Slope Stability of Final Covers Containing Geosynthetic Clay Liners

H. -Y. Shan
National Chiao Tung University, Hsinchu, Taiwan
D. E. Daniel
University of Texas, Austin, TX, USA

ABSTRACT: The use of geosynthetic clay liners (GCLs) in waste disposal facilities is becoming more common. However, the bentonite component of the GCLs is known to undergo reduction of shear strength upon hydration. As a result, placing GCLs in final covers on slope raises question about the stability. Laboratory tests were performed to study the shearing behavior of GCLs. In addition, finite element computations were carried out to evaluate the possible long-term deformation of the final covers on slopes containing GCLs. The computations showed that with the use of geosynthetic reinforcements, not only the stability of the final covers can be increased but the deformation can be greatly reduced. In addition, based on the result of computations, a modified limit equilibrium method was proposed for final cover design.

1 INTRODUCTION

In recent years geosynthetic clay liners (GCLs) have been increasingly used as hydraulic barriers in facilities for various purposes. The most important application for GCLs is in the lining systems and final covers of waste disposal facilities (Figs 1 and 2). The hydraulic properties of GCLs have been tested very extensively. However, limited information available shows that the shear strength of GCLs is the major concern regarding the stability on slopes which, in turn, seems to be hindering their potentially wide-range applications. The objective of the study described in this paper is to obtain a better understanding of the shearing properties of GCLs and at the same time develop a mathematical model to evaluate the performance of the GCLs on slopes. On the other hand, the mathematical model was implemented with finite element method.

2 BACKGROUND

Geosynthetic clay liners are a group of manufactured clay blankets. The designation GCL-1, GCL-2, GCL-3, and GCL-4 is used to represent the four GCLs that are currently manufactured, i.e., Bentofix®, Bentomat®, Claymax®, and Gundseal® in the remainder of this paper. These GCLs can be divided into two major categories: (1) bentonite sandwiched between two geotextiles, and (2) bentonite mixed with an adhesive and glued to a geomembrane (Fig. 3). All GCLs contain approximately 5 kg/m^2 of bentonite and the thickness before hydration is about 5 mm. Some of the GCLs are internally reinforced with fibers needle punched through the components to increase their resistance to shear.

