The Use of Geosynthetic Reinforcement for Enhancing the Stability of the Geomembrane–Soil Interface along the Slopes of Cover Systems of MSW Landfills

Riya Bhowmik[1]; Reshma Sukumar[2]; Manoj Datta[3*]; and J. T. Shahu[4]

[1]Ph.D. Student, Dept. of Civil Engineering, IIT Delhi, Hauz Khas, New Delhi, India. E-mail: riyabhowmik89@gmail.com
[2]Former M.Tech. Student, Dept. of Civil Engineering, IIT Delhi, Hauz Khas, New Delhi, India. E-mail: sreshma.ep@gmail.com
[3*]Professor and Head of the Department, Dept. of Civil Engineering, IIT Delhi, Hauz Khas, New Delhi, India. E-mail: mdatta@civil.iitd.ernet.in
[4]Professor, Dept. of Civil Engineering, IIT Delhi, Hauz Khas, New Delhi, India. E-mail: shahu@civil.iitd.ac.in

Abstract: In this paper, the tensile capacity and anchorage required for making the geomembrane – soil interface stable against sliding have been evaluated for an old MSW waste dump. Four methods of analysis have been used – two infinite slope analysis (ISA) and two finite slope analysis (FSA) methods. Slope inclinations are varied from 3.0H:1.0V to 1.5H:1.0V and heights between berms are taken as 5 m and 10 m. The tensile capacity required to stabilize a slope as steep as 1.5H:1.0V, calculated from all the methods, varies between 33 to 85 kN/m for berm height of 5 m and 90 to 170 kN/m for berm height of 10 m. Estimation of anchorage capacity for run – out and trench anchor has been done by four methods – Koerner (2005), Qian et. al. (2002), Sharma and Lewis (1994) and Villard and Chareyre (2004). For run – out anchor, the anchorage capacity for 2 m wide berm is observed to lie between 30 to 46 kN/m. This increases to the range of 48 to 110 kN/m for trench anchor. These capacities are adequate for anchorage of geosynthetics when height between berms is limited to 5 m. The present study demonstrates the wide variation in results obtained by different methods and the need for improving the current practice.

INTRODUCTION

For recovery of gas from MSW waste dumps and landfills, cover systems having geomembrane as a component of the barrier system are used. Geomembrane (GM) prevents the escape of landfill gases, but inclusion of GM inserts a weak interface in the cover system. GM-soil interfaces have low shearing resistance and thus the stability against slippage along these interfaces offers a problem. In such cases, stability of the cover system can be enhanced by utilizing the tensile strength of the geomembrane as well as introducing high strength geogrids/geotextiles as veneer reinforcement in cover systems. These geomembranes and geogrids/geotextiles are then anchored at the berms on intermediate locations along the slope. Usually the contribution of the geomembrane is a small fraction in comparison to that of the high strength geogrids/geotextiles.

The stability of cover systems of landfills depend partly on the tensile strength of the reinforcement and partly on the efficiency of the anchorage of the reinforcement at the berms. The most common types of anchors are simple run-out anchor, V – shaped anchor or rectangular shaped anchor in a trench along the berm. Since berms have limited widths (usually 2 to 3 m), the reinforcements are often buried in trenches to increase the anchorage capacity.

At an old waste dump in the eastern region of national capital of India, an impermeable cover was provided along the side-slopes and on the top of a small area of the dump as a part of a pilot landfill gas extraction project. At some locations, slopes were as steep as 1.9H:1.0V and cover (with HDPE geomembrane) had to be provided without flattening the slope. A combination of reinforcement and geocells was used (Datta 2014). The possibility of using such covers in slopes as steep as 1.5H:1.0V is the prime factor that has resulted in this study.

OBJECTIVE AND SCOPE

The present study investigates the following aspects of stability of MSW landfill cover systems with gas recovery:

a) Identify the variations obtained by using four methods of stability analysis on
   i. The factors of safety of cover slopes.
   ii. The long term tensile capacities of geosynthetics required for ensuring stability.

b) Examine the differences in four methods of evaluating anchorage capacity of geosynthetic reinforcement embedded on berms and identify the variations in the capacities of run – out anchor and rectangular trench anchors.

c) Examine whether the required tensile force generated in the reinforcement can be withstood by anchorage developed along the berm at top of the slopes for slopes as steep as 1.5H:1.0V.

