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Shear strength tests on geosynthetic liner systems

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ABSTRACT

In order to follow the behaviour of a liner system on the slope, it was decided to carry out a real scale experimentation at Montreuil/Barse (France). Three different systems are tested on the slope 1/1 and ½ of one landfill cell. The different components of the liner system are instrumented for displacement measuring. At the same time a laboratory program is carried out to evaluate the shear strength of the materials and their interfaces. The obtained characteristics are used to determine the theoretical stability of the different systems by limit equilibrium methods. The collected data, compared to predicted stability, constitute an efficient way to validate these design methods.

1-INTRODUCTION

Four experimental cells were realised in an existent municipal waste landfill at Montreuil/Barse (France) with the support of the French Agency for the environment (ADEME). The research was piloted by the French Institute of Agriculture and Environmental Engineering Research (CEMAGREF of Antony). The in situ experimental program was performed by a French waste management company CGEA-ONYX. Publics laboratories, CEMAGREF, LRPC of Nancy and University of Grenoble 1 I.R.I.G.M-Lgm were associated.
These cells covering an area of about 1.7 10^4 m^2, were constructed in order to study the behaviour of four different slope liner systems :

* Geosynthetic Clay Liner (Bentonite membrane) ;
* Compacted Clay Liner ;
* HDPE Geomembrane ;
* HDPE Geomembrane + Clay.

Each landfill cell has a squared shape with 50 m side and with side slopes 1/1 and 1/2. In this paper, we have taken a particular interest in the stability analysis of the Geosynthetic Clay Liner (GCL). A plan view with a cross section of the landfill cell is given on Figure 1.
The bentonite membrane is covered by a granular soil to avoid the development of high pressure on it and to protect it against the mechanical and biochemical damaging.
Two kind of cover soil are tested. On the slope 1/2, the GCL is covered by a 0.3 m of gravel or by a silty sand layer. This layer was unstable and was completely leached after big precipitations, so a geosynthetic cellular structure was used to confine it on the slope 1/1. A geotextile was also added between the silty sand and the bentonite membrane as a mean of reinforcement.

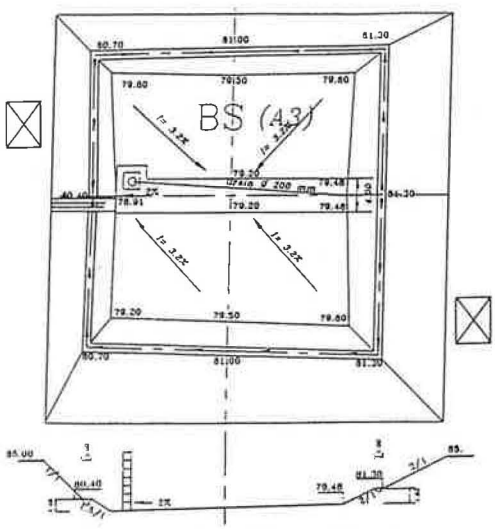


Figure 1 : Plan view of the GCL landfill cell.

The stability of the GCL after laying the cover soil on the slope, and after filling the cell with waste, is monitored by measuring the displacements in each component of the lining system. The data collected from this real scale experimentation, compared to predicted stability, allowed the validation of the design methods based on limit equilibrium.
To conduct the stability analysis, an experimental program is carried out to evaluate the shear strength at critical interfaces using a large box. We detail the operative method which concern the fixation of geosynthetics, physical conditions of soils and application of vertical load.

2-REAL SCALE EXPERIMENTATION DATA

The data presented in this paper and which are available in several publications [1,7,8], are obtained at the experimental waste landfill of Montreuil.

2.1-Cross section of the lining system

The bentonite membrane is a geosynthetic product where the bentonite is enclosed between two geotextiles connected by longitudinal stitches (Figure 2).
It was laid on the subgrade soil (clay), anchored on the top of the slope and covered with :

*0.17 m of silty sand confined in cellular structure on the slope 1/1 (Figure 3b).
*0.3 m of gravel or silty sand on the slope ½ (Figure 3a)

2.2-Instrumentation of the lining system

LRPC Nancy was in charge of the monitoring of the liner. Both the lining systems on slopes 1/1 et ½ are instrumented for measuring the displacements in the bentonite membrane and in the cover soil. The positions of the measuring points (landmarks) are given by Figure 4 and Figure 5
The displacements in the bentonite membrane are measured in 14 points at the slope ½ and 13 points on the slope 1/1. The displacements in the gravel, on the slope ½, are measured by 14 metallic T implanted in the middle of the layer. For the slope 1/1, 9 landmarks are fixed in the intersection of the alveolar structure.

