GGR	Uniaxial, PVA	2	215	50	5.3	557
INTERFACES						
Type	K_s (MPa)	K_n (MPa)	c (kPa)	ϕ_{peak} (°)	ϕ_{res} (°)	
I1: sand - GTX	30			33.2	31.1	
I2: GTX - GMB	10			9.2	7.8	
I3: GMB - GCL	30	100	0	12.0	9.0	
I4: GCL - sand	30			33.6	31.5	
I5: GGR- sand	30			34.4	32.4	

635

636 Table 2. Comparison of results of uniaxial tensile test with results of numerical model.

Force within the geosynthetics(kN) using :	Strain within the geosynthetics							
	2%				4%			
	GTX	GMB	GCL	GGR	GTX	GMB	GCL	GGR
Uniaxial tensile test	0.7	22.2	1.8	18.2	1.4	29.6	4.2	38.6
Numerical model	0.6	21.3	2.0	16.4	1.2	29.0	4.5	37.9

637

638 Table 3. Comparison of results of direct shear test with results of numerical model at 100 kPa
 639 normal stress.

	Relative shear displacement at interface									
	10 mm					50 mm				
Shear stress at interface (kPa) using :	I1	I2	I3	I4	I5&I6	I1	I2	I3	I4	I5&I6
Direct shear test	58.2	14.3	16.3	58.9	49.1	61.8	12.1	16.0	62.9	61.4
Numerical model	58.1	14.8	17.1	58.1	50.2	60.3	13.7	15.8	61.3	63.5

640

641 Table 4. Comparison of results of large-scale test with results of numerical model.

	Geogrid			Geomembrane		
Applied load (kPa)	10	50	100	10	50	100
Strains (%) at the center of cavity using:						
Large-scale test	0.37	1.88	2.53	0.45	1.23	1.74
Numerical model	0.46	1.54	2.57	0.32	0.99	1.65

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Figure 1. Tensile test device used in this study.

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System to apply
normal stress

Test box

Jack for shear
displacements

650

651

Figure 2. Direct shear test device used in this study.

Figure 3. Large-scale test apparatus to simulate a cavity.

Figure 4. Tensile force vs strain measured by uniaxial tensile tests (solid curves) and compared with results of numerical simulation (symbols) for (a) geomembrane and geogrid, and (b) geosynthetic clay liner and geotextile.

Figure 5. Modulus of geosynthetic versus strain for (a) geomembrane and geogrid and (b) geosynthetic clay liner and geotextile.

Figure 6. Shear stress vs relative shear displacement measured by direct shear tests (solid curves) and compared with results of numerical simulation (symbols) for the following interfaces: (a) sand-GTX (I1). (b) GTX-GMB (I2), (c) GMB-GCL (I3), (d) GCL-sand (I4), (e) GGR-sand (I5, I6).

Figure 7. Mobilized friction angle versus interface displacement for the following interfaces: (a) sand-geotextile (I1), geosynthetic clay liner-sand (I4), geogrid-sand (I5, I6) and (b) geotextile-geomembrane (I2) and geomembrane-geosynthetic clay liner (I3).

Figure 8. Procedure used to verify the numerical modelling technique.

(a)

677

678 Figure 9. Overall deformed shape of the reinforced lining system with 100 kPa confining

679 pressure: (a) experimental results, (b) numerical results.

680

681 Figure 10. Axial strains within the geosynthetics measured by microstrain gauges vs applied

682 pressure and compared with results of numerical simulation for (a) geogrid and (b)

683 geomembrane.