The study has been carried out for slope inclinations varying from 3.0H:1.0V to 1.5H:1.0V, for heights between berms of 5 m and 10 m and berm width of 2 m. The analyses have been done for the case of dry slope only. For flow parallel to outer slope as well as for earthquake, the acceptable factor of safety is lower than 1.5, and depends on submergence ratio. The effects of these parameters have not been studied, as the objective of this study is to highlight the variability in results.

LITERATURE REVIEW

Stability is the primary issue for cover systems on steep slopes. A combination of low interface values between geosynthetics or between geosynthetics and soil and steep slopes gives rise to failures. The use of high – strength geotextiles and geogrids to provide structural support to cover systems has been widely reported in the literature (Christopher 1991, Carroll and Chouery – Curtis 1991, Koerner 1998). The basic design for structural support consists of identifying the failure plane and evaluating the corresponding factor of safety, and thus selecting the required support to achieve the acceptable factor of safety.

There are basically two different kinds of methods presently available in the literature for the stability analysis of cover slope systems under static conditions, namely the infinite slope analysis (ISA) methods and the finite slope analysis (FSA) (two – wedge) methods.

ISA methods apply for the case where the thickness of the sliding mass is very small compared to slope height. The free body diagram showing the forces and stresses acting on a soil volume on an infinitely long slope is shown in Fig. 1.

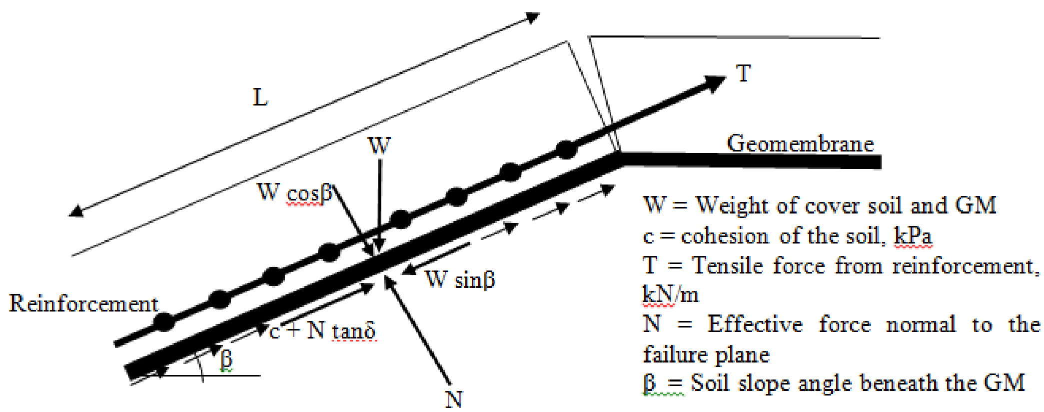


FIG. 1. Limit equilibrium forces associated with ISA method for a slope with veneer reinforcement

When veneer reinforcement with long term (allowable) design strength (LTDS) of T is inserted in soil mass, it opposes the driving force (imparted by overburden material) along with the frictional resistances developed at any particular interface (c + N tanδ). Typically in multi – component cover systems, low shear strength interface is located beneath the veneer reinforcement. The factor of safety (FoS) is calculated as the ratio of summation of resisting forces to summation of driving forces. Now, based on whether T in the veneer reinforcement would be added to the resisting forces or reduced from the driving forces, two different methods can be defined:

ISA- Enhanced Resisting Force (ERF)

In this method, T is added to the resisting forces acting on the interface under consideration, and hence for any interface having an interface friction angle of δ inclined at an angle β with the horizontal, FoS can be defined as in Eq. 1. L is the slope length for a berm height of H.

$$FoS = \frac{\tan\delta}{\tan\beta} + \frac{c}{\gamma h\, \sin\beta} + \frac{T}{L\gamma h\, \sin\beta} \tag{1}$$

ISA- Reduced Driving Force (RDF)

When T is deducted from the net driving forces due to self-weight of the cover soil, FoS can be defined as shown in Eq. 2.

$$FoS = \frac{\frac{c}{\gamma h} + \frac{tan\delta}{tan\beta}}{1 - \frac{T}{L\gamma h \, sin\beta}} \tag{2}$$