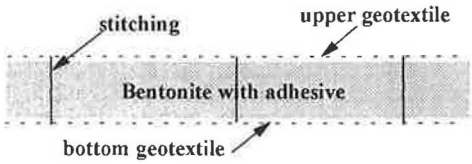
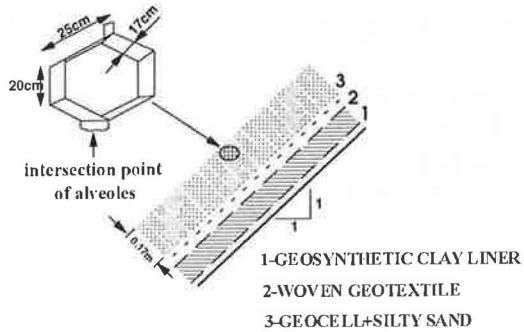
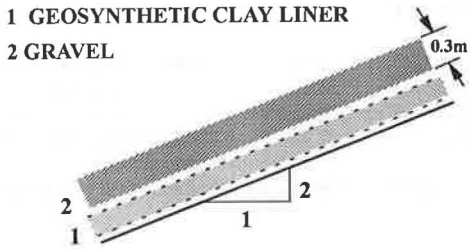


Figure 2: Scheme of a stitched GCL.



(a)- Cross section of the GCLiner system on the slope 1/1.



(b)-Cross section of GCLiner system on the slope 1/2.
Figure 3 : Cross section of the lining system on 1/1 and ½ slopes for the experimental GCL landfill cell.

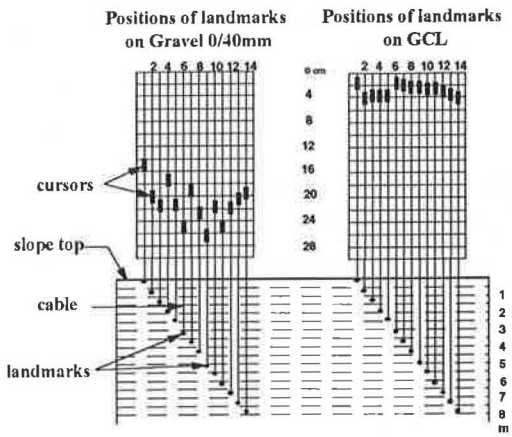


Figure 4 Instrumentation of the lining system of the GCL landfill cell. Slope 1/2.

The landmarks are linked by thin metallic cables to a monitoring box. Displacements are measured by a cursor attached at each cable which is braced by counterweights. The measuring error is estimated at 0.5 cm.

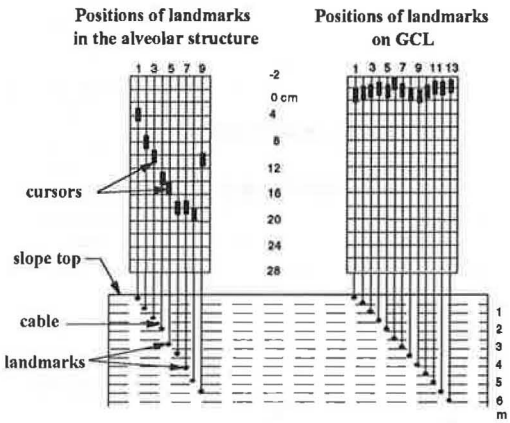


Figure 5 : Instrumentation of the lining system of the GCL landfill cell. Slope 1/1.

3-DISPLACEMENT MEASURES

These displacements are obtained before filling waste in the landfill cell.

3.1-Behaviour of the lining system on the slope ½

The distribution of displacements along the ½ slope is given for the bentonite membrane and the gravel, 43 days after the settlement of the cover soil on the slope (Figure 6).
The displacements in the bentonite membrane are very low with regard to the displacements in the gravel layer. The displacements are more important on the toe of the slope. It is interesting to note here that the specific construction doesn't allow soil buttress at the toe. These measures remained unchanged after 500 days of observations.

