Effect of Landfill Construction Activities on Mobilised Interface Shear Strength

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ABSTRACT: Most geosynthetic interfaces show strong strain softening behaviour, i.e. the interface shear strength reduces with displacements after the peak value has been reached. To date, several authors have established design methods for veneer stability assessments which take into account equipment loading, but no-one has considered the effect of equipment loading on strain softening interfaces. This paper describes an investigation of these construction effects using numerical modelling techniques. A method of modelling strain softening interfaces has been developed, and has been applied previously by the authors to the study of landfill side slope stability during waste settlement. The FLAC finite difference code has been used for all analyses. A typical textured geomembrane/geotextile interface has been used in the analysis. Results are presented of the effects of two common tracked dozers on the development of interface shear strength in the geosynthetic lining system.

1 INTRODUCTION

It has become common practice to use geosynthetics in the construction of landfill containment systems. A range of materials (e.g. geomembranes, geotextiles, geonets, etc.) are used to perform a variety of functions, such as low permeability barriers to leachate and gas, and filter, protection and drainage layers. The introduction of these materials into basal/side slope lining systems and cover systems, results in the presence of relatively weak geosynthetic/geosynthetic and geosynthetic/soil interfaces. The shear strength of these interfaces must be measured and taken into consideration during the design process.

Of particular concern to the designer is that the majority of these interfaces show strong strain softening behaviour (i.e. the interface shear strength reduces with displacement after the peak value has been reached). Many designers now consider strain softening behaviour in the design process by using partial factors on the peak values. However, it is rare that the influence of construction activities are taken into consideration, even though only small displacements at an interface are often required to reduce the shear strength close to its residual value.

One of the main construction effects on the development of interface shear strength (and therefore on the overall stability) is the loading of the lining system by the earth moving equipment. The equipment may take the form of excavators, dozers, graders etc. and each will have different loading characteristic based on whether they are tracked or wheeled vehicles. The use of such equipment on a geosynthetic lined slope will certainly have a detrimental effect on the shear strengths mobilised along the interfaces. The consequence of using common types of tracked dozers on mobilised shear strength is investigated in this paper.

2 BACKGROUND

Several authors have developed methodologies for the inclusion of equipment loading on the overall veneer stability of a geosynthetic lined slope. Druschel & Underwood (1993) consider a sliding block limit equilibrium analysis which takes into account the forces due to equipment loading by the addition of the equipment's self weight, together with a force due to breaking/accelerating. The breaking load is taken to be 30% of the equipment's self weight as suggested by Richardson & Koerner (1987). A more rigorous approach to equipment loading is given by Koerner and Soong (1998) where the actual acceleration of the equipment is used in the analysis. This method also uses the sliding block approach.

Kerkes (1999) criticises the conventional sliding block approach since the passive block at the toe of the slope provides support to the active block irrespective of where the equipment may be located on the slope. Hence, only one global failure scenario is considered and a localised failure in the vicinity of the equipment is not assessed. Kerkes (1999) then proposes a sliding block analysis that considers three (active, central and passive) blocks above the potential failure surface.

In all the above methodologies, no account is taken of the strain incompatibility between the various soils and geosynthetics that make up the lining and cover soil system. Wilson-Fahmy & Koerner (1993) describe a finite element analysis of cover soil on a geosynthetic lined slope. Whilst the model considers the development of interface shear strength with relative displacement, it uses a hyperbolic relationship up to a maximum shear strength (i.e. post peak behaviour is not modelled). In addition, Wilson-Fahmy & Koerner (1993) do not consider the effect of equipment loading on the overall stability.

There is therefore a need to investigate the effect of equipment loading on the mobilised interface shear strengths and consequently on the stability of the slope.

3 DEVELOPMENT OF NUMERICAL MODEL

3.1 *Finite difference code used*

The numerical analysis described in this paper was carried out using the finite difference code FLAC (Itasca, 1993). Dynamic equations of motion are used to give a static solution to a problem; thus the numerical scheme is stable even when the physical system being modelled is unstable, which is ideal for modelling large strain problems. However, even if the procedure is successful at preventing numerical instability, the stress paths taken may not be realistic and so only the end point of an iteration can be relied upon.

An interface between two structural elements at which slippage can occur is represented in FLAC as a normal and shear stiffness between two planes. The normal and shear forces are calculated at each point defined along the interface and an adjustment is then made for the Coulomb shear strength criterion. In other words, the shear force is limited by:

$$F_{smax} = \alpha L + F_n \tan\delta \tag{1}$$

where α = cohesion along the interface, L = effective contact length, δ = friction angle of interface surfaces and F_n is the normal force.

The shear force at the interface calculated by the global stress redistribution within FLAC, F_s, cannot exceed the maximum shear force calculated using the Coulomb strength criterion, F_{smax}. This approach enables strain softening behaviour at interfaces where relative deformations are occurring to be taken into account.

A major factor in the development of the numerical model is the representation of the geosynthetic interface shear strength. Shear strengths, and hence friction angles and cohesion intercepts used to define them, vary with displacement, and this must be modelled to ensure representative shear stresses are mobilised at the geosynthetic interface in response to construction equipment loading.

3.2 *Interface shear strength parameters*

Interface shear strengths are modelled in FLAC using the conventional interface shear strength parameters of friction angle (δ) and cohesion intercept (α), It is possible to vary both δ and α with displacement by entering tables of friction angles and cohesion intercepts against interface displacement. Results of laboratory testing carried out using a 300mm direct shear apparatus (e.g. Jones & Dixon, 1998) provide information on the shear stress vs. displacement behaviour of typical interfaces and therefore input parameters for the numerical modelling. The modified Coulomb equation can be written as:

$$\tau_f = \alpha + \sigma_n.\tan\delta \tag{2}$$

This can be rearranged to give a value for the friction angle:

$$\delta = \arctan\left\{\frac{\tau_f - \alpha}{\sigma_n}\right\} \tag{3}$$

For each test, values of τ and σ are known, however the value of cohesion intercept α has to be estimated. A simple distribution of α with displacement has been assumed, comprising a build-up of α from zero at zero displacement to a peak value, α_p, at the same displacement as the peak shear strength. This is then followed by a reduction to a large strain or residual value, α_r. This residual value was taken to occur at a further displacement corresponding to half the displacement required for peak shear stress. Using this approach, the distribution of mobilised friction angle with displacement can be calculated for each normal stress used in the laboratory. This can then be simplified by dividing the distributions into a series of linear portions to represent a "best fit", e.g. by calculating the numerical mean for the friction angle vs. displacement envelopes.