In the FSA methods (also known as two-wedge methods), apart from the above forces, the resistance offered by the passive wedge at the toe of the slope is also included. This method assumes that the entire sliding block comprises of two wedges (refer Fig. 2): wedge ABCE or A′BCE, which is the active wedge, and wedge DEC which is the passive wedge that offers the toe buttressing effect. The inherent assumptions in these methods are that the inter – wedge interface EC is vertical and force P transmitted between the two wedges across EC is parallel to slope. Giroud et. al. (1995) adopted the two – wedge method of analysis to deduce the expression for factor of safety. Fig. 2 shows the schematic representation of this approach where A′BCE is the active wedge and DEC is the passive wedge. The forces are balanced parallel to the slope to arrive at Eq. 3. The most important feature of this method is that it quantifies the contribution of each governing parameter separately, which enables in evaluating the stability of a layered system on slopes under progressive deformation, as strength of each component is not mobilized simultaneously. Also, it simplifies design calculations.

$$FoS = \frac{\tan\delta}{\tan\beta} + \frac{c\,cos\emptyset}{\gamma h\,sin\beta\,\cos(\beta+\emptyset)} + \frac{t\,sin\phi}{h\,sin2\beta\,\cos(\beta+\emptyset)} + \frac{a}{\gamma t\,sin\beta} + \frac{T}{\gamma Ht} \tag{3}$$

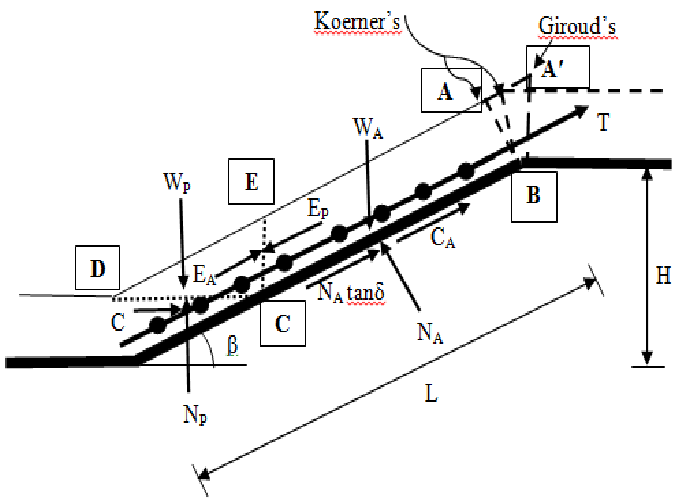
Koerner and Soong (1998) also adopted the two – wedge method for analyzing veneer slope stability problem. Their analysis method was refinement of the approach taken by Koerner and Hwu (1991). Unlike Giroud et. al.'s approach, this approach assumed a tension crack at the crest, as shown in Fig. 2 by A – B, and balanced the forces in the vertical direction. Eq. 4 shows the expressions for calculations according to this method.

$a = (W_A – N_A \cos\beta – T \sin\beta) \cos\beta$

$b = - [(W_A – N_A \cos\beta – T \sin\beta) \sin\beta \tan\phi + (N_A \tan\delta + C_A) \sin\beta \cos\beta + (C + W_P \tan\phi) \sin\beta]$

$c = (N_A \tan\delta + C_A) \sin^2\beta \tan\phi$

$$FoS = -b + \frac{\sqrt{b^2 - 4ac}}{2a} \tag{4}$$



W_A=weight of active wedge
W_P=weight of passive wedge
N_A= normal eff. force of active wedge
N_P=normal eff. force of passive wedge
C_A=adhesive force between cover soil of the active wedge and the geosynthetic
C=cohesive force along the failure plane of the passive wedge
E_A = interwedge force acting on active wedge from passive wedge
E_P= interwedge force acting on passive wedge from active

FIG. 2. Limit equilibrium forces associated with FSA methods for a slope with veneer reinforcement

The tensile force, T generated in the reinforcement to stabilize the cover system on slopes has to be resisted by the anchorage, T_{anch} developed along the slope due to embedment of the reinforcement at the berms where the geosynthetic is anchored (refer Fig. 3).

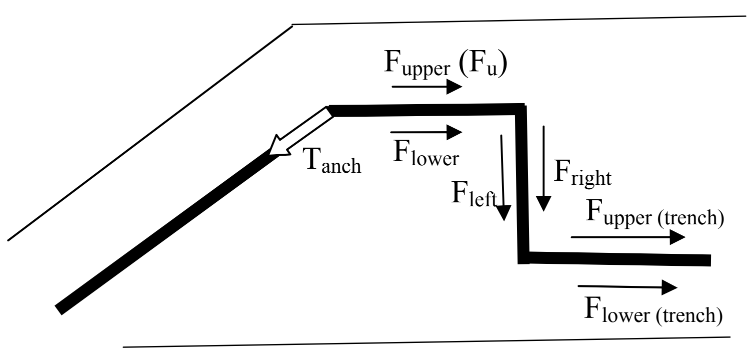


FIG. 3. Development of T_{anch} at berms

Existing design methods for simple run – out anchors and trench anchors (refer Figs. 4 and 5) have been developed for geosynthetic sheets and can be categorized on the basis of assumptions regarding increase in tensile strength at bends. The assumptions vary from (a) frictionless pulley, where tension in the sheet remains unaffected at bends to (b) tension increases at bends due to increase in normal stress caused by inclination of geosynthetic tensile force and (c) frictional pulley, where tension in sheet increases at each bend by a factor.