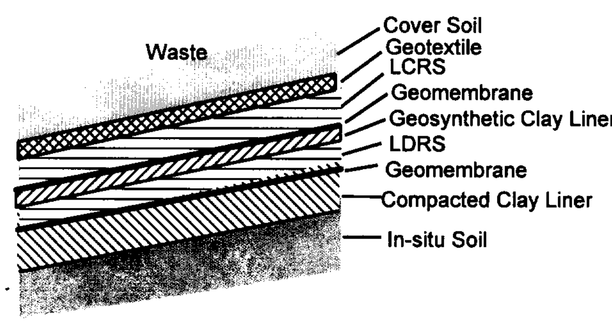

Fig. 1 Hazardous Waste Landfill with GCL in Primary Composite Liner

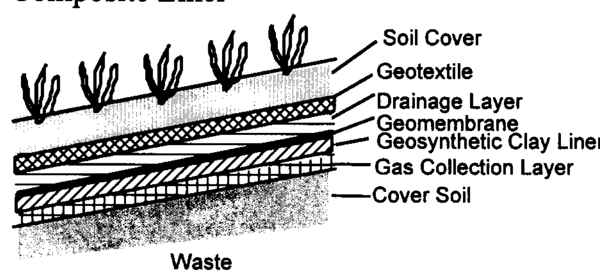

Fig. 2 Final Cover with GCL in Composite Hydraulic Barrier

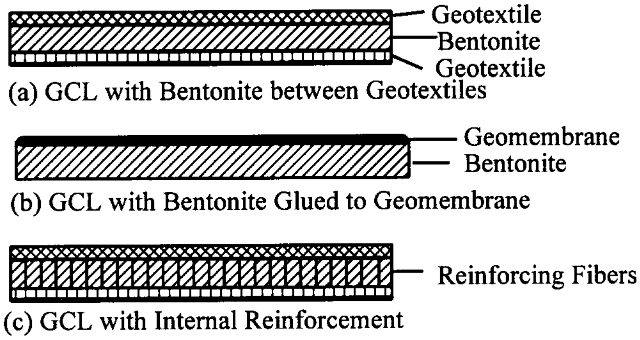

Fig. 3 Various Types of Geosynthetic Clay Liners

Properties of GCLs have been compiled by Daniel and Estornell (1990) and Daniel and Boardman (1992). The amount of information on shearing properties in these documents is relatively small when compared to hydraulic properties. Nevertheless, it was clearly indicated that the problem that causes most concern is the reduction of shear strength of GCLs upon wetting. The internal friction angle of dry GCLs is around 35° but decreases to below 10° when the GCL becomes fully saturated.

Currently, design of final cover is usually accomplished by using either the infinite slope method or passive wedge method (Giroud and Beech, 1989; Koerner and Hwu, 1991) which both are based on limit equilibrium analysis (LEA). Preliminary investigations of this study found that these methods yield very different results under most conditions. Use of more rigorous methods for analyzing the stability of the final cover seems warranted. Boschuck (1991), Gilbert et al. (1993), and Wilson-Fahmy and Koerner (1993) have developed computer codes based on finite element method or finite difference method for studying the stability of soil vaneers. However, due to the limitations of particular models used in these codes, they are not suitable to model final covers that contains GCLs.

3 LABORATORY TESTS

GCLs from all four manufacturers have been tested. The combination of laboratory tests is listed in Table 1. One dimensional consolidation tests were performed to determine the shearing rate of direct shear tests. Due to the thinness of GCLs, direct shear test seems to be the appropriate tool for determining the shear strength of GCLs. The size of the specimens sheared is 60 mm in diameter. The tilt table tests were performed to determine the shear strength of GCLs under low normal stress. The size of the specimens for the tilt table tests was 460 mm x 460 mm. The normal load was provided by the 25.4 mm thick steel plates that put on top of the specimens. The relationship between displacement of GCLs and the tilt angle obtained from the tilt table tests are shown in Fig. 4.

The results of the shear tests are shown in Fig. 5. The internally reinforced GCL-2 has a friction angle of 26 degrees, but the friction angle of the GCLs without reinforcement is only approximately 10 degrees. Note that when the results of the tilt table tests are considered alone, the cohesions are almost zero and the friction angles are 16 and 25 degrees for GCL-3 and GCL-4, respectively.

Table 1 Laboratory Tests Performed on GCLs

Test	GCL-1	GCL-2	GCL-3	GCL-4
Consolidation	yes	yes	yes	yes
Direct Shear	NP	Saturated	Dry Saturated	Dry Saturated
Tilt Table	NP	NP	Saturated	Saturated

*NP means no tests was performed.

