Geosynthetics: Applications, Design and Construction, De Groot, Den Hoedt & Termaat (eds)
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Reinforcing geotextiles for the slope of a waste disposal site

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ABSTRACT: Municipal waste landfills are often designed looking for the steepest possible slopes to increase the storage volume. For both inclined base lining system and the cappings slope stability is a decisive point to be taken into account in the design. All methods are based on a factor of safety against limit equilibrium of the slopes.
Slope stability is checked by taking into account geometry and properties of the used materials. One of the most important input parameters is the frictional behaviour of between soil and the geosynthetic materials respectively in between these materials. In the design a sufficient factor of safety both against rupture and pull-out must be achieved. Principles of design are described and an example for a reinforced slope of a waste disposal site is given.

1. INTRODUCTION

Slope stability problems are of crucial importance in geosynthetic landfill lining systems for correctly engineered and safe design of landfills. Cover soils on geomembranes placed above the waste as in landfill caps as well as lined side slopes beneath the waste are usual methods of constructions. As it is of basic economical interest to gain storage volume the slopes have to be designed as steep as possible.

Many failures have already been described, which shows that the problem and its boundary conditions is still not fully understood by every consulting engineer.
The stability of slopes has to be checked especially for failure mechanisms of which the sliding plane lies completely or partially within the contact plane of geosynthetics.
Two approaches do exist for this problem : [1]

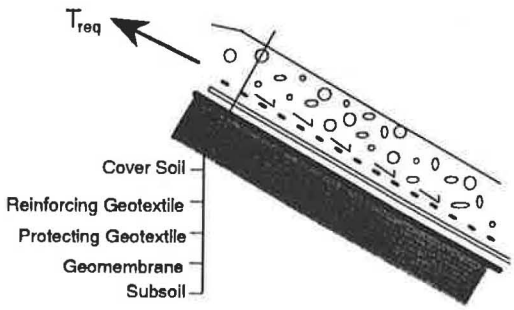
- Stability check of cover soil above a geomembrane
- Stability check for reinforcement of cover soil on a geomembrane

Whereas in the first case only the friction between geomembrane, protective layer and cover soil is regarded, in the second case a high modulus geotextile or geogrid is used.

2. DESCRIPTION OF THE PROBLEM

The main problems which have occured within the last few years with waste disposal slopes are the following:

- Uncomplete trial tests which often are not adequate to the real situation
- Use of materials not fulfilling the requirements of the specification



To reduce these difficulties, questions of stability have to be addressed and carefully studied. This paper is focussing on the stability of inclined lining systems for waste disposals and gives information about the checking of allowable shear stresses. Limit equlibrium calculations are performed, whereas deformations and servicability limits are not addressed.

3. BASIC LABORATORY TRIALS

3.1. Test Materials

The shear resistance between geomembranes and soil respectively between geomembranes and geotextiles can be described analytically by the use of Mohr´s or Coulomb´s failure condition. The parameters are the so called Mohr respectively Coulomb shear parameters. The test values are obtained by direct shear evaluation in simulated laboratry tests. As many aspects of these tests have not yet been standardized, many important details still need to be clearly defined and results from laboratory test cannot be transferred directly to design.

In numerous tests various nonwoven continuos filament needlepunched geotextiles have been used to determine their friction behaviour against various geomembranes and against soil.

For the shear test two different types of geotextiles have been used: A 500 g/m^2 and a 800 g/m^2 continuos filament nonwoven geotextile. The results show, that the weight of the geotextiles is of no significant influence on the shear behaviour.
Geomembranes with smooth and textured surface (sand rough and scrim („wafer-type") surface) were used.

Table 1 Properties of clay

Property	Symbol	Value
Solid Unit weigth	ρ	2,63 g/cm^3
Proctor density	ρ_{pr}	1,52 g/cm^3
Water cont. at ρ_{pr}	w_{pr}	25,4 %
Max. Grain diam.	d_{max}	2,0 mm
Water cont. at liqu. limit	w_l	63,6 %