The assumption of frictionless pulley at every bend was adopted by Sharma and Lewis (1994). This yields the most simplified but conservative method of analysis, where the tension in the sheet remains unaffected at bends and slope inclinations. For simple run – out anchors, Koerner (2005) assumes that in addition to the frictional resistance mobilized at the interface due to deformation/slippage, additional normal stress is generated due to inclination of T_{anch}; the value of which is equal to its vertical component. Qian et. al (2002) developed a method which is a combination of the approaches taken by Sharma and Lewis (1994) and Koerner (2005). This method assumes a frictionless pulley at 90° bends, and it also considers the effect of increment in normal stress due to inclination of T_{anch}. Villard and Chareyre (2004) proposed an analytical method to estimate pullout strength provided by anchors, based on experimental investigations conducted on anchored geotextiles in sandy silt and sand (Chareyre et. al. 2002). This method considered the increment in tension in geosynthetic sheet at the bends by assuming a frictional pulley. The increment is a function of angle at the bend, β and is expressed by a factor $K = \exp(\beta \tan\delta)$. Two failure mechanisms, namely [1] failure at soil-geosynthetic surface and [2] failure in soil mass on which geosynthetic sheet rests were taken into account. The failure mechanism to be considered depends on the type of soil mass on which the geosynthetic is placed and the function of the geosynthetic sheet.

For an anchor trench, while all the described methods assume K_0 condition for normal stress on the vertical segment of sheet buried in trench, Koerner's method assumes active or passive earth pressure, depending on the side being considered.

ANALYSIS

The final cover cross-section for a typical MSW landfill having gas collection system chosen in this study consists of a topsoil cover to support vegetation, a sand drainage layer for draining and a composite barrier ($GM_{textured}$ and compacted clay liner) in top-down order.

The properties of soil used for the analysis are given in Table 1 and the interface parameters are listed in Table 2. The values are taken from the literature (Datta 2014, Koerner and Narejo 2005).

Geomembrane – to – clay ($GM_{textured}$ – clay) interface being the weakest interface in the cover system is considered as the critical interface governing the stability of the entire cover system. The interface friction angle for a textured – geomembrane ($GM_{textured}$) to clay is 18° for the peak and 16° for the residual conditions. Choice of peak or residual values depends on the relative movement anticipated between the geomembrane and clay during installation. In the present study, the peak value is adopted. The adhesion of the geomembrane to clay is neglected.

Table 1. Soil Properties

Soil layer	Cohesion (kPa)	Friction angle(◦)	Unit weight (kN/m^3)
Topsoil	0	30	18
Drainage sand	0	32	20
Compacted Clay	15	0	20

Table 2. Interface Parameters

Interface	Friction angle(◦)		Adhesion (kPa)	
	Peak	Residual	Peak	Residual
GM $_{textured}$-clay	18	16	10	0
Drainage Sand- GM $_{textured}$	30	28	8	0

Ten different inclinations and two different heights of berms provided in a landfill were chosen for the present study. Heights considered were 5 m and 10 m. The inclinations ranged from 3.0H:1.0V to 1.5H:1.0V. They were chosen so as to represent the gentle slopes in new landfills and steep slope conditions encountered in old waste dumps. The desired factor of safety for slope stability is 1.5 and where this value is not achieved, T required to stabilize the cover system has been computed.

Two cases of anchor system have been analysed; (a) run – out anchor which is embedded along 2 m width of berm (refer Fig. 4) and (b) rectangular trench anchor which is embedded after 1 m in a trench of depth 0.5 m and width 1 m (refer Fig. 5). Wider berms have not been considered as they result in average slope become flatter over the full height of the landfill. Also, lack of availability of horizontal space to make wider berms limits resizing of anchor trench.