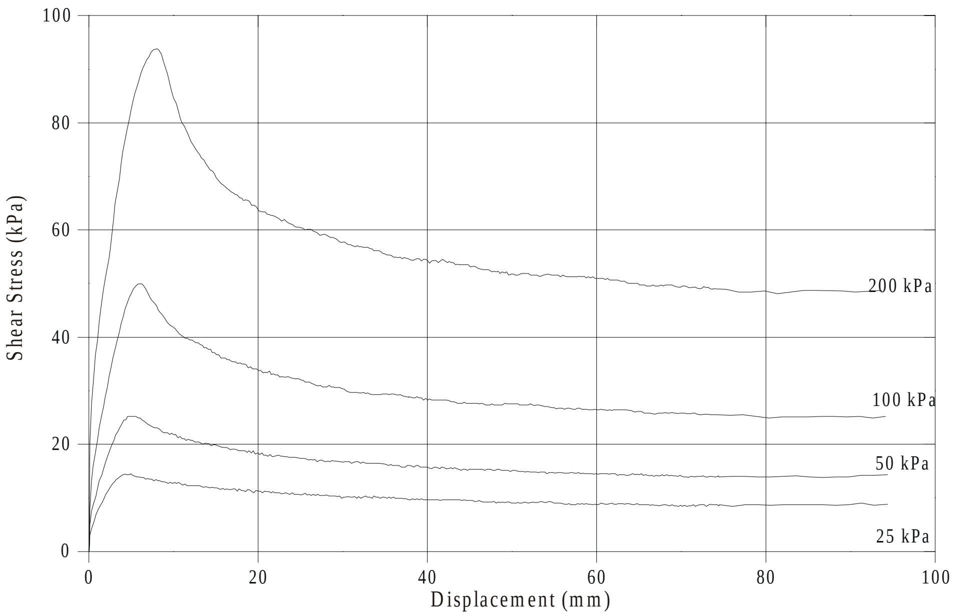

Figure 1. Typical shear stress vs. displacement behaviour for a textured geomembrane/geotextile interface.

An example of the above procedure for the textured geomembrane/geotextile interface used in this analysis follows, based on the typical shear stress vs. displacement curves shown on Figure 1. Linear regression analysis (Figure 2) of the peak and large displacement shear strength values gives the following interface shear strength parameters: δ_p = 24.5° and α_p = 3.2 kPa δ_r = 12.8° and α_r = 2.5 kPa.

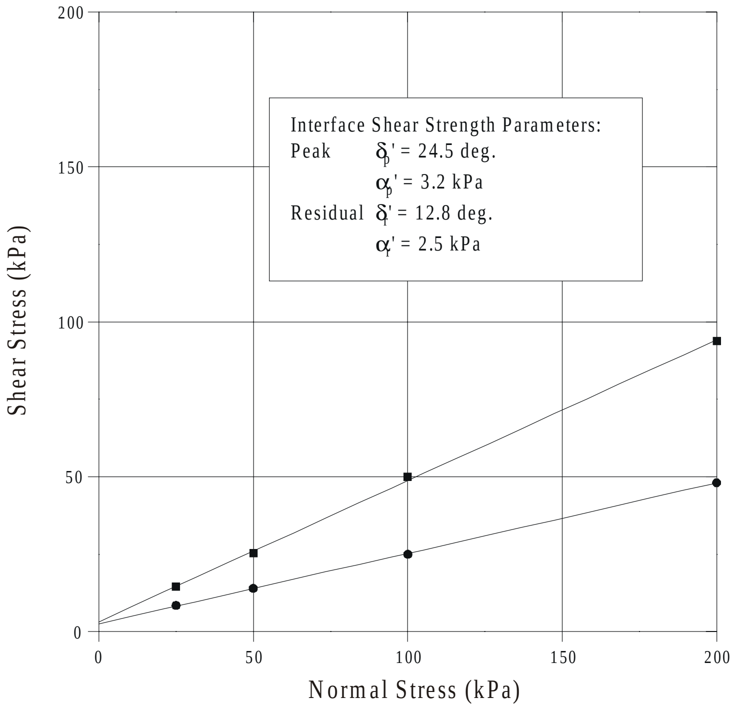

Figure 2. Shear stress vs. normal stress relationship for a textured geomembrane/geotextile interface.

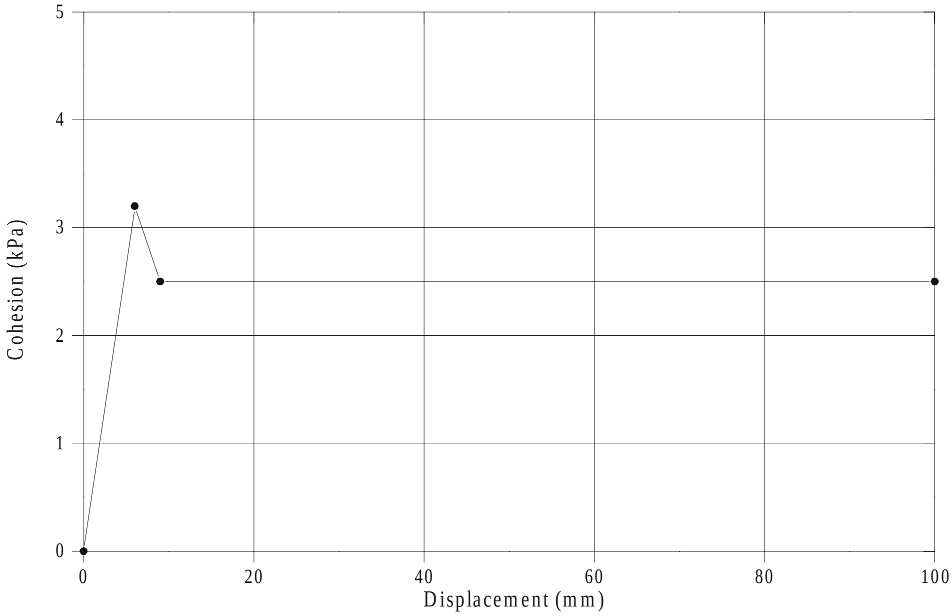

Figure 3. Cohesion distribution used in numerical analysis.

