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Numerical modeling of the mechanical behavior of geosynthetic liner systems (GLS) installed on slopes

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ABSTRACT: Geosynthetic systems constitute a preferential slip line when they are laid down on the slopes of civil engineering structures. To improve the design of GLS on slopes under low normal stresses, a numerical model based on the finite element method (FEM) is proposed in this paper. This 2-D model makes it possible to take into account the compatibility of the strains and displacements of the various components in the GLS and the effect of flow parallel with the slope in the cover soil. Two full-scale instrumented experiments are simulated to validate the model presented.

1 INTRODUCTION

Geosynthetic lining systems (GLS) are increasingly widely used in many works, in particular in civil engineering structures and in landfills. The "geosynthetics" complex installed on a slope constitutes a preferential slip surface for the cover layer. In most cases, the slope is too steep for the cover layer to be self-stable, because optimization of the design of the structure tends to increase the slope and this can disadvantage the stability of the slope and sometimes involve slipping of the cover soil or the failure of some geosynthetics in the GLS.

The analysis and design of the GLS can be simulated by global and local approaches. Global approaches based on equilibrium limit methods have been developed for the design of GLS (Soong and Koerner 1996, Giroud et al 1995). The limits of these approaches come from the simple assumptions made on the behavior of the geosynthetics systems on slopes. A fine analysis of these systems is necessary to take into account the compatibility of the strains and the non-linearity of the behavior of the various components of a GLS. That can be done only while working on a local scale. Many authors have proposed numerical models based on local approaches, such as the finite element method (FEM) (Wilson et Koerner 1994, Villard et al. 1999), the finite differences method (Lalarakotoson 1998) or the discrete element method (Chareyre, 2003). A finite element model is proposed to improve knowledge of behavior of each component and their interactions in any point of the GLS; the effect of water is taken into consideration by this model. The comparisons of numerical and experimental results are satisfactory and parametric studies are thus possible.

2 FINITE ELEMENT ANALYSIS

2.1 *Finite element configuration*

Many applications of the ABAQUS finite element code have shown its performance in modeling systems comprising geosynthetic layers. An ABAQUS mesh used in our model is given in Figure 1. The soil cover is represented by porous elements with 4 nodes (CPE4P) which make it possible to take hydraulic conditions into account and in particular the pore pressure. The sub-soil is simulated with the same type of element, but it is considered rigid and fixed in this case. Two-node bar elements are appreciated by many authors to represent the geosynthetic liners. These structural elements are effective and economic

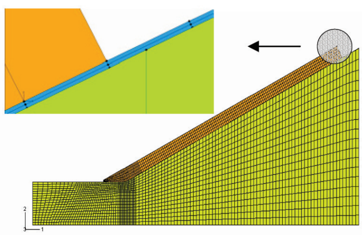

Figure 1. Mesh used for modeling the slope protection system with the buttress.

for calculation time compared to the solid elements with 4 nodes. However to take into account water in our model, we used quadrilateral porous elements with a thickness of 2 mm. The anchorage of geosynthetics was simulated by fixing the nodes at the head of slope. The case without a buttress was carried out simply by removing the buttress in the principal model and using an elastic model for the cover soil.

2.2 *Material properties*

2.2.1 *Cover soil*

ABAQUS provides a very broad choice of models of the behavior of materials. We chose the modified Mohr-Coulomb plasticity model for the cover soil. This model allows the material to harden and/or soften isotropically, the plastic rate of displacement is governed jointly in status and direction by a traditional criterion of slip and by a non-associated law of slip proposed by Menétrey and Willam (1995).

2.2.2 *Geosynthetics*

Considering the strain of geosynthetic reinforcement, which is relatively weak in reality, an elastic model was applied to simulate the geosynthetic sheets. The phenomenon of setting in compression of the cover soil and the bottom of the slope (buttress) was taken into account by using a user' subroutine in FORTRAN (Figure 2). A criterion of calculation stop of event of the failure in the geosynthetic sheets was carried out by giving a null module to geosynthetic for a maximum strain. According to results of a parametric study, a ratio of 1/100 was selected to represent the very low resistance in compression of the geosynthetics.

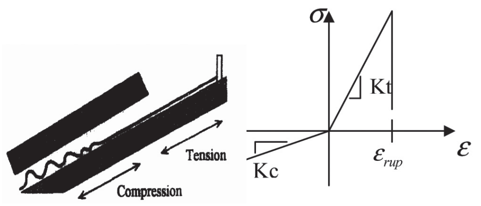

Figure 2. Material model of the geosynthetics.