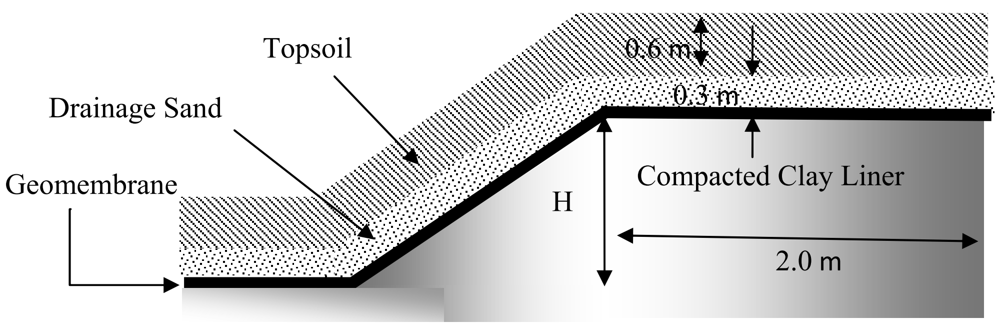


FIG. 4. Landfill cover system with run – out anchor

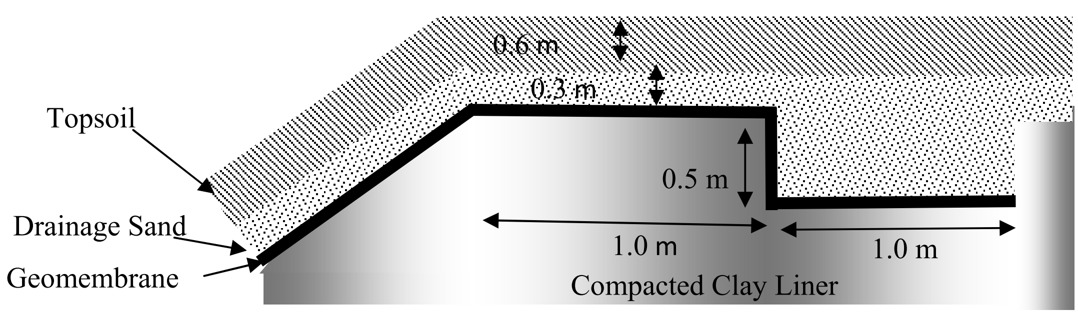


FIG. 5. Landfill cover system with rectangular trench anchor

RESULTS

Factor of Safety

The FoS obtained for different slope inclinations were evaluated by the two ISA methods and two FSA methods. The influence of passive wedge resistance on the stability, as considered by the Koerner and Soong's (K&S) and Giroud et. al.'s (GIR) methods was studied through plots of factor of safety and compared with the results obtained from two methods of ISA.

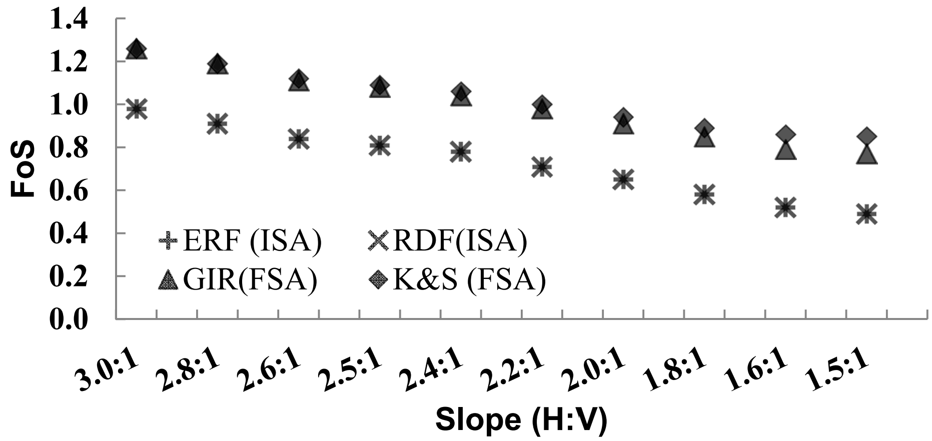


FIG. 6. Comparison of FoS obtained from ISA and FSA methods for varying slope inclinations and 5 m height between berms