Peak shear stresses are mobilised at displacements of between 4.9 mm and 8.8 mm, with a mean value of 6.2 mm. The cohesion intercept is thus assumed to increase from zero to a maximum of 3.2 kPa at a displacement of 6.2 mm, and falls to its residual value of 2.5 kPa at a displacement of 9.3mm, see Figure 3.

Using the shear stresses from Figure 1 and the distribution of intercept cohesion values (Figure 3) the mobilised friction angle, normalised over the four normal stresses can be calculated, see Figure 4. The four normalised distributions are then simplified into a series of straight lines; in this example, nine linear portions have been used.

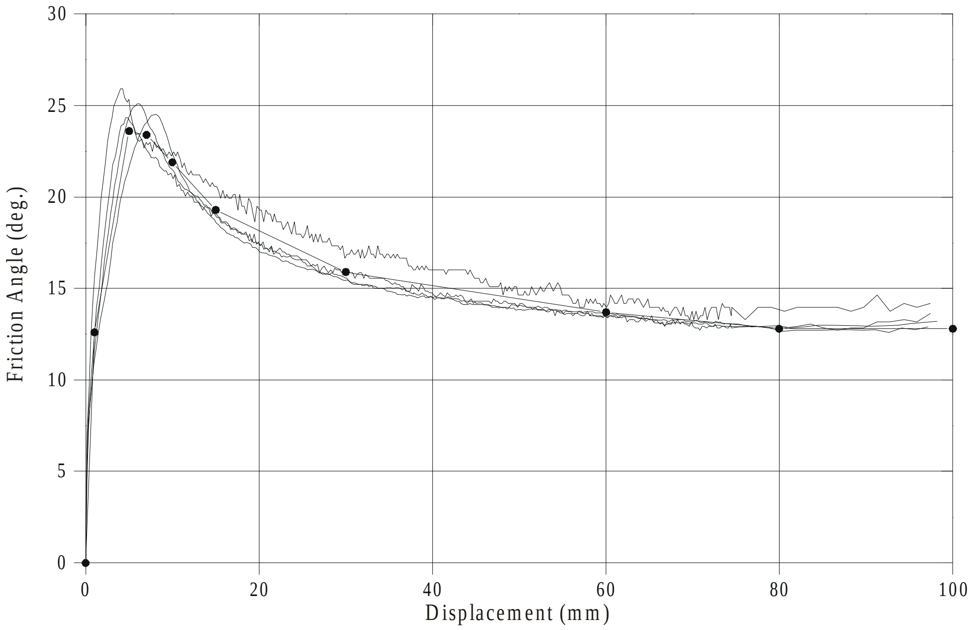

Figure 4. Calculation of friction angle distribution.

3.3 *Mesh generation and boundary conditions*

The barrier system investigated is typical of present UK practice. It comprises a textured geomembrane overlain by a geotextile protection layer. The analysis considers a section of landfill cap or slope on which construction equipment operates. Shear strengths mobilised at the geosynthetic interface are calculated based on the stresses induced in the lining system due to three loading conditions: Weight of cover soil, self weight of the construction equipment and breaking/acceleration of the equipment.

A typical finite difference mesh used in the analyses is shown in Figure 5. All zones within the mesh were free to move in any direction, with the exception of the zone beneath the interface (i.e. the subgrade beneath the geomembrane) which was fixed in both horizontal and vertical directions. This represents the case where the geosynthetic elements of the lining system are underlain by a stiff and stable substrata. The model comprises an interface overlain by a granular cover soil. No relative displacement is allowed to occur at the cover soil/geotextile interface. External loads are applied to simulate the self weight and breaking/acceleration of the construction equipment.

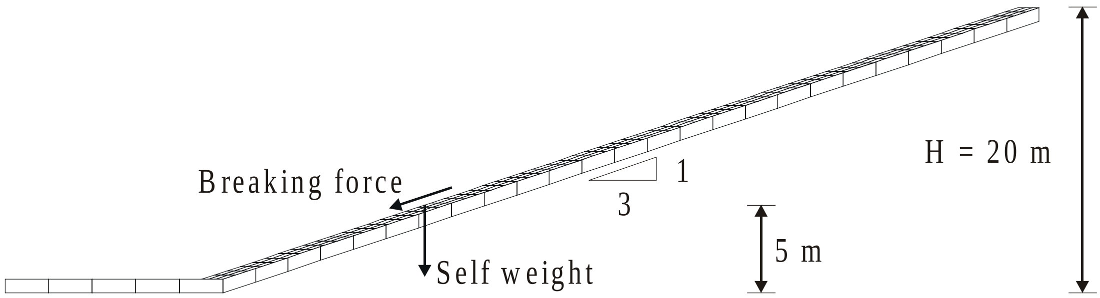

Figure 5. Typical finite difference mesh layout.

Two slopes were considered in the analysis. A 1 (vertical) to 3 (horizontal) slope and a 1 to 2.5 slope; both were 20 m high and the equipment loading was applied at a distance of 5 m vertically from the base of the slope.

3.4 *Loading of the slope*

The first part of the analysis is placement of a 0.5 m deep cover soil layer on the slope, and as the model is stepped towards equilibrium, global stress redistribution applies the self weight of the cover soil onto the geosynthetic interface. The cover soil was modelled as a sand with the properties given in Table 1.

