

Modelling of lining system integrity failure in a steep sided landfill

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ABSTRACT: Landfill design must consider the stability of a lining system and the integrity of its components. When waste settlement occurs, forces are applied to the elements of the lining and drainage system that can result in the development of axial strains in the geosynthetic components. Multilayer geosynthetic lining systems must be designed to withstand and dissipate these forces. In this paper a lining system is analysed using the finite difference modelling code FLAC, to assess the mechanisms behind overstressing and loss of function of the geomembrane in the steep sided lining system of a South East Asian landfill. The model incorporates strain dependant interface shear strength and axial strain behaviour of the geomembrane, staged construction and non linear volumetric and shear behaviour of the waste mass. The model demonstrates that the geomembrane could have experienced stresses greater than yield stresses due to waste settlement, and that large strains in the geotextile protection layer may have resulted in loss of protection.

1 INTRODUCTION

Design of landfills must consider stability and integrity of landfill lining systems (Jones and Dixon, 2005). This paper considers assessment of reported failures that resulted in loss of geosynthetic lining component integrity. Numerical analysis techniques have been applied to assess modes of integrity failure. Two mechanisms are investigated relating to waste/lining system interaction; tensile failure of the geomembrane and loss of protection to the geomembrane resulting from tensile failure of an overlying geotextile protection layer.

The problem analysed in this paper is a benched quarry side with a lining system from a large landfill in South East Asia at which integrity failures have been reported. A hard rock quarry subgrade was lined with a multilayer geosynthetic lining system. The lining system comprised, from the bottom up, a geocomposite drainage layer, a 2 mm smooth HDPE geomembrane, a non-woven protection geotextile, and a 500mm leachate drainage layer (Figure 1).

1.1 Numerical modelling methodology

The numerical modelling code, FLAC^{2D}, has been utilised for modelling of municipal solid waste (MSW) landfills due to its ability to model the large strains occurring within the waste body and displacements

at the lining system interfaces (Fowmes *et al.*, 2005). The geosynthetic liner elements have been modelled as individual beam elements with interfaces defined between them. This allows the transfer of stresses through the lining system to be modelled. The model for municipal solid waste allows compression due to gravitational forces; however, settlement due to time dependant degradation was not modelled.

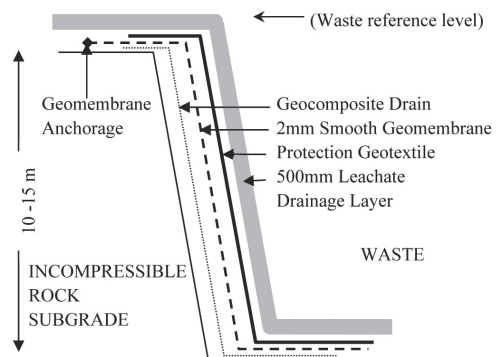


Figure. 1 Schematic of lining system used on rock benched subgrade.

2 NUMERICAL ANALYSIS

2.1 Numerical models

Two models were used in the analysis; the first modelled a full height section of side slope to assess the waste and lining system behaviour on a benched quarry subgrade. The second model looked in more detail at a single section of the side slope in order to assess the behaviour of the lining system in more detail over a single bench height.

For model 1 the finite difference grid (Figure 2) consisted of 4284 mesh zones with an increased model density around the area of the geosynthetic side slope lining system. The lining system was assessed on each of the benches to identify where the overstressing would be greatest. The lining system comprised of three geosynthetic layers that interacted through interfaces. The geosynthetic element behaviour was defined through specifying the material dimensions and Young's Modulus. User defined code was applied to allow strain dependant behaviour in the geomembrane by varying the Young's Modulus as axial strain occurs. Table 1 shows the thickness and peak modulus values.

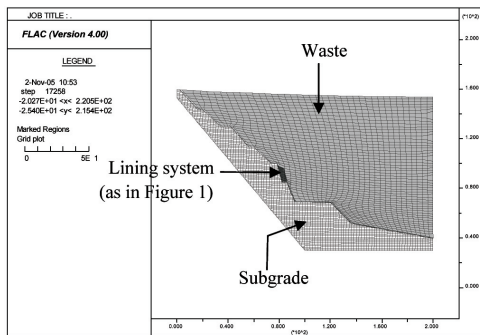


Figure 2. Finite difference grid used in model 1.

In a multilayer lining system it is unlikely that all interfaces will have displaced to mobilised peak shear strength and no post peak strength loss has occurred, hence using a single strength value representing peak or post peak values is unlikely to represent reality. The interfaces between the geosynthetics and those between the geosynthetics and zones representing the subgrade and leachate drainage stone are controlled by user defined code that allows displacement dependant friction and adhesion values to be defined. This allows shear stress vs. displacement behaviour to be defined for each interface. A summary of the interface properties is shown in Table 1.

Model 1 showed that the geometry and height of the benches and slope batters were critical to the behaviour of the lining system but not the number of the waste lifts or their geometry. This is because waste

Table 1. Interface and Geosynthetic properties.