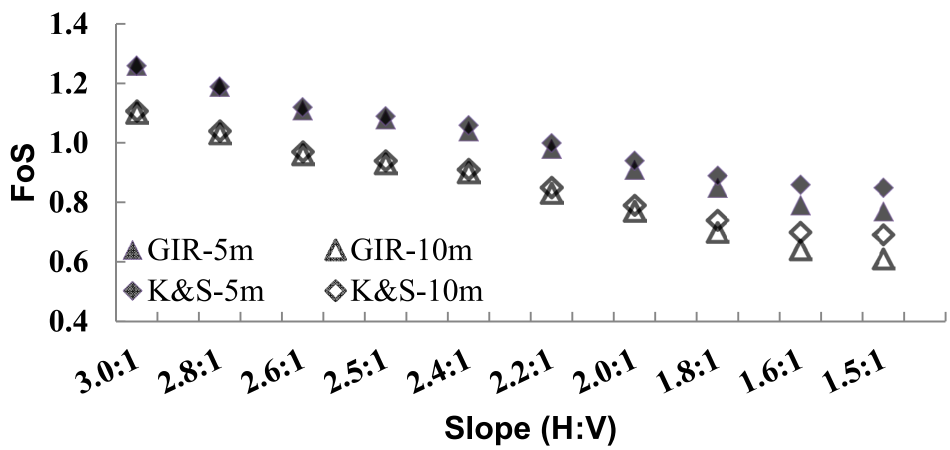


FIG. 7. Comparison of FoS obtained from FSA methods for heights between berms of 5 m and 10m

Fig. 6 compares the factor of safety with varying slope inclinations of the cover system for a typical berm height of 5 m. This figure shows that FSA methods yield FoS values which are approximately 1.3 times the values yielded by ISA methods.

Both the ISA methods yield same results for the same slope geometry. The same can be said for FSA methods for flatter slopes. But as the slopes become steeper, K&S method starts yielding higher FoS values than that of GIR method. At the slope of 1.5H:1.0V, K&S value is 10% higher than that of GIR. Fig. 7 compares the FoS values for heights between berms of 5 m and 10 m by FSA methods. It can be seen that for larger height, FoS values are lesser. This is so because with increasing height between berms, the contribution of passive wedge reduces. The FoS value reduces by 12% when height increases from 5 m to 10 m at the slope of 3.0H:1.0V., while it reduces by 18 to 20% for the slope of 1.5H:1.0V. The factor of safety is not influenced by varying heights between berms when ISA methods are used.

Tensile Capacity

As seen in Figs. 6 and 7, that in none of the cases, the desired FoS of 1.5 is achieved. Thus, the required tensile force (T, kN/m) to stabilize the cover system and achieve FoS of 1.5 is computed and plotted, as shown in Figs. 8 and 9. It can be seen from Fig. 8 that as the slope steepens, requirement of T also increases. It is interesting to find that although FoS values calculated from ISA methods (ERF and RDF) yielded same values (refer Fig. 6), but the T values yielded by them are different (refer Fig. 8). ERF method gives values 50% higher than that of RDF method. Similar observation can be made for FSA methods also. Though for flatter slopes, K&S and GIR methods yielded similar values, but values of T are different. GIR method gives values which are 48 – 50% higher than that yielded by K&S method. Even more important behavior to note is that the range of variations in required T is large (33 – 85 kN/m) amongst all methods for slope as steep as 1.5H:1.0V (height between berms 5 m). Such large tensile capacities can be provided by high strength geogrids/geotextiles available in the market.

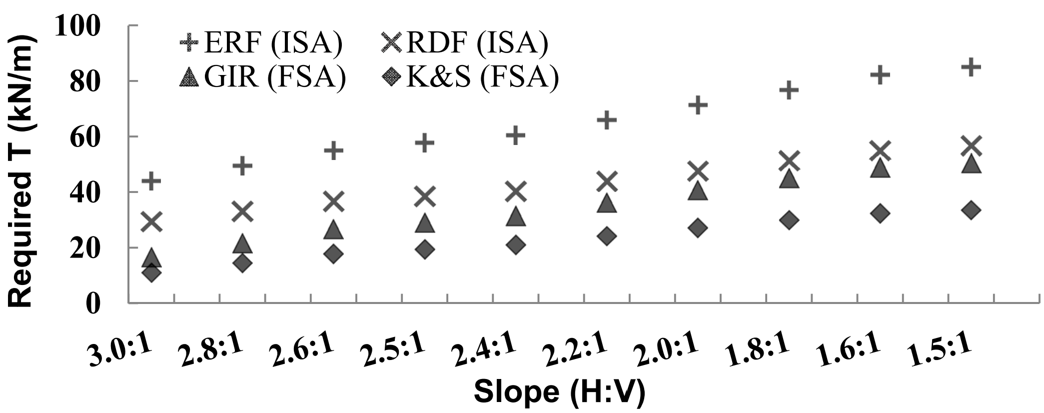


FIG. 8. Comparison of T obtained from ISA and FSA methods for 5 m height between berms