Table 2 Properties of silt

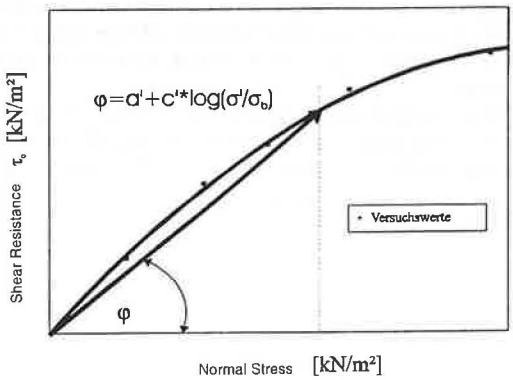
Property	Symbol	Value
Solid Unit weigth	ρ	2,73 g/cm^3
Proctor density	ρ_{pr}	1,75 g/cm^3
Water cont. at ρ_{pr}	w_{Pr}	15,5 %
Max. Grain diam.	d_{max}	2,0 mm

In the frame of these studies three types of tests have been executed to study the contact shear strength between soil and geosynthetics as well as between different geosynthetics:

- Shear box tests with 100 mm diameter
- Shear box tests with 100 mm x 100 mm
- Large frame shear tests (500 x 500 mm)

3.2 Evaluation of tests

If different shear tests at different normal stresses are performed, the obtained values lie on an arched curve, which goes through the 0 Point.



An analytical possibility to describe this curve is the Mohr failure criterion:

$$\varphi = a' + c' \times \log (\sigma' / \sigma_b) \qquad (1)$$

where φ is the shear angle, σ_b is an assumed relation-stress (here set to 60 kN/m^2). The Mohr-shear parameters have to be defined by tests. a' is the shear angle at $\sigma' = \sigma_b$ and c' describes the dependance of the shear angle to the normal stress.
For the practical application of equation (1) σ' is calculated and then by the use of equation 1 the shear angle can be calculated. The shear resistance is then

$$\tau_f = \sigma' \times \tan \varphi \qquad (2)$$

To show the results of the tests clearly and easy understandable design charts have been worked out. From the numerous tests some typical values are shown in table 3:
Polyfelt TS 006 was used as protecting geotextile in combination with a smooth, 2,0mm geomembrane (GM-S). Additionally the shear behaviour between

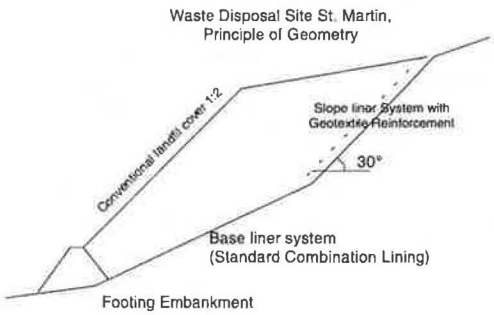
Table 3 Typical shear parameters obtained in shear tests

Materials used	d_{wet}	d_{dry}'
TS 006 - GM-S	9,3°	8,0°
Silt - GM - S	21°	
Clay - GM - S	21°	
PEC/PEC - Sand	30,7°	

the drainage material and the reinforcing geotextile (Polyfelt PPC) was studied.

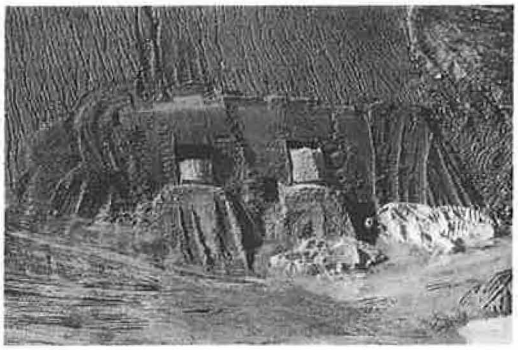
4. FIELD TRIALS AND CALCULATION FOR COVER SOIL ABOVE A GEOMEMBRANE

In the waste disposal site of St. Martin in Austria a total volume of waste of 750.000 m^3 has to be stored. The lining system was designed as a double liner consisting of a three-layered mineralic liner and a 2 mm HDPE geomembrane liner, which is protected by an 800 g/m^2 continuous filament PP - geotextile.
The special topographic situation of the site required special consideration of the internal stability of the waste and the lining system. Although some experiences from abroad with steep and long slopes of pit waste disposal sites do exist, the special situation of a hillside-slope landfill made a careful examination necessary.
For this specific project full scale on site testing has been done to proove the assumptions of landfill slope stability. The shear parameters between the single elements of the lining system were controlled by a model on site.