2.2.3 *Interactions*

The behavior of the interfaces was described in the normal direction and the tangential direction respectively. A hard contact pressure-overclosure relation was selected so that the contact pressure could be transmitted only when surfaces were in contact. In order to avoid numerical problems, once two surfaces are in contact, they are not allowed to separate any more in the continuation of the analysis. Various tangential behaviors (linear or non-linear) with the interfaces were programmed in FORTRAN depending on materials in contact. In our case, tangential interaction of the surfaces in contact is described by a elasto-perfectly plastic stress-strain behavior with criterion of Mohr-Coulomb which is expressed by a coefficient of friction (μ) and an elastic slip (E_{slip}). The E_{slip} parameter, representing the relative displacement necessary before the maximum shear resistance is reached, describes the tangential rigidity of the interface or module (G); according to Perkins (2004) this rigidity is a function of the normal stress and the cyclic test load (Figure 3).

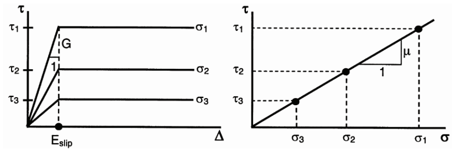

Figure 3. Schematic of the Coulomb interface friction model.

3 LOADS

3.1 *Progressive loading*

The cover soil is generally installed gradually from the bottom to the top of the slope on the construction site. In order to respect this process in our model, the layer of the cover soil is divided into a series of identical blocs, whose weights are activated one after the other in the different stages of analysis.

3.2 *Seepage in the cover soil*

In the literature, few studies of the influence of water on the stability of GLS have been conducted. The design methods in which seepage is taken into account are generally developed using global approaches. In this paper, the use of the porous elements enables us to carry out analysis integrating the influence of water. For the design model to be presented, seepage parallel to the slope is assumed just after the end of the progressive loading (Figure 4).

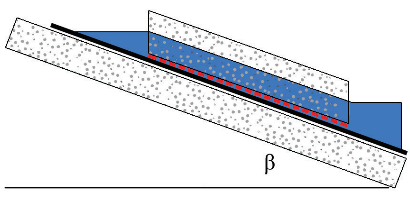

Figure 4. Diagram of a DEG with flow in the cover soil.

In other words, a flow within the cover soil mass consists of flow lines parallel to the slope. Note that such a condition is often reached in the lower portions of natural slopes. This is carried out by imposing pore pressures generated by a flow on the nodes of waterproof sheet in the different stages of the analysis. The value of these pressures gradually increases with the thickness of saturation in the cover soil. No transitory effect is taken into account.

4 VALIDATIONS

The model described above was then validated by two full-scale experiments. The course of these two experiments and the comparisons of experimental and numerical results are presented here.

4.1 *Case of Montreuil Sur Barse*

To evaluate the performance of various GLS, a full-scale experiment was carried out on an instrumented GLS. A detailed presentation of this work and its instrumentation was made by Feki (1996). The full experimental programme consisted of four successive stages; we will only show here the results obtained with the model presented and experimental measurements in stage I which involved monitoring the forces and displacements in the various GLS components while loading the granular material layer meter by meter on the slope over a total loading length Lc up to 6 m (Figure 5). The characteristics of the interfaces are given in Table 1. The axial stiffness in traction of geotextile and geomembrane are 458 kN/m and 56 kN/m respectively.

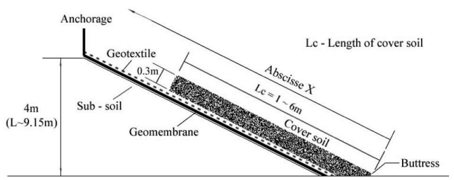

Figure 5. Diagram of the Montreuil Sur Barse experiment.

Table 1. Characteristics of the interfaces (Villard, 1999).

	Sub-soil/GMB	GMB/GTX	GTX/Soil
δ (°)	9	12	29
Eslip (mm)	0.2	2	2

Figure 6 shows the relationship between length of cover soil and tension force in the geosynthetics at the anchorage point. As can be seen, the theoretical and experimental curves are close to each other. The differences observed in the tensile force at the head of the geotextile, especially at the beginning of the loading deserve a precise study into the behavior of the cover soil, in particular the buttress. The reduction in the force of anchorage at end of the loading is due to the displacement of the anchorage system which was noted on the site.