Fig. 9 shows that when the height between berms increases from 5 m to 10 m, T required to stabilize the slope also increases. For slopes with inclination of 3.0H:1.0V and berm heights of 5 m, T required is in the range of 11 to 45 kN/m and this increases to 40 to 88 kN/m for 10 m height, whereas for inclination of 1.5H:1.0V, the range is 33 to 85 kN/m for 5 m height, which increases to the range values of 90 to 170 kN/m for 10 m height. The required T values calculated using GIR method are almost 1.5 times higher than those calculated by K&S method, while ISA methods yield T which are approximately 1.7 times higher than that of GIR.

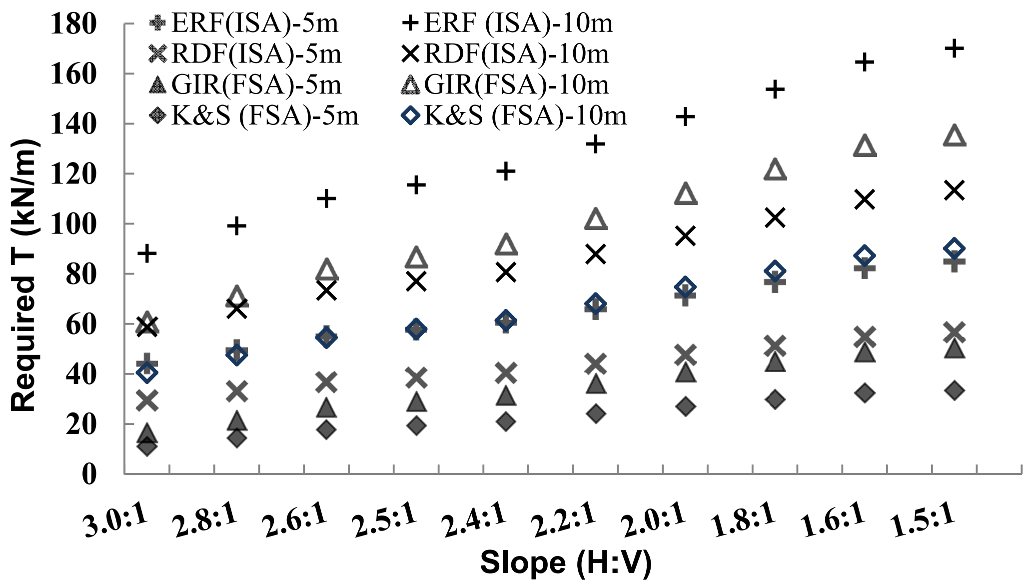


FIG. 9. Comparison of T obtained from different methods for heights between berms of 5m and 10m

Anchorage

The maximum anchorage capacities, T_{anch} for run – out and anchor trench configurations have been computed as per Koerner (KOE), Qian et. al. (QIA), Sharma and Lewis (S&L) and Villard and Chareyre (V&C) methods. The authors have chosen to compute the capacities by, both, neglecting as well as including the resistance offered by cover soil on top of the reinforcement (F_U).

While evaluating T_{anch} using V&C method for both run – out and rectangular trench anchor, the authors chose to use only Mechanism [1] (refer Section 3) with the assumption that probability of soil – to – soil failure occurring in compacted clay is less.

Fig. 10 presents the results for run – out anchor and Fig. 11 for anchor trench in terms of anchorage capacity as a function of inclination of slope. There is a wide variation in results for the same configuration of anchor because of variability in mechanism adopted by different investigators. The value of T_{anch} remains unchanged with slope inclinations in cases where a frictionless pulley is assumed to exist at bends (S&L method). Without a trench, the anchorage capacity for 2 m wide berms is observed to lie between 30 to 46 kN/m, when frictional resistance offered by the top cover soil (F_U) is considered. Without F_U, this is as low as 10 kN/m to 16 kN/m. The capacity enhances to the range of 48 to 110 kN/m when an anchor trench is provided.

These values of T_{anch} are adequate to resist the magnitude of T generated in the reinforcement when height between berms is limited to 5 m.