Table 1. Cover soil parameters used in the analysis

Property	Value	Unit
Young's modulus	50	MPa
Poisson's ratio	0.3	-
Density	2000	kg/m^3
Friction angle, ϕ'	35	Degrees
Cohesion, c'	0	kPa

Once equilibrium is reached under the self weight of the soil layer, external loads are applied at the soil surface to simulate the self weight of the construction equipment. Two sizes of tracked dozers have been assessed and the loading details, taken from Caterpillar (2000) are given in Table 2.

Table 2. Description of dozers modelled in the analysis

Dozer type	CAT D4C LGP	CAT D6M LGP
Operating weight	7785 kg	16930 kg
Track on ground	2055 mm	3080 mm
Track width	635 mm	860 mm

The self weight of the dozer was applied simply as a vertical uniformly distributed load over lengths of the cover soil surface of 2 m and 3 m for the D4 and D6 respectively. The breaking force was considered to act parallel to the slope and down slope to give the worst-case scenario. Since only vertical and horizontal forces can be applied in the numerical analysis code used, the breaking force was resolved into these two directions and applied to the mesh once the self weight loading had reached equilibrium.

The operating weight of each dozer was divided equally between the two tracks. This was then pro-rated up to give an equivalent load for a 1 m strip. This assumption is required since the numerical code used was a two-dimensional code and considers plane strain conditions.

4 RESULTS OF NUMERICAL ANALYSIS

4.1 *D4 dozer on a 1 in 3 slope*

The results of the numerical analysis are presented in two ways; firstly the mobilised shear stresses along the interface are given, and secondly the displacements mobilised along the interfaces are presented. Figure 6 shows the calculated shear stresses developed along a 1 in 3 slope. Initially, the placement of a cover soil on to the geotextile induces shear stresses at the geotextile/geomembrane interface in the region of 3 kPa. This is fairly constant along the interface with lower shear stresses at the two extremities of the slope. Inducing the self weight of the D4 dozer results in an increase in mobilised shear stresses to just over 20 kPa in the vicinity of the loaded area. The shear stresses over the majority of the slope are not affected by the application of the dozer loading; it is only a zone around 5 m in length that is affected. The application of a breaking load down the slope actually reduces the maximum shear stress mobilised in this zone, with a peak shear stress of 19 kPa.

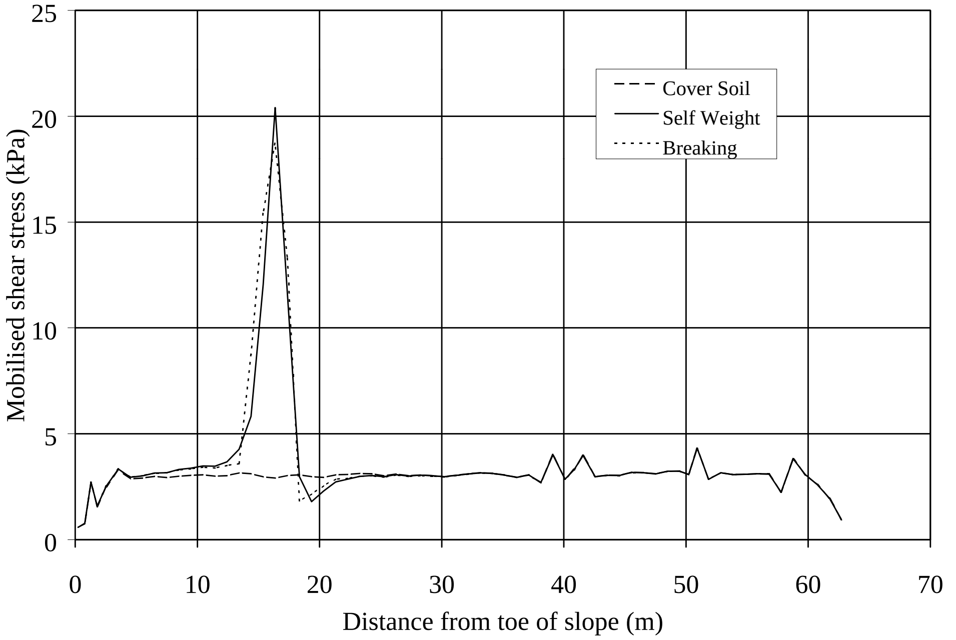

Figure 6. Mobilised shear stress for a 1 in 3 slope with a D4 dozer.

It is not immediately evident why the increased load should produce a lower shear stress. However, if the displacement mobilised at the interface is considered (Figure 7), it can be seen that the breaking force increases the displacement at the interface.

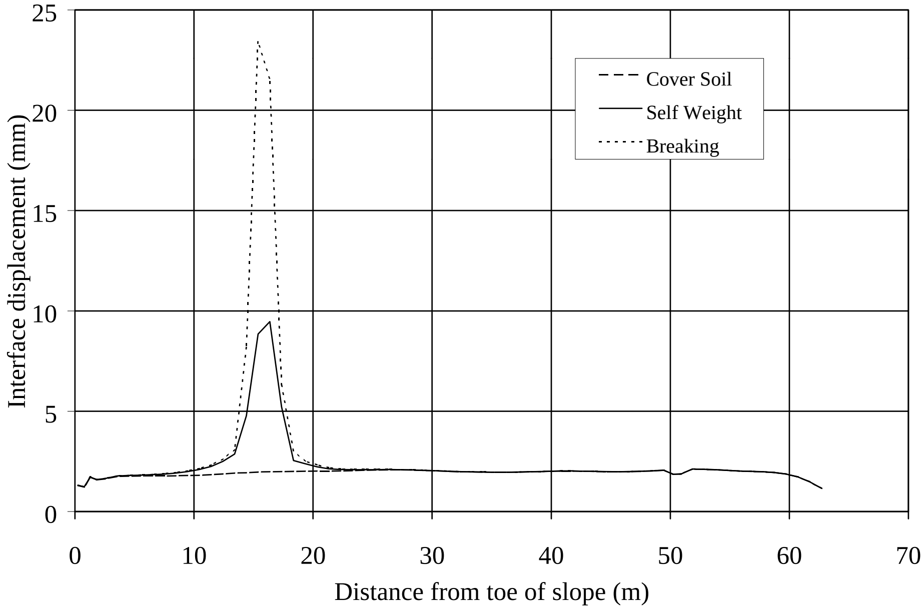

Figure 7. Mobilised interface displacement for a 1 in 3 slope with a D4 dozer.