Interfaces Properties	ϕ_{peak}	ϕ_{residual}
Waste vs. Geotextile	32.0°	17.0°
Geotextile vs. Smooth Geomembrane	12.4°	8.2°
GCL vs. Smooth/ (textured) Geomembrane	13.6° (28.7°)	8.2° (14.0°)
Geosynthetic Properties	Thickness	Young's Modulus
Geotextile	5 mm	15 MPa
HDPE Geomembrane	2 mm	150 MPa
GCL	9 mm	30 MPa

is compressed down onto the rock benches more than onto the waste of previous lifts. It was therefore decided to concentrate on a single lift and study the effect of subsequent waste loading on this single waste lift, thus representing the base of the slope. The greatest overstressing was predicted in the steepest side slopes and as a result 85° side slope geometry was analysed in model 2.

Model 2 looked in detail at the behaviour of the benched lining system over a single bench height (Figure 3). Model 2 contained 846 grid zones with 56 individual structural elements to represent the geosynthetic elements, each of which could individually vary in properties controlled by user defined code.

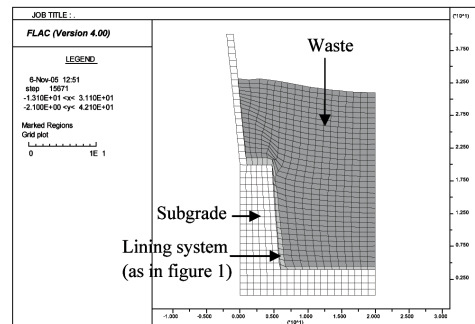


Figure 3. Finite difference grid used in model 2.

This smaller model allowed greater construction detail to be included. Whilst the initial model used a single waste lift for each bench, model 2 represented the construction sequence with 2 m waste lifts and placement of the geosynthetic layers. Once the waste has been placed in lifts up to the height of the bench being studied, two 10 m lifts of waste were added above this to represent the next two benches. Application of pressure to the surface of the waste was then used to represent further waste lifts. The pressure was added in 140 kPa increments to represent 10 m of waste being placed in each.

2.2 Numerical modelling of the waste mass behaviour

Numerical modelling of waste has been discussed by several authors (e.g. Krase and Dinkler, 2005) however there is no readily available material model that can be used in a commercially available large strain numerical code. Therefore, a soil model must be assumed for the waste material and parameters chosen to best fit the waste behaviour. In model 1, a Mohr-Coulomb linear elastic model was used. However, in model 2, a modified Mohr-Coulomb model was adopted with strain dependant shear strength parameters and volumetric yield criterion. This allowed the waste properties to alter as compression of the waste occurred and it became subsequently harder to compact.

The properties used for the waste were as follows: A peak friction angle of 25 degrees, a cohesion value of 10 kPa and a density of 1400 kg/m³. (Cowland *et al.*, 1993). A Young's Modulus of 2 MPa was adopted for the compacted waste with a Poisson's ratio of 0.3. These values allowed generation of vertical compression in the order of 30% in the waste layers under the maximum vertical loading. The model used a strain dependant volumetric yield criterion, such that the material became harder to compress once initial compression had occurred, and also shear strain dependant friction angle and cohesion values were applied. This model only considers gravitational compression under self weight and does not consider time dependant degradation of the waste.

2.3 Anchoring of geosynthetics

When representing the construction of a geosynthetic lining system, anchorage of the geosynthetics is essential when modelling the relative displacements and strains experienced by each of the geosynthetic components.

Three methods of anchoring are possible using this model.

1. Modelling of the actual bench anchoring with its berm/trench structure.
2. Fixing the end of the beam used to represent the geosynthetics.
3. Using a flexible attachment to represent the effects of the anchoring.

Just fixing the end of the beam is simplistic, however, attempting to represent the full effects of the anchoring and all of its components is complex and introduces more uncertain variables, hence a flexible attachment was adopted with bond yield criterion and a geomembrane tensile yield value.

3 RESULTS OF THE MODEL

With the geosynthetic anchorage located on the horizontal section of the benches, the overburden of

waste rapidly exceeds the value at which geosynthetic slip along this plane can occur. Even for relatively low strength interfaces, movement along the interface is restricted by a few metres of waste body located above the bench. The result of this is that the geosynthetics are pinned at the corner of the bench. With subsequent waste loadings, waste settlement occurs resulting in material moving past these pinned geosynthetics, and therefore stresses are induced in these materials. Table 2 shows the stresses and strains

Table 2. Axial strains and tensile forces in the geomembrane related to waste height.

Waste height above bench.	Vertical pressure (kPa) at waste ref level	Maximum axial strain in geomembrane (%)	Maximum tensile stress in geomembrane (kN/m)	Location of max stress (m below top of bench)
0	0	0.14	0.42	3.2
10	140	0.17	0.51	1.2
20	280	0.20	0.60	1.2
30	420	0.20	0.59	1.2
40	560	0.37	1.32	2.4
50	700	0.40	1.43	4.8
60	840	8.37	25.1	3.6
70	980	14.7	44.9	1.2

developed in a smooth geomembrane against the height of waste above reference level (Figure 1).