The comparison between experiment and modeling for displacement in various points from geotextile is presented on the Figure 7, for several lengths of loading

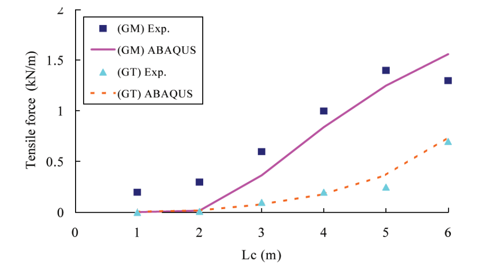

Figure 6. Tensile force at the head of the geosynthetic sheets – comparison with FEM.

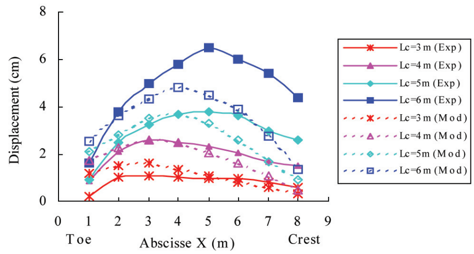

Figure 7. Geotextile displacement – comparison with FEM.

(Lc = 3,4,5,6 m). It is noted that the experimental displacements obtained close to anchorage are too great because displacements are theoretically null at the point of anchorage. Lifting of the measurement cables by the bits of fasteners of the geosynthetic sheets was noted for Lc = 6 m and can explain this difference.

All the results obtained reveal good coherence of the modeling carried out with measurements.

4.2 *Case of the experimental slope of Cemagref*

The principal objective of this experiment was to check the influence of the saturation of cover soil on the stability of a GLS. The experiment proceeded in two stages:

- progressive loading without buttress,
- saturations of the cover soil.

The diagram of the complex geosynthetic and the characteristics of materials used as well as their interfaces are presented in the Figure 8 and Table 2 and 3 respectively. The details of this experiment were presented by Briançon (2002).

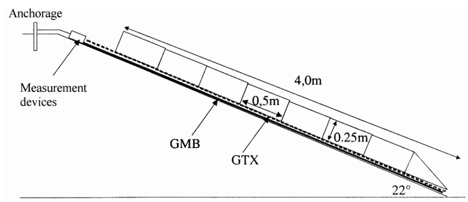

Figure 8. Diagram of the CEMAGREF experiment.

Table 2. Characteristics of the interfaces (Briançon, 2001).

	Sub-soil/GMB	GMB/GTX	GTX/Soil
δ (°)	32	16	37
Eslip (mm)	1	0.5	2

Table 3. Material properties (Briançon, 2001).

	Cover soil	GMB	GTX
E (kPa)	2000	7500	312000
ν	0.3	0.3	0.25

The comparison of the tensile forces of anchorage in geotextile between numerical results, calculated by software G-SCAP (calculation software to the ultimate equilibrium based on the method of two blocks, Poulain and al. 2004) and experimental measures is shown in Figure 9.

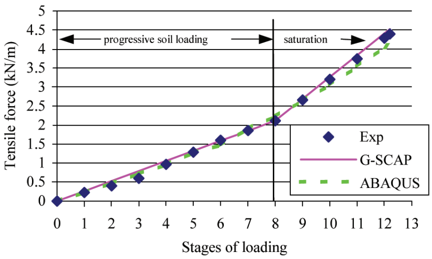

Figure 9. Tensile force at the head of the geosynthetic sheets comparison with FEM and G-SCAP software results.

Good coherence was observed in this comparison. In the absence of a buttress, it can be noticed that the global approaches also give satisfactory tensile forces in geosynthetics. More, finite element analysis showed a slip of cover soil while the height of flow reached 20 cm and the rupture in site took place at the end of the test where a 21 cm height of water in cover soil was measured by the pressure sensors.

5 CONCLUSIONS

The development of this new model makes it possible to improve knowledge of the mechanical behavior of the geosynthetics in GLS. The influences of the progressive loading of cover soil and its saturation on the stability of whole system are correctly modeled.

The validation of the suggested model means we can continue our research tasks by parametric studies. One of the essential contributions of the model proposed is the possibility of taking into account the hydraulic conditions which we know to play a key negative role on the stability of GLS.

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