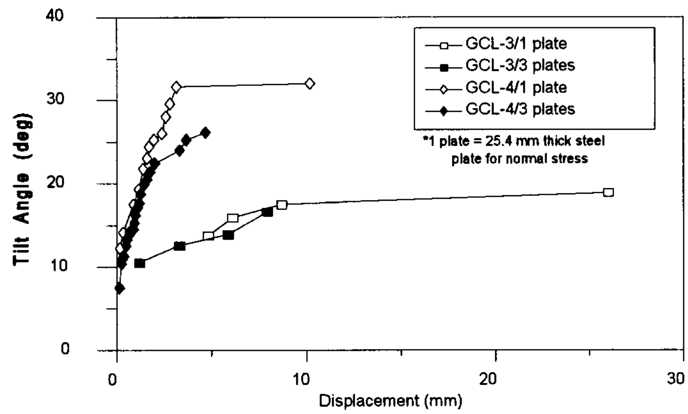

Fig. 4 Results of Tilt Table Tests

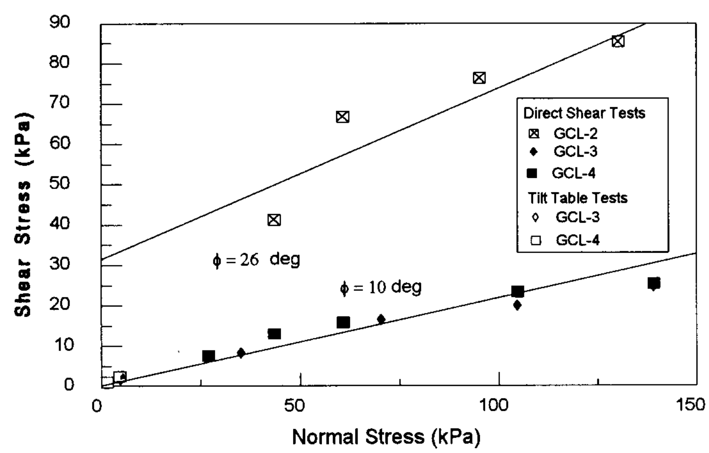

Fig. 5 Mohr-Coulomb Diagrams of Hydrated GCLs

4 NUMERICAL MODEL

The final cover was simulated with numerical models which were implemented with finite element method (FEM). Three types of elements were employed to model various components of the final cover. Soil components of the system such as top soil, drainage layer, and foundation were modeled with solid elements. Hyperbolic model (Duncan and Chang, 1970) and modified Cam-clay model were used for the constitutive relationship of soil in the short-term and long-term conditions, respectively. The GCL and interfaces between different components were simulated by interface elements (Goodman et al., 1968). The element were assumed to behave as elastic-perfectly plastic interface. Before shear strength is reached, the shear stress-displacement relationship was assumed to follow the hyperbolic stress-strain model (Duncan and Chang, 1970). After the shear strength is reached the shearing resistance was assumed to be fixed and relative displacement can continue without further increase in shear stress. Bar elements were used to model the geosynthetic components such as geogrid, geotextile, and geomembrane. The geosynthetics were assumed to behave linear-elastically and can not resist compressive force.

The profile and boundary conditions of the final cover systems modeled are shown in Fig. 7. When reinforcement is used, it is placed directly above the GCL in FC-1 or geomembrane in FC-2. Unless otherwise indicated, the length of final cover along the slope used for computations is 30.3 m (100 ft). The simulation was performed in two steps. First, the construction deformation of the final cover was computed by assuming the GCL was dry. Long-term deformation is assumed to be caused by the hydration of GCL and properties for the saturated GCL were used. The parameters related to the material properties are listed in Table 2. While the parameters for GCLs were obtained from the laboratory test resutls, the properties of the geosynthetics other than GCL, interfaces, and soils were assumed.

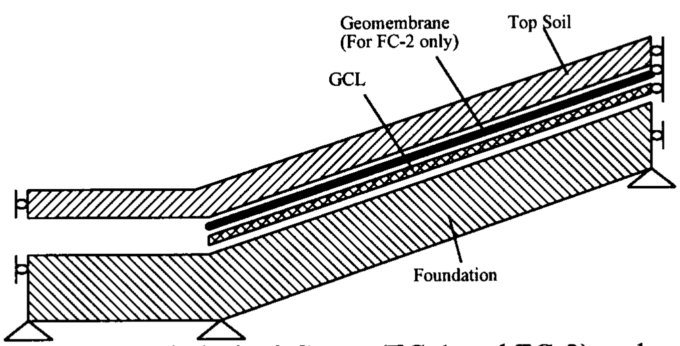

Fig. 7 Modeled Final Cover (FC-1 and FC-2) and Boundary Conditions

Table 2 Material Properties for the Numerical Models
(a) Parameters for the Hyperbolic Model for GCLs

	Dry	Saturated GCL-A	Saturated GCL-B	Saturated GCL-C
c (kPa)	12.0	0	0	0
ϕ (deg)	36	15	20	25
Modulus number, K (kPa/m)	408	58	11	31
Modulus exponent, n	0.56	0.82	1.1	1.0
Failure ratio, R_f	0.5	0.7	0.7	0.6
Normal stiffness, k_N (kPa/m)	235,500	23,550	23,550	23,550

(b) Parameters for Geomembrane-Soil Interfaces

Adhesion, c_a (kPa)	0
Friction angle, δ (deg)	25
Modulus number, K (kPa/m)	126
Modulus exponent, n	0.8
Failure ratio, R_f	0.8
Normal stiffness, k_N (kPa/m)	235,500