Placement of cover soil induces around 2 mm of displacement along most of the interface. This is increased over a zone of around 5 m either side of the position of the dozer, with a peak of 9.5 mm at the centre of the dozer. The application of a downward breaking load equal to 30% of the self weight of the dozer increases this peak interface displacement to over 23 mm, see Figure 7.

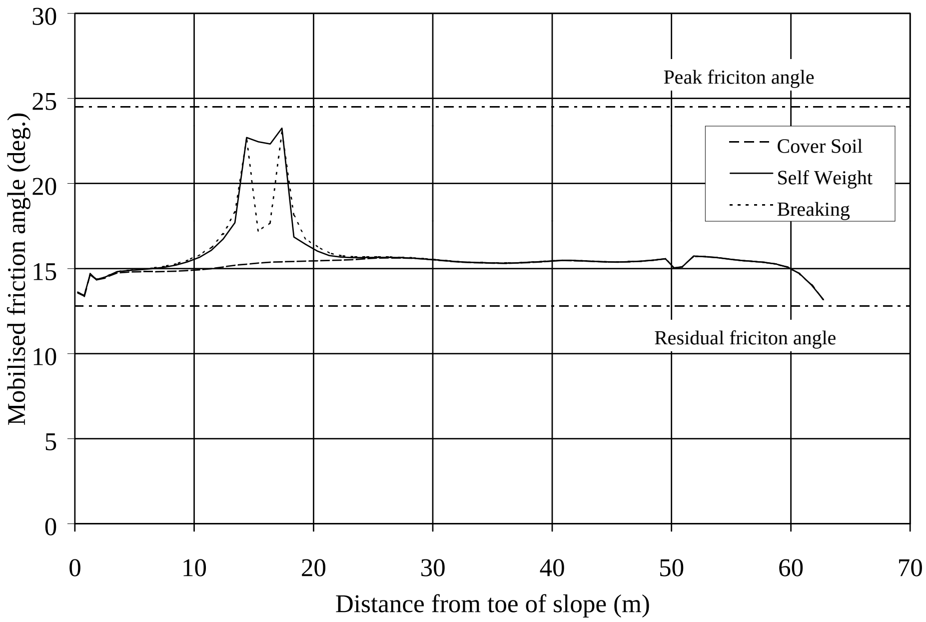

Figure 8. Mobilised friction angle along interface for a 1 in 3 slope with a D4 dozer.

Inspection of the shear stress vs. displacement curves that were used as the basis for the input interface parameters (Figure 1) indicates that the peak shear strengths are mobilised at displacements of between 4.9 mm and 8.8 mm. The peak displacements mobilised during the self weight loading of the D4 therefore mobilise peak interface shear strength at the point of load application. It follows therefore that any subsequent displacements, such as that induced by the breaking force, will mobilise post-peak shear strengths. These post-peak stresses will be less than the peak values, and this explains the reduced mobilised shear stress under the breaking load (Figure 6).The mobilised friction angles along the interface for the three load cases are given in Figure 8.

The application of the D4 self weight, mobilises friction angles very close to peak values in the region of the load application, with post-peak friction angles mobilised at the centre of the loaded area. A breaking force of 30% of the self weight results in a lower friction angle in this zone, as the shear strength of the interface reduces post-peak.

4.2 *Effect of breaking load*

The assumption that the dynamic breaking/acceleration load can be allowed for in deign by considering a static load of 30% of the self weight of the construction equipment has been used by many authors (e.g. Druschel & Underwood, 1993, Kerkes, 1999). This assumption is based however, on original guidelines produced by Richardson & Koerner (1987) which do not seem to be based on definitive research. This investigation therefore looked at a range of breaking loads corresponding to 20%, 30% and 40% of the self weight of a D4 dozer. The mobilsed shear stresses along the interface are all very similar and range from 19 kPa to 20 kPa.

Mobilised interface displacements for the three breaking loads used are shown in Figure 9. The distribution of displacements is the same for the three cases with only the maximum values being different. These peak values are 19.8 mm, 23.4 mm and 25.8 mm for the 20%, 30% and 40% load cases respectively. All three cases therefore mobilise post-peak shear strength at the geosynthetic interface.

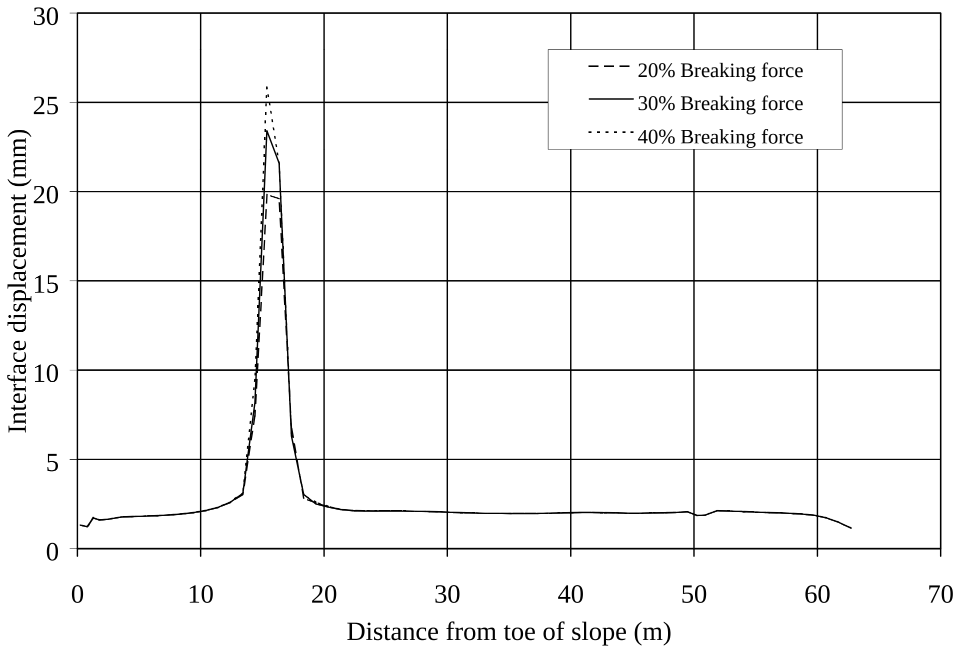

Figure 9. Mobilised interface displacements for a 1 in 3 slope with differing applied breaking loads.