Average strains calculated from the displacements of two consecutive points, shows that the bentonite membrane is in extension on the lower half of the slope. The maximum strain is about 1.5%.

3.2-Behaviour of the lining system on the slope 1/1

The displacements in the bentonite membrane and the confined silty sand show a more important difference comparing with those in the previous lining system (Figure 7).

Silty sand

The displacements of the confined silty sand shows a movement in a lamp of about 3 cm, one day after its installation (14 Oct.).
These movements are significantly higher after 44 days and reach a displacement of 45 cm at mid-slope. Measurement taken until 3 months after the installation correspond to a stabilisation of the movement after this last date (Figure 8).

Bentonite membrane

One day after its installation, the bentonite membrane is extended at the top of the slope and the average strain reaches 5.5%. After this date, the displacement had significantly decrease till 3 months of observations (Figure 9).
By reason of the excessive strain (>2%), it could be assumed that the important displacements had caused partial failure of the bentonite membrane on the different landmarks.

4-DIRECT SHEAR INTERFACE TESTS

In order to characterise the different materials used in the field and their interfaces, direct shear tests are

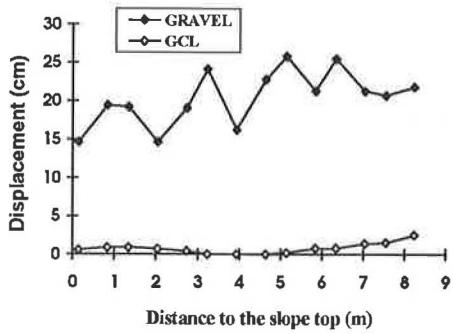


Figure 6: Displacements in the bentonite membrane and in the gravel along the slope 1/2, 43 days after installation.

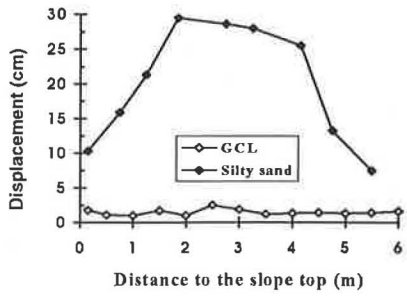


Figure 7 : Displacements in the bentonite membrane and in the confined silty sand just after installation (13 Oct.).

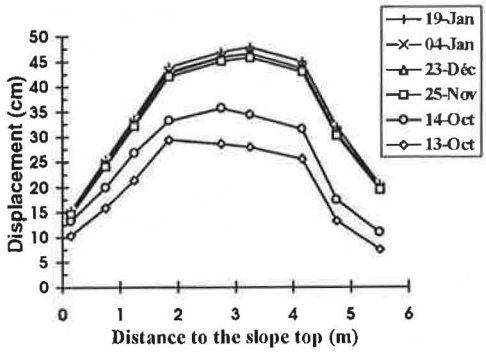


Figure 8 : Evolution of displacements in the confined silty sand on the slope 1/1 (until 3 months after installation).

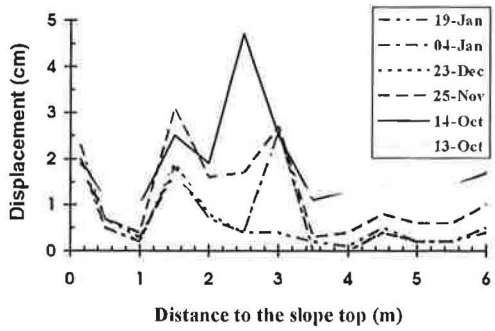


Figure 9 : Evolution of displacements on the bentonite membrane (until 3 months after installation).

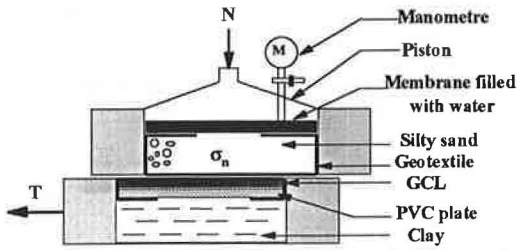
realised in the I.R.I.G.M laboratory using a large box.