Figure 4 shows the axial forces and strains in the geomembrane with a vertical stress above the study bench of approximately 1000 kPa. The analysis gives a maximum tensile force and extension just below the bench corner of approximately 44.9 kN/m and

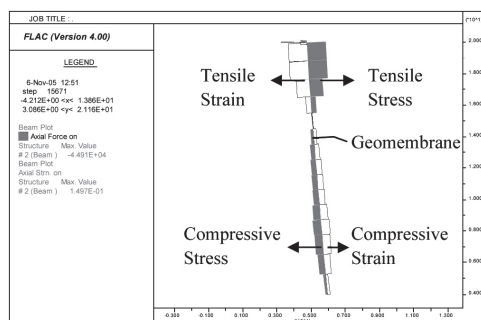


Figure 4. Axial Strains (hollow) and axial force (filled) developed in the geomembrane.

14.7% respectively. This exceeds the 28 kN/m tensile yield strength of the 2mm HDPE geomembrane.

To prevent tensile forces developing in the geomembrane, mobilised forces on the interface above the geomembrane must not exceed the shear strength mobilised on the interface below the geomembrane. When the lower surface of the geomembrane is smooth, slippage occurs at the interface beneath the

geomembrane. Slippage on this interface results in the mobilisation of tensile forces in the geomembrane, and axial strains develop. Large increases in the maximum axial tensile stress and strain values occur when the stress mobilized at the upper interface exceeds interface shear strength of the interface below the geomembrane.

The smooth geomembrane-geocomposite drainage interface has a peak interface friction angle of 13 degrees and a large displacement friction angle of approximately 8 degrees. As a comparison, an analysis was carried out with a mono-textured geomembrane (textured side down). The textured geomembrane generated a peak strength of 29°, thus retarding slip along the interface below the geomembrane and preventing axial tension being developed in the geomembrane.

For an 80 m waste height the axial strains developed in the protection geotextile are predicted to be 76% when a smooth geomembrane is used. When a textured geomembrane surface is used, an axial strain of 72% is still predicted. At these strains geotextile thinning and tensile failure is possible. The result of this may be to expose the geomembrane to the leachate drainage stone leading to geomembrane damage and loss of integrity.

4 DISCUSSION OF RESULTS

To model waste-barrier interaction, the behaviour of the waste must be appropriate. Constitutive models for municipal solid waste have been proposed by several authors; however, this is an area that requires significant further development. The model adopted in this analysis allows for compression under vertical loading, but it does not include time dependant degradation. Deformations due to settling waste may induce additional forces on the lining system where the interface is not perfectly flat.

The analysis carried out has shown overstressing of geosynthetic elements can be problematic, particularly on benched quarry walls. Although the potential for overstressing of geomembranes can be identified using numerical modelling techniques, the exact behaviour of the geosynthetics in terms of localised rupture and tearing of protection layers is not shown.

This method of analysis assumes that the two dimensional profile is continuous and uniform in the out of plane direction, and this may be un-conservative as slope geometric irregularities may add to the potential overstressing of the material.

The interface shear strength between geosynthetics and also mineral materials will have natural variability and the material testing may not give actual strength values (Dixon *et al.*, 2006). The values used in this analysis were best estimates from the site specific interface shear strength tests and literature, and do not represent conservative values or a worst case scenario. As such the strains mobilised in the waste

mass may be significantly greater than those suggested in the model. Also, the modelling carried out here considered a waste mass confined on all sides with no open waste face, and this reduces basal slippage. If horizontal waste movements were to occur, this may increase the vertical movement of waste at the barrier interface and increase the strains in the geosynthetic elements.

The numerical model acts as a simplification and only provides an indication of the deformations and strains that may occur. The modelling of municipal solid waste is one of the greatest uncertainties with regards the settlement that occurs and also the pressure at the waste barrier interface.

5 CONCLUSIONS

Integrity failures predicted by the model are similar to those occurring in the field. As observed in the field, the model predicts the maximum axial tensile strains occur just below the corner of the benches as the waste compresses, inducing axial forces in the multilayer geosynthetic lining system.

When double sided smooth geomembrane was used the model predicts integrity failure of the geomembrane just below the top of the bench. Integrity failure is caused by slippage of the geomembrane resulting in mobilisation of tensile stresses. When a mono-textured geomembrane (textured side down) is used, the axial strains in the geomembrane are greatly reduced as sufficient strength is mobilised in the textured GM-geocomposite drain interface, thus reducing slippage and axial strains from developing.

This modelling technique is a useful tool in assessing the potential for overstressing of geosynthetic layers, including strain dependant nature of interfaces, geosynthetics and MSW.

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