4.3 *Effect of slope angle*

Although landfill capping systems in the UK are generally shallow, slopes with gradients of up to 1 (vertical) to 2.5 (horizontal) are not unknown. Therefore, the effect of steepening the slope on the geosynthetic interface was investigated.

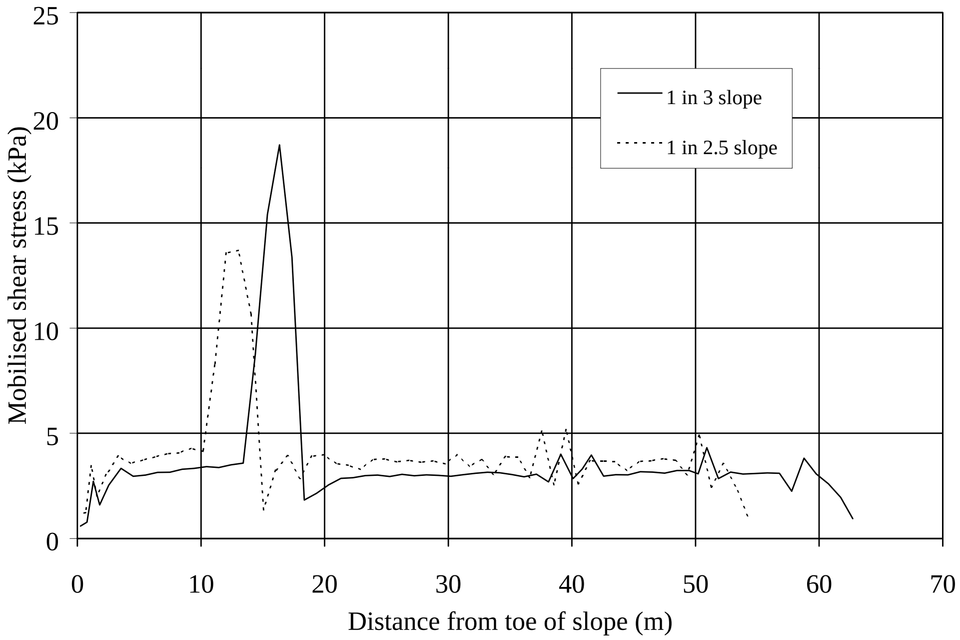

Figure 10. Mobilised shear stresses for 1 in 3 and 1 in 2.5 slopes with a D4 dozer.

The maximum shear stresses are mobilised at different distances along the interface from the base of the slope due to the geometry. Loads were applied 5 m vertically above the toe of the slope; this corresponds to 15.8 m and 13.5 m along the interface for the 1 in 3 and 1 in 2.5 slopes respectively. The steeper slope mobilises a lower shear stress; 13.7 kPa compared to 18.7 kPa for the 1 in 2.5 slope.

Displacements mobilised at the geosynthetic interface for both 1 in 3 and 1 in 2.5 slopes are given in Figure 11. The 1 in 2.5 slope gives a much higher maximum interface displacement; 79.6 mm compared to 23.4 for the 1 in 3 slope. These large displacements for the 1 in 2.5 slope result in sections of interface with significant post-peak shear stresses, and in the vicinity of the loading, the friction angle nears the residual friction values, see Figure 12.

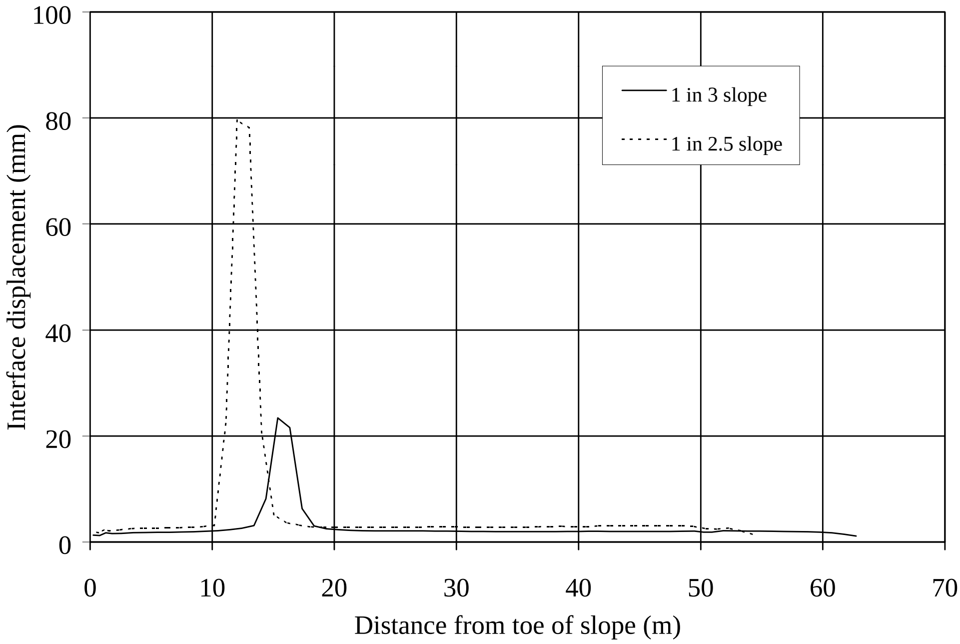

Figure 11. Mobilised interface displacements for 1 in 3 and 1 in 2.5 slopes with a D4 dozer.