4.1-Test conditions

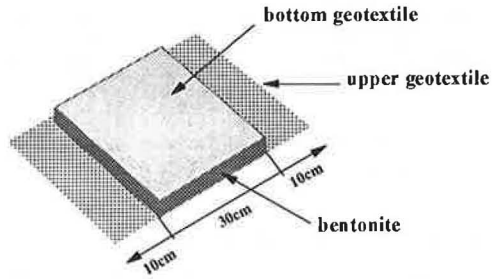
Direct shear tests of various interface combinations are performed using a 300 mm x 300 mm direct shear testing apparatus. It is fitted with electronic gauges for monitoring the shear forces applied to test specimens and its horizontal displacement.

The normal stress is applicated by an hydraulic jack controlled by a steam gauge.

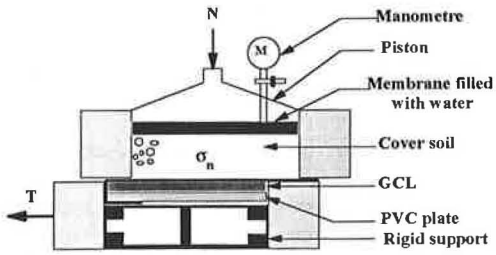
Figure 10a shows a schematic cross section of a typical interface-sample configuration for interface combinations between two geosynthetics (GT/GCL). The lower geosynthetic is generally fixed on a rigid support (to assure the flatness of contact area). For these tests, the one centimeter thick GCL sample, is



(a)-Interface combination GCL/Geotextile/Silty sand.



(b)-Dimensions of the GCL sample. Downward view.



(c)-Interface combination GCL/cover soil. (Gravel and Silty sand).

Figure 10 : Test conditions for direct shear interface tests.

cut in a 300 mm x 500 mm rectangular. The bottom geotextile is cut in 300 mm x 300 mm square and the bentonite out the square is removed. (Figure 10b)

The two sides of the upper geotextile are fixed to a PVC plate setting on a clay layer. This subgrade soil is compacted in the lower box. The contact area initially dimensioned 300 mm x 300 mm decreases during the test, so it has to be adjust.

In the upper box, the cover soil (silty sand) is surrounded by the woven geotextile (Geolong 200), which is laid on contact with the GCL. The silty sand is compacted to initial dry density of 16 kN/m^3 and at initial water content of 14%.

Figure 10c shows a schematic cross section of a typical interface-sample configuration for interface combination between soil and geosynthetic. These combinations include :

- Silty sand/GCL
- Gravel/GCL.

The GCL specimen is installed with the same procedure described above but only one side of the upper geotextile is fixed to the PVC plate, so that the contact area remains constant during the test. The PVC plate is setting on a rigid support.
The cover soil (gravel or silty sand) is compacted in the upper box with 5 centimetres thick, at dry density and water content representative of field conditions.
In all tests, the lower box moves at a constant rate of 1 mm/mn. The normal stress is distributed by a membrane and controlled with a steam gauge. Normal stresses applied on the sample interfaces during testing are 25, 50, 75 and 100 kPa.

5-TEST RESULTS

First tests are realised to characterise the interface soil/soil (Gravel and silty sand). The friction characteristics are given in Table 1.
We call ϕ_{g1} the angle of friction and C_{g1} the cohesion of the GCL/geotextile (GCL / GT) interface, ϕ_{g2} the angle of friction and C_{g2} the cohesion of the GCL/silty sand interface and ϕ_{g3} the angle of friction and C_{g3} the cohesion of the GCL/gravel interface.

5.1 GCL / GT Interface

GCL/GT interface samples are tested under 25, 50, 75 and 100 kPa. The relationship shear stress-shear displacement (Figure 11) doesn't show any particular behaviour.
The slight peak obtained under 100 kPa, is due to the presence of the silty sand above the geotextile. It is important to note that the maximum shear stress is mobilised after a displacement of 10 mm under a normal stress of 25 kPa which is relatively important. The friction angle is 15° (Figure 12).

Table 1 : Soil friction characteristics

soil	unit weight (kN/m³)	water content	ϕ °	c (kPa)
Clay	20	20%	0	60
Gravel	18	6%	41	0
Silty sand	19	13-14%	37	8

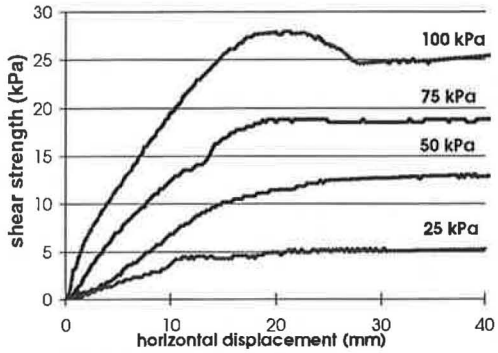


Figure 11 : Relationship shear stress-shear displacement at the interface GCL/Geotextile.