(c) Parameters for the Hyperbolic Model for Soil

Parameter	Top Soil	Foundation
Cohesion, c (kPa)	12	24
Friction angle, ϕ (deg)	20	15
Unit weight, γ (kN/m^3)	18.9	18.9
Modulus number, K (kPa/m)	12.6	12.6
Modulus exponent, n	0.8	0.8
Failure ratio, R_f	0.8	0.8
Poisson's ratio, ν	0.45	0.45

(d) Parameters for the Cam-Clay Model

Parameter	Top Soil	Foundation
Cohesion, c (kPa)	2.4	4.8
Friction angle, ϕ (deg)	30	25
Unit weight, γ (kN/m^3)	18.9	18.9
Λ	12.6	12.6
κ	0.8	0.8
M	0.8	0.8
Poisson's ratio, ν	0.45	0.45
p_0	96	144
e_0	0.8	0.8
Initial void ratio, e_i	1.0	1.0

5 RESULTS AND DISCUSSION

According to the results of computations, the deformation of the final cover is mainly due to the shearing displacement within the GCL. As shown in Fig. 8, the displacement of the surface of the final cover is very close to the displacement within the GCL.

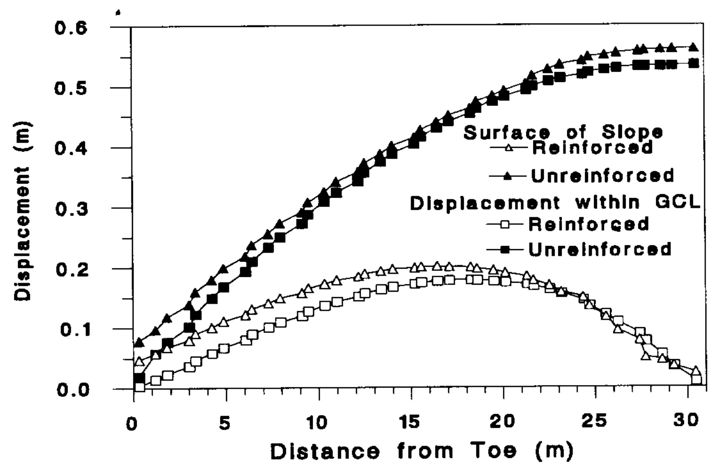

Fig. 8 Comparisons of Typical Displacements within GCL-A and on Top of the Final Cover (3:1 Slope)

As shown in Fig. 9, the displacement of GCL in final covers increases with the slope angle. The displacement remains small before the slope angle becomes greater than the friction angle. For final cover containing GCL-A, when geosynthetic reinforcement was included in the final cover, the deformation is effectively reduced. In addition, the location of maximum displacement of GCL occurs near the middle of slope since the reinforcement was anchored at the top of the slope (Fig. 8). The computation also showed that the reinforcement starts to take tension near the location where GCL displaces most below which there is no tension developed (Fig. 10). It has to be noted that short-term deformation only accounts for a very small part of the total deformation and that about 3 kN/m of tension in the reinforcement developed during the construction. In addition to reinforcing, it was found that shortening the final cover on slopes can greatly reduce the deformation (Fig. 11).

In the reinforced final cover FC-2 which contains a composite hydraulic barrier of geomembrane and GCL, the reinforcement not only limited the displacement but also reduced the tension in the geomembrane. In addition, the higher the elastic modulus of the reinforcement the smaller the displacement (Fig.12). In the meantime, tensile strain in both the reinforcement and geomembrane decreased (Fig. 13) but the tensile load in the reinforcement became larger since higher stiffness made it take more down slope loading off other components (Fig. 14). On the other hand, using geomembrane of low elastic modulus would hardly increase the deformation of the final cover but could effectively reduce the tension in itself (Figs. 15 and 16).