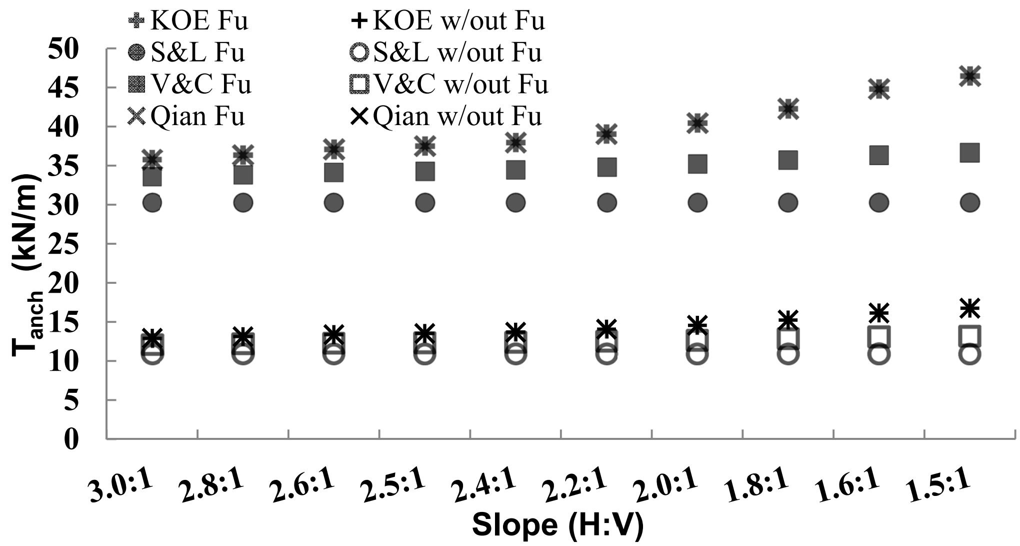


FIG. 10. Comparison of T_{anch} obtained from different methods for runout anchor, with and without resistance from soil on top of reinforcement (F_u)

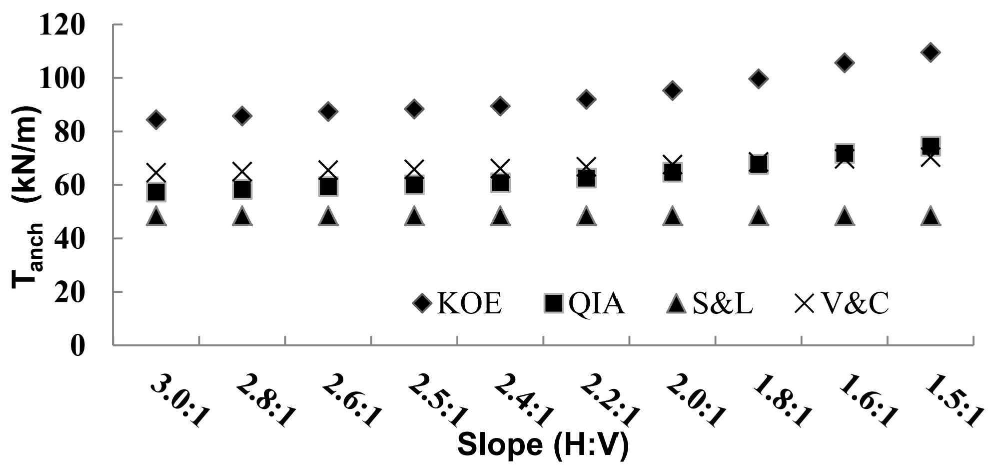


FIG. 11. Comparison of T_{anch} obtained from different methods for trench anchor

CONCLUSIONS

The present study reveals the following about the role of geosynthetic reinforcement in enhancing the stability of geomembrane – soil interface along slopes of cover systems of MSW landfills.

a) Methods using finite slope analysis yield higher factors of safety and lower values of required tensile capacity of reinforcement in comparision to methods using infinite slope analysis. High strength geogrids/geotextiles can provide the tensile capacity required to stabilize soil along interfaces of geomembrane – soil for steep slopes (1.5H:1.0V) with height between berms of 5 m.

b) The anchorage capacities offered by 2 m wide berms are inadequate to resist the tensile force developed in reinforcements along steep slopes (1.5H:1.0V) with height between berms of 5m, if run – out anchor is used. The capacities are enhanced by the use of trench anchor and these meet the requirements for stabilizing slopes of inclinations as steep as 1.5H:1.0V.

c) There is a wide variation in magnitudes of tensile capacities required as well as anchorage capacities offered when different computational methods listed in literature are used