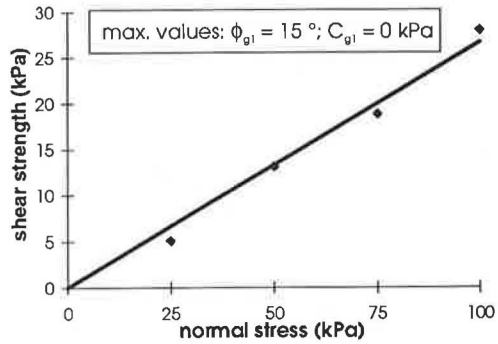


Figure 12 : Mohr diagram for GCL/Geotextile interface.

5.2 GCL/Silty sand Interface

GCL/silty sand samples are tested under 50, 75 and 100 kPa. The shear stress continuously increases during the period of test (1 hour) (Figure 13). This behaviour seems to be related to the kind of soil. In fact the same observations are available for the sand/sand shear tests. The increase level of the shear stress is very low and require an important displacement. The maximum values of the shear stress obtained at the end of tests are considered to determine the interface shear strength. The found values are a friction angle of 26° and an adhesion of 2 kPa (Figure 14).

5.3 GCL/Gravel Interface

The GCL/Gravel samples are tested under 25, 50 and 75 kPa (Figure 15). The shear stresses increase with the normal stress and reach a conatant stress level. The friction angle corresponding is 40° (Figure 16).

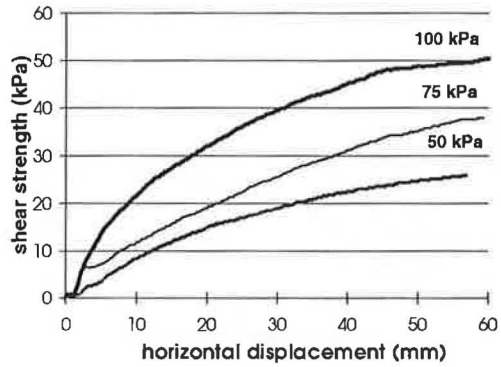


Figure 13 : Relationship Shear stress-Shear displacement at the interface GCL/Silty sand.

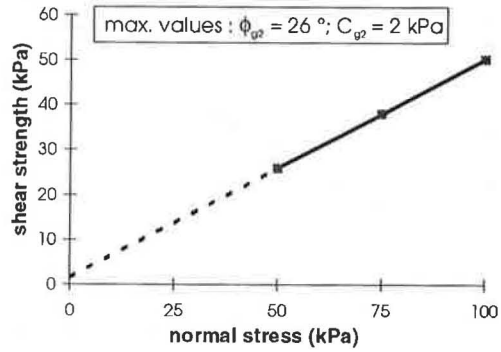


Figure 14 : Mohr diagram for GCL/ Silty sand interface.

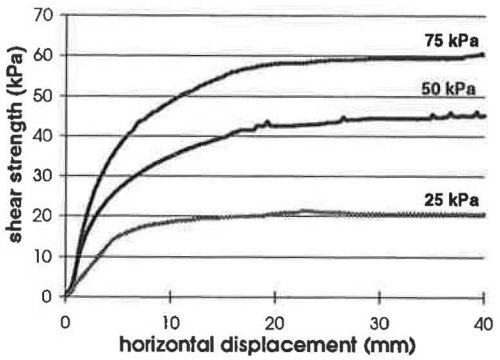


Figure 15 : Relationship Shear stress-Shear displacement at the interface GCL/Gravel.

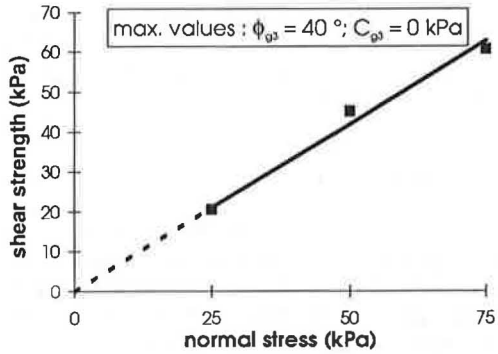


Figure 16 : Mohr diagram for GCL/Gravel interface.