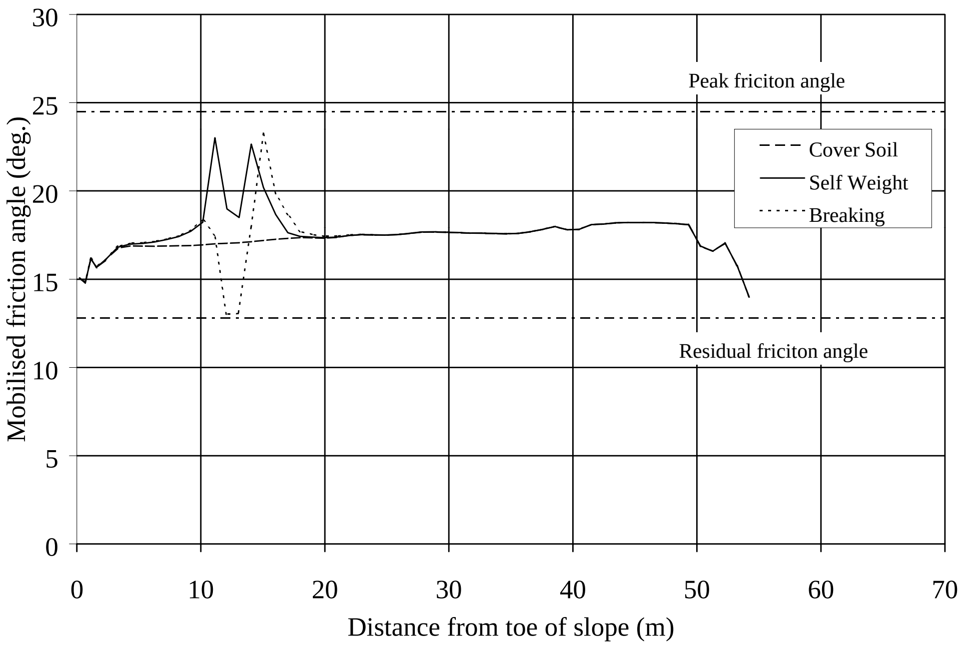

Figure 12. Mobilised friction angle along interface for a 1 in 2.5 slope with a D4 dozer.

4.4 *Comparison of D4 and D6 dozers*

In the analysis described in this paper a small tracked dozer (such as a Caterpillar D4) has been modelled. It may be that in some instances a larger dozer would be used. The effect of increasing the dozer size to a D6, as analysed by Kerkes (1999) has also been investigated.

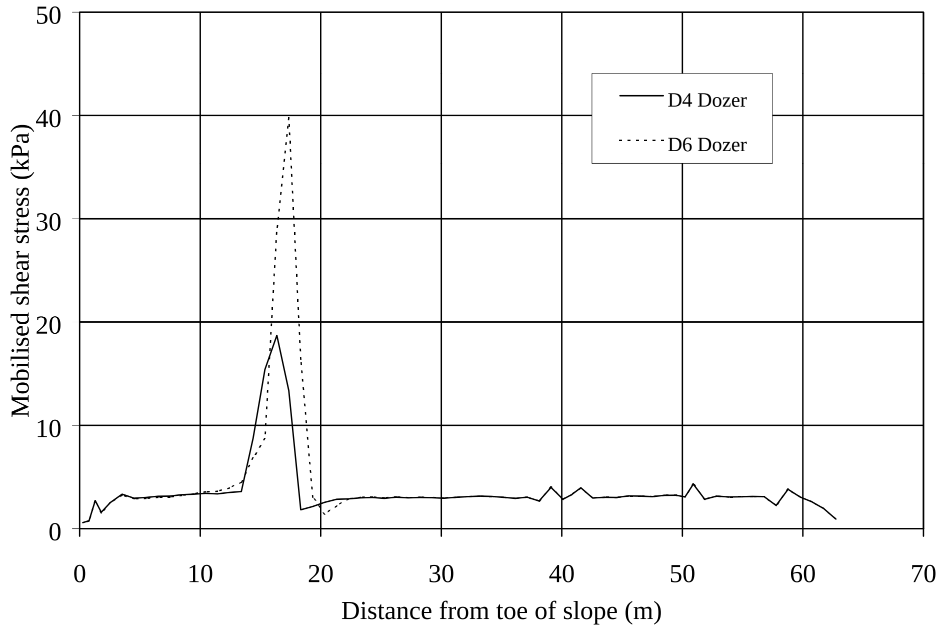

Figure 13. Comparison of mobilised shear stress for a 1 in 3 slope with D4 and D6 dozers.

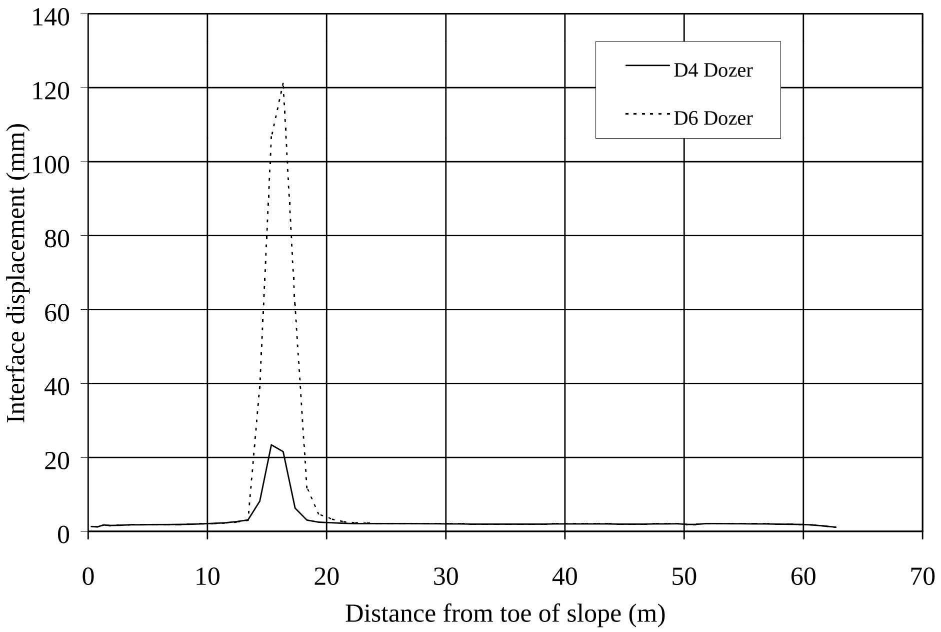

Figure 14. Comparison of mobilised interface displacements for a 1 in 3 slope with D4 and D6 dozers.