In cooperation with the Institute of Soil mechanics, rock mechanics and Subsoil Engineering at the University of Innsbruck exhaustive laboratory shear tests have been done.

The most important results from the comparison between the laboratory tests were that the shear results differed between the field tests and the laboratory tests. This effect is mainly due to the fact, that the field tests have been done at low normal stresses, which again proves the assumptions made in 3.2.
In any case the lowest friction was achieved between the geomembrane and the protective felt.



In this case the Factor of safety for the stability of the cover soil (drainage gravel) was checked by the equations from Koerner [1]

$$FS = [- b \pm v(b^2 - 4ac)] / 2a \text{ where}$$

$$a = 0{,}5\, \gamma L H \sin^2 2\omega$$

$$b = - [\gamma L H \cos^2 \omega \tan \delta \sin (2\omega) + c_a L \cos \omega \sin (2\omega) + \gamma L H \sin^2 \omega \tan \phi \sin 2\omega + 2 c H \cos \omega + \gamma H^2 \tan \phi]$$

$$c = (\gamma L H \cos \omega \tan \delta + c_a L) (\tan \phi \sin \omega \sin 2\omega)$$

where:
- γ unit weight f soil
- L length of the slope
- H thickness of soil cover on the slope
- ω slope angle
- δ the lowest angle of the system shear resistance
- C_a adhesion of soil to geocomposite
- ϕ angle of soil to soil shearing resistance
- C soil to soil cohesion

For $\omega = 30°$, $L = 35$ m, $H = 0{,}2$ m, $\gamma = 17$ kN/m^3 ,

$c = 20\ kN/m^2$, $c_a = 0$, $\phi = 42°$, $\delta = 21°$ a factor of safety slightly > 1 was obtained. For a non-permanent structure this was regarded as sufficient.

5. PROPOSED METHOD OF CALCULATION FOR GEOTEXTILE REINFORCEMENT

For the waste disposal site of Hehenberg inAustria a more than 66 m long slope-reinforcement inclined at 1:2 had to be designed.
The design calcution was decided to be done according to the proposals of Koerner [1].
The loading was due to a 0,5 m thick layer of drainage gravel at a density of 19 kN/m^3. Due to exceptional geometric and loading conditions it was decided to build a geosynthetic reinforced structure to keep away any stress from the lining system. The calculation for T_{reqd} is descreibed in the following:
The value of T_{reqd} represents the strength required to keep the cover soil stable with a factor of safety of 1.

$$T_{reqd} = [\gamma L H \sin(\omega - \delta)] / [\cos \delta] - c_a L - \cos \phi [c H / \sin \omega + (\gamma H^2 / \sin 2\omega) \tan \phi] / \cos(\phi + \omega)$$

where:

T_{reqd}	reinforcement tensile strength requirement
γ	unit weight f soil
L	length of the slope
H	thickness of soil cover on the slope
ω	slope angle
δ	the lowest angle of the system shear resistance
C_a	adhesion of soil to geocomposite
ϕ	angle of soil to soil shearing resistance
C	soil to soil cohesion

For $\omega = 27°$, $L = 66$ m, $H = 0{,}5$ m, $\gamma = 19\ kN/m^3$, $c = 15\ kN/m^2$, $c_a = 0$, $\phi = 32°$, $\delta = 20°$ a required design tensile strength of 48 kN/m was obtained. Due to the temporary function a high modulus non woven with a tensile strength of 100 kN/m (Polyfelt PPC 100) was chosen.
PPC 100 a reinforcing geotextile consisting of a mechanically bonded continuos filament nonwoven, which is reinforced by biaxially orientated yarns. The raw material of these yarns is high-modulous Polypropylene. For reasons of accurate specification not only the rupture strength, but also strength and certain strain rates (e.g. 2%, 3% of rupture elongation) were specified.

6. CONCLUSION

The design proposal presented in this paper emphasizes the importance of evaluating shear stress conditions above and underneath geosynthetic liners. Correct design always has to rely upon laboratory test results, especially for questions of stability. It has to be shown that the shear stress above the liner is less or equal to the shear stress below, in order to minimize the geomembrane tensile stress.
By applying the design by function concept two examples illustrate the unique ability of high modulus geotextiles to provide veneer stability for landfill slope design.

7. REFERENCES

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