Table 2 : Friction characteristics at the tested interfaces.

interface	normal stress (kPa)	interface characteristics
GCL/GT	25,50,75,100	ϕ_{g1}=15°, C_{g1}=0 kPa
GCL/Silty sand	25,50,75,100	ϕ_{g2}=26°, C_{g2}=2 kPa
GCL/Gravel	25,50,75	ϕ_{g3}=41°, C_{g3}=0 kPa

6-STABILITY ANALYSIS

In this paper the stability of the lining systems is only analysed by limit equilibrium methods :

* GIROUD and KOERNER (I) method [3,4] based on limit equilibrium of two wedges ;
* The software ETAGE developed by the LCPC and based on the perturbation method.

The two methods give the safety factor and evaluate the necessary tension to ensure the stability of the lining system to obtain the desired safety factor.

The software ETAGE was developed for the stability study of the lining system where the length on the slope is very large with regard to the thickness of the cover soil. The failure surface is determined by a linear segment bounded at the top and at the toe of the slope by two circular arcs and it supposed to pass above the segment CD as considered by Giroud-Koerner (figure 17).

The ETAGE computations estimate the security factor of the lining system on the slope. If the security factor Fs is found less than the desired safety factor Fr (introduced by the author), the resolution of the system is undertaken after adding a tensile force T supported by a mean of reinforcement (geotextile

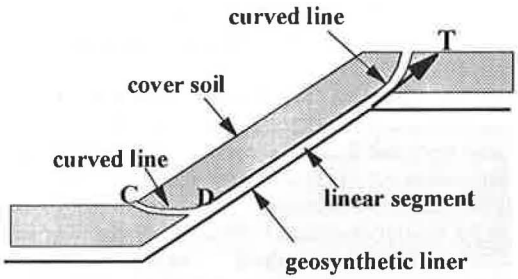


Figure 17 : Geometry of the failure surface used in ETAGE software.

or geogrid). This force is calculated by $T = F_m(1 - F_s / F_r)$, where Fm is the average driving force calculated at the intersection points between the circular arcs and the linear segment (Figure 17).
The friction characteristics of the different materials and their interfaces determined by the shear tests (table 1 and 2) have allowed this stability analysis.

6.1-Stability analysis of the liner system on the slope 1/1

On the slope 1/1, the woven geotextile was added between the confined silty sand and the GCL in order to reduce the tensile force that will be supported by the geocell.
Two simulations are realised without and with geotextile, to illustrate the beneficial effect of the woven geotextile used as a reinforcement.
Without geotextile, we have analysed the stability of the confined silty sand covering the GCL. The obtained results (Table 3) are compared to those with geotextile (Table 4), where the failure surface is assigned at the interface GCL/GT.
In the first case the tension is supported by only the geocell T_{GC}. With the geotextile, the total tension T_T calculated when Fs<1, is provided by both the geocell and the woven geotextile. These results are obtained with :
- a unit weight of silty sand γ=19 kN/m^3 and a soil thickness h = 0.17 m.
- a slope angle β =45° and a slope length L = 6.5 m.
The safety factors calculated with the shear caracteristics determined by experimental tests, are more than one with and with out reinforcement. These results don't reflect the real behaviour as the displacements measured on the confined silty sand were very importants.
As the field conditions don't allow a high density of soil on the slope, we consider no cohesion for soil and at the interface GCL/silty sand, for the following simulations.

Table 3 : Stability analysis without geotextile

silty sand		GCL/soil		method I		ETAGE	
ϕ	c	ϕ	c	Fs	T_{GC}	Fs	T_{GC}
37	8	26	2	1.8	0	1.86	0
37	0	26	0	0.84	5	0.77	5

Table 4 : Stability analysis with geotextile on the slope 1/1.

silty sand		GCL/GT		method I		ETAGE	
ϕ	c	ϕ	c	Fs	T_T	Fs	T_T
37	8	15	0	1.05	0	1.05	0
37	0	15	0	0.81	8.1	0.63	9

With out geotextile, the tension supported by the geocell is estimated at 5 kN/m. With the geotextile, the total tension is 9 kN/m. The important proportion will be effectively supported by the geotextile as it has the more rigid stifness (J=2500 kN/m). The real tension in the geocell will be determined if the tension in the geotextile was measured.