The results of the analysis of a D6 dozer on a 1 in 3 slope is compared to the D4 dozer in Figure 13. The footprint of the peak mobilised shear stress is slightly larger for the D6 due to it having a longer length of track in contact with the ground. The maximum value of mobilised shear stress (Figure 13) is significantly greater for the D6 than for the D4 even though the D6 induces much greater interface displacements (Figure 14). This is because the weight is much greater for the D6 than the D4; 16,930 kg compared to 7,785 kg. It is clear from the mobilised friction angle plot (Figure 15) that the larger load of the D6 has induced residual interface shear strengths in the vicinity of the dozer.

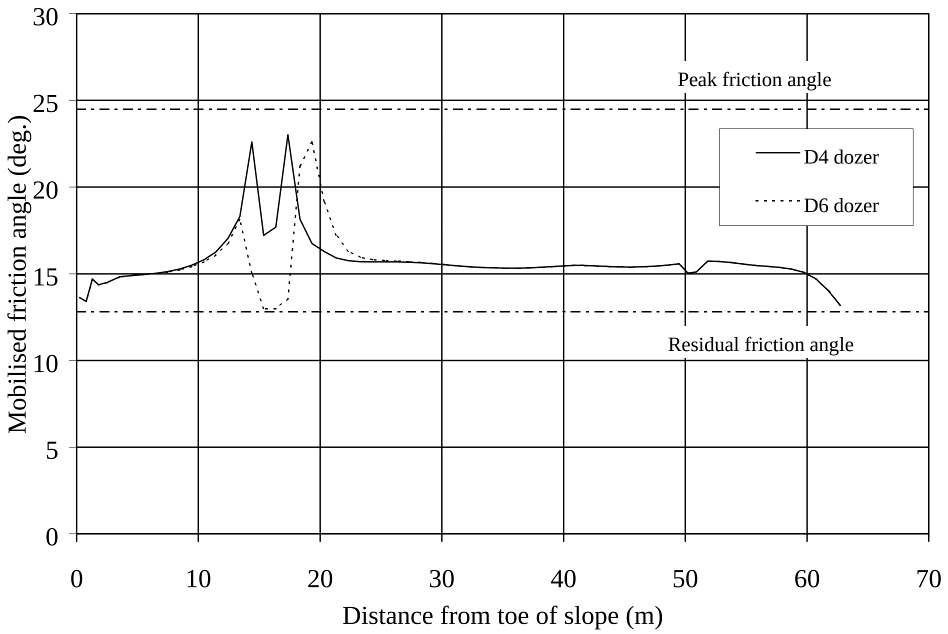

Figure 15. Comparison of mobilised friction angles for a 1 in 3 slope with D4 and D6 dozers.

5 DISCUSSION

The analyses presented in this paper demonstrate that construction plant loading on slopes can result in significant displacements at a geomembrane/geotextile interface with an overlying soil veneer. These displacements can cause reductions in interface shear strength to post peak values, and are achieved for relatively small pieces of plant (i.e. a D4), and for shallow slopes (i.e. 1 in 3). As would be expected, increasing the weight of the plant and angle of slope both result in increased displacements and sections of interface with mobilised residual shear strengths. Figures 8, 12 and 15 show only relatively small lengths of the interface (about 3 m) with post peak shear strengths. However, the analyses are for the construction plant at one location on the slope. As the vehicle travels up and down the slope carrying out normal construction operations, most of the slope area will experience both the self-weight and breaking/accelerating forces, and some areas will experience repeated loading. Therefore, it is likely that a significant area of the interface will undergo shear displacements large enough to form extensive zones with post peak shear strengths.

Under these conditions it is possible for local uncontrolled slippage to occur at the interface. As the deformations will be non-uniform, this could cause local tensile failure of the geotextile protection layer. This would result in removal of the protection to the geomembrane, and hence ultimately to the cover soil damaging the geomembrane. Even if failure does not occur during construction, the interface will be severely weakened and this could lead to failure in the future, either local or global, resulting from other construction or site activities. This work has important implications for designers with respect to the selection of appropriate shear strength parameters for use in design.

A number of simplifications in the method of analysis mean that care should be taken when applying the above results. The use of a two dimensional analysis for what is clearly a three dimensional problem inevitably introduces a significant approximation. The loads from the construction plant have been factored to take account of this, but the three dimensional distribution of the load can not be modelled. In addition, although failure of the geotextile is discussed as a possible result of displacements at the interface, the tensile strength, or stress/strain behaviour, of the geotextile is not included. However, the authors are confident that the general trends of predicted behaviour are valid. Work is ongoing to investigate the implications of using a two dimensional analysis, the cumulative effect of plant movement over the site and the influence of cover soil depth.

6 CONCLUSIONS

This investigation demonstrates the importance of considering the strain softening behaviour of geosynthetic/geosynthetic interfaces when used below cover soils traversed by construction plant. In particular the following conclusions can be drawn:

- Typically used construction plant (e.g. a D4 dozer) can result in post peak shear strengths being mobilised on textured geomembrane/geotextile interfaces located on slopes under relatively thin veneers of granular cover soil.
- Residual shear strengths can be mobilised if heavy plant (e.g. a D6 dozer) is used, or light plant (e.g. a D4 dozer) is used on steep slopes (e.g. 1 in 2.5).
- If the movement of the construction plant over the entire slope is considered, it is likely that significant areas of the interface will have mobilised shear strengths at or close to residual.
- Non-uniform displacements in response to this construction related plant movement could result in local failure of the geotextile protection layer, and hence damage of the geomembrane.
- The results of this preliminary investigation demonstrate the general trends of behaviour. Future work is planned to assess in detail the key factors controlling behaviour.
- This work has important implications for designers when selecting appropriate shear strength parameters for use in design.

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