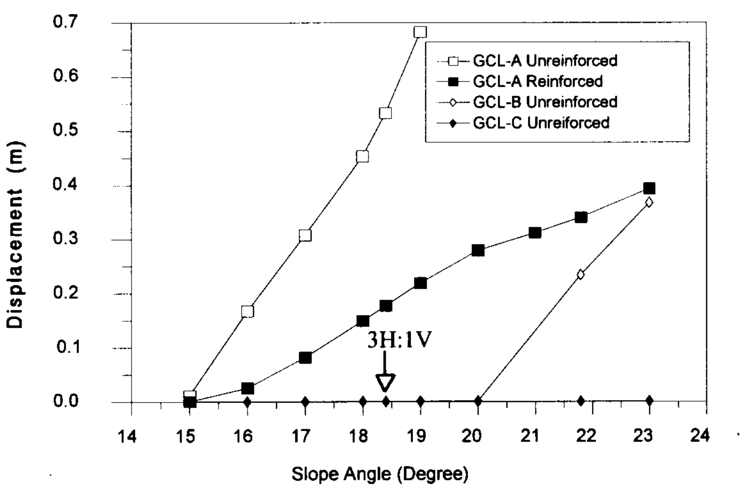

Fig. 9 Variation of Displacement of GCL with Slope Angle

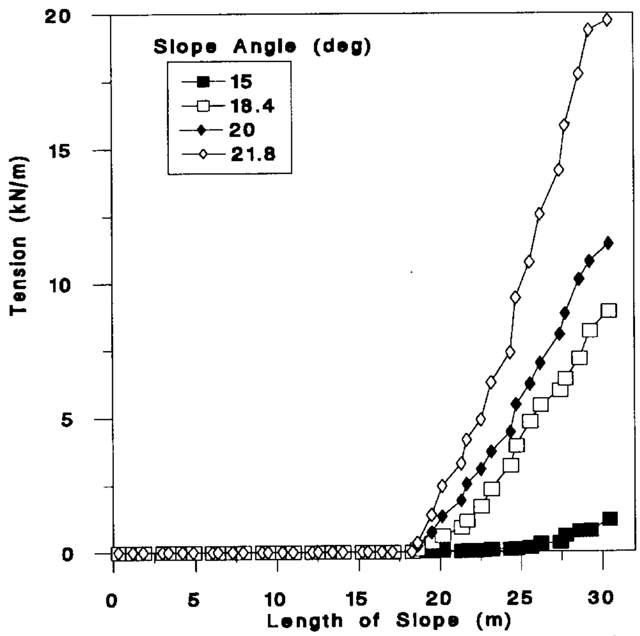

Fig. 10 Variation of Tension in Reinforcement along Slope

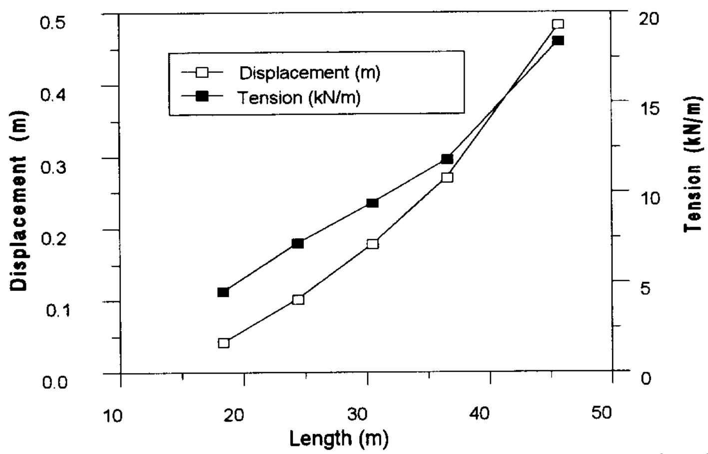

Fig. 11 Variation of Displacement of GCL and Tension in Reinforcement with Length of Final Cover (3H:1V, GCL-A)

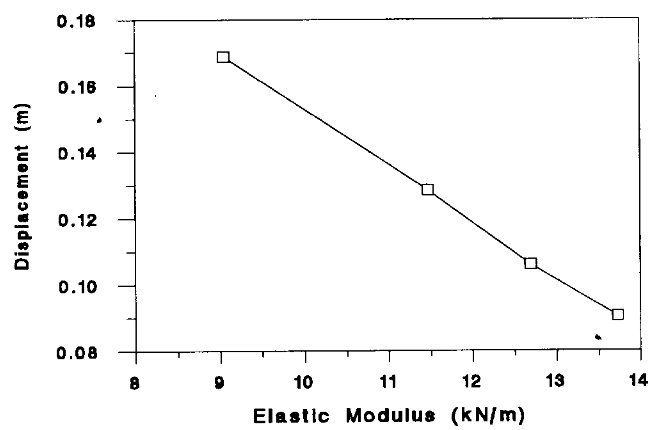

Fig. 12 Variation of Displacement in GCL with the Elastic Modulus of Reinforcement (GCL-A on 3:1 Slope)