With no buttress, the required total tension $T = [\gamma hL \sin\beta - \gamma hL \cos\beta tg\phi_{g1}]$, to obtain an equilibrium state (Fs=1), is of 10.8 kN/m. The buttress force is estimated at 2 kN/m obtained by the diffrence between tensions with and without butress effect.
These methods based on limit equilibrium methods under estimate the effect of soil strength, when other studies, based on numerical methods, gave a more realistic approach of tensions carried by the different components of the system.

6.2-Stability analysis of the liner system on the slope 1/2

As the specific construction doesn't allow a soil buttress at the toe, the safety factors are calculated with this hypothesis. In ETAGE computations, the soil characteristics are decreased until zero at the buttress level.

GCL-silty sand system

The friction angle at the interface GCL/silty sand is slightly lower than the slope angle, which explain the failure occurred after laying the silty sand on the slope.
The safety factor calculated by $F_s = tg\varphi / tg\beta$ (infinite slope hypothesis) is about 0.98.
By ETAGE, the minimum safety factor is roughly equal to the former F_s=1.01 with a unit weight of soil

γ=19 kN/m3, thickness of cover soil h=0.3 m, length slope L=8.3 m, slope angle β=26.5°, friction angle of soil ϕ=37° and friction angle at the interface GCL/GT ϕ_{gl}=26°.
The critical state (β=26.56° and ϕ_{gl}=26°), and the low capacity drainage of the silty sand, explain its failure on the slope 1/2.

GCL-Gravel system

The shear strength at the interface GCL/Gravel is high enough to ensure the stability of the liner system even with no buttress effect.
Although, these design methods are enable to predict the important displacements measured on the gravel (25 cm).

7-CONCLUSION

The difficulties to conduct shear tests with this type of geosynthetics, consists to find how to fix the GCL sample. Different types of anchorage are tested to achieve the one used for these tests.
To better approximate the shear characteristics, it is needful to conduct these tests at the same field conditions (normal stress, unit weight of the cover soil and its water content). As the field normal stresses are generally low, it would be necessary to complete these shear box tests by inclined plane tests. These methods based on limit equilibrium have the advantage to be easy and rapid to use. They give a first approach on the liner system stability and the tension developed on the geosynthetic components.
To have more realistic results, it is relevant to use numerical methods that take into account the strain compatibility and which give the displacements on the different components.

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REFERENCES

[1] C.BERNARD, Y.MATICHARD, P.PIERSON AND I.LETELLIER « Construction of four municipal landfill lining systems at Montreuil sur Barse ». Rencontres 93, Tour, France, Tome 2, pp 249-433.
[2] G.BRESSI, M.ZINESI, F.MONTANELLI AND P.RIMOLDI « The slope stability of GCL layers in geosynthetic lining system ». Proceedings Sardinia 95, Fifth International Landfill Symposium, Italy; 2-6 Oct. 1995.
[3] J.P GIROUD, R.H.SWAN, P.J.RICHER and P.R.SPOONER « Geosynthetic landfill cap : Laboratory and field tests, design and construction ». Geotextiles and Geomembranes and related products 1990. Balkema Rotterdam.
[4] J.P.GIROUD AND J.F.BEECH « Stability of soil layers on geosynthetic lining system ». Geosynthetics'89 conference, San Diego, USA.
[5] R.KOERNER AND HWU « Stability and tension considerations regarding cover soils on geomembrane lined sloped ». Geotextiles and Geomembranes 10 (1991), pp 335-355.
[6] J.H.LONG, R.B.GILBERT AND J.J.DALY « Geosynthetic Loads Landfill Slopes : Displacement Compatibility ». Journal of Geotechnical Engineering, vol. 120, n°11, nov. 1994.
[7] E.TANAYS, I.LETELLIER, C.BERNHARD AND J.P.GOURC « Behaviour of Lining Systems on Waste Landfills Slopes : An experimental Approach ». Fifth International Conference on Geotextiles, Geomembranes and related products. Singapore, 5-9 Sep.1994, pp 977-980.
[8] B.SOYEZ, P.H.DELMAS, CH.HERR AND J.C.BERCHE « Computer evaluation of the stability of composite liner ». Geotextiles and Geomembranes and related products 1990, Balkema Rotterdam. pp 517-522.