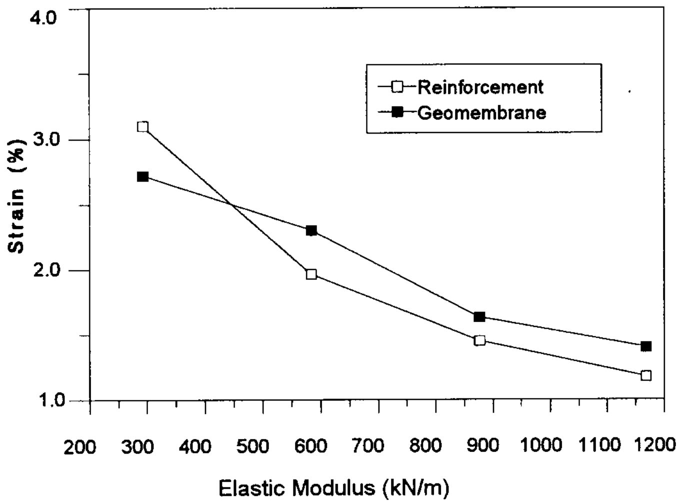

Fig. 13 Variation of Strain in Geosynthetics with the Elastic Modulus of Reinforcement (GCL-A on 3:1 Slope)

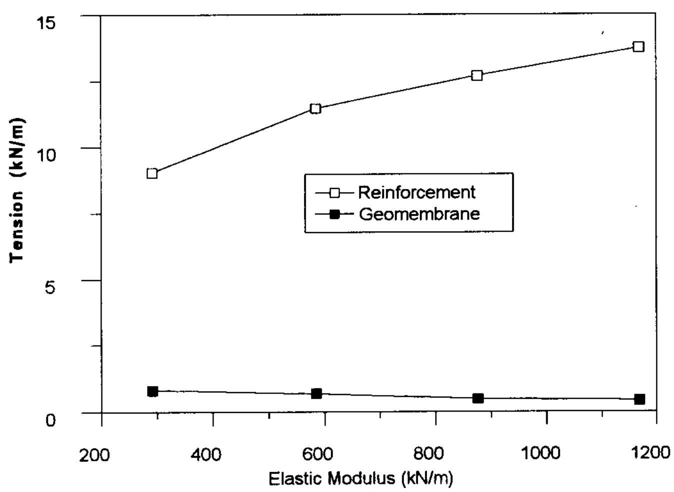

Fig. 14 Variation Tension in Geosynthetics with the Elastic Modulus of Reinforcement (GCL-A on 3:1 Slope)

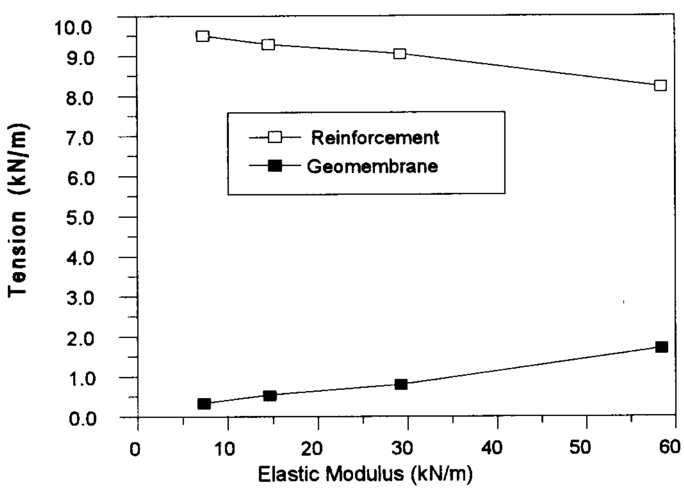

Fig. 15 Variation of Tension in Geosynthetics with the Elastic Modulus of Geomembrane (GCL-A on 3:1 Slope)

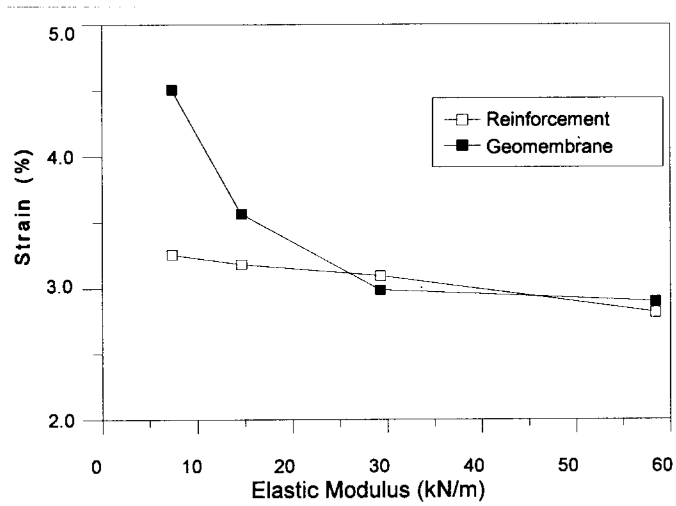

Fig. 16 Variation of Strains in Geosynthetics with the Elastic Modulus of Geomembrane (GCL-A on 3:1 Slope)

To examine the current design procedures, the tensile forces in the reinforcement for FC-1 are compared with the values computed with infinite slope method (Tensar, 1989) and passive wedge method (Koerner and Hwu, 1991) (Fig. 17). It seems that the results obtained by these two methods based on limit equilibrium analysis can be treated as the upper bound and the lower bound values. Ignoring the passive resistance lead to overestimation of the tensile forces in the geosynthetics. On the other hand, in most conditions the passive resistance is not fully mobilized, assuming full passive resistance would underestimate the tensile forces needed for maintaining the stability of final covers. Furthermore, since the geosynthetics at lower part of the final cover remains unstressed, it may not necessary to place the reinforcement along the whole slope but only from half way up the slope. It is suggested that the tensile forces in the geosynthetics can be estimated by integrating the equation based on infinite slope assumptions from half way up to the top of the slope:

$$T = 0.5\, L\, [\, \gamma\, H\, (\sin\beta - \cos\beta \tan\delta) - c_a] \tag{1}$$

where γ is the unit weight of the top soil; β is the slope angle; c_a and δ are the cohesion and friction angle of GCL, respectively. The tension T happens to be half of what the original infinite slope method predicts. The tensile forces computed by this modified limit equilibrium (MLE) method is always on the safe side (Fig. 17). When tension develops in more than one component in the final cover, the tension computed by the MLE method is distributed to the components according to the stiffnesses just as in the system of springs connected in parallel. For FC-2, the tension computed with MLE method seemed satisfactory when compared with FEM computations (Fig. 18).

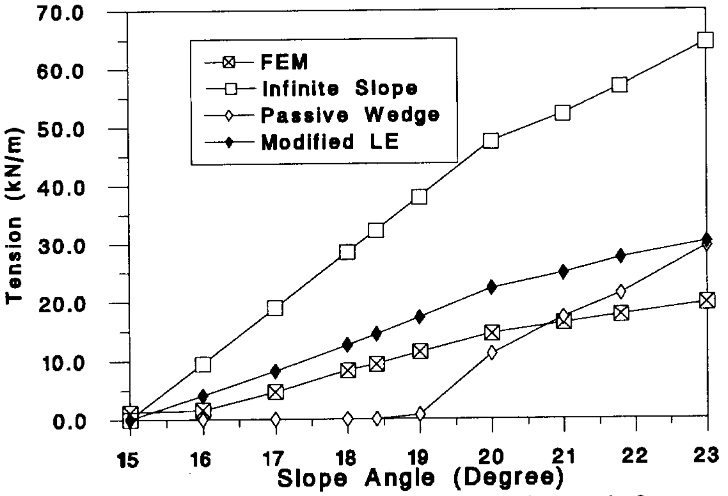

Fig. 17 Comparison of Tension in the Reinforcement Computed by Different Methods

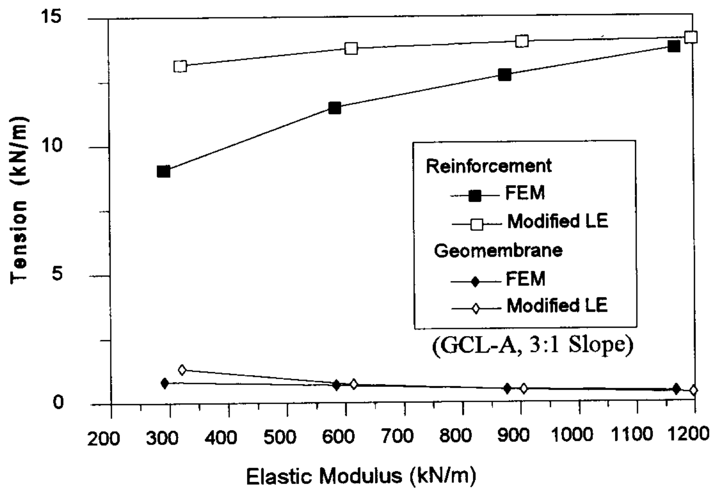

Fig. 18 Comparison of the Tension in Geosynthetics using the Modified Method and Finite Element Method

7 CONCLUSIONS

Using GCLs as the hydraulic barrier in final covers has great promise for future applications. When the final covers containing GCLs with internal reinforcement hydrates, little deformation would occur. On the other hand, the structural stability of the final covers contains GCLs without internal reinforcement can be ensured by incorporating geosynthetic reinforcements into the system. The use of reinforcement also helps limit the magnitude of defomration.

Based on the results of this study, it is recommended that modified limit equilibrium method be used for designing final covers on slopes. In addition, when geosynthetic reinforcement is included it only has to be placed on upper half of the slope. Finally, whenever deformation of final cover has to be estimated for evlauating the long-term performance, it should be obtained by using computer programs developed with sound numerical models.

REFERENCES

Cuenca, F. A., (1990), Stability of Reinforced Embankments Constructed of High Plasticity Clays, Dissertation, University of Texas, U.S.A., 226 p.

Daniel, D. E., and Estornell, P. M., (1990), Compilation of Information on Alternative Barriers for Liner and Cover Systems, U.S.EPA, EPA 600/2-91-002

Daniel, D. E., and Boardman, B. T., (1992), Report of Workshop on Geosynthetic Clay Liners, U.S.EPA, Cincinnati, 100 p.

Duncan, J. M., and Chang, C.-Y., (1970), "Nonlinear Analysis of Stress and Strain in Soils," ASCE, *Journal of Soil Mechanics and Foundation Engineering Div.*, Vol. 96, No. SM5, pp. 1629-1653

Gilbert, R. B., Long, J. H., and Daly, J. J., (1993), "Structural Integrity of Composite Geosynthetic Lining and Cover Systems," *Proceedings*, Geosynthetics '93, Vancouver, Canada, pp. 1389-1401

Giroud, J. P., and Beech, J. P., (1989), "Stability of Soil Layers On Geosynthetic Lining Systems," *Proceeding*, Geosynthetics '89, San Diego, U.S.A., pp. 35-46

Goodman, R. E., Taylor, R. L., and Brekke, T. L., (1968), "A Model for the Mechanics of Jointed Rock, " ASCE, *Journal of Soil Mechanics and Foundation Engineering Div.,* Vol. 94, No. SM3, pp. 637-659

Koerner, R. M., (1990), Designing with Geosynthetics, Prentice-Hall, Englewood Cliffs, N.J., U.S.A., 652 p.

Koerner, R. M., and Hwu, B.-L., (1991), "Stability and Tension Considerations Regarding Cover Soils on Geomembrane Lined Slopes," *Geotextiles and Geomembranes*, Vol. 10, pp. 335-355

Tensar, (1989), Tensar Technical Note: TENSAR Geogrid Reinforcement of Soil Vaneer Covers, TTN:WM2, Tensar Environmental Systems, Inc., Morrow, GA, Draft

Wilson-Fahmy, R. F., and Koerner, R. M., (1993), "Finite Element Analysis of Stability of Cover Soil on Geomembrane-Lined Slopes," *Proceedings*, Geosynthetics '93, Vancouver, Canada, pp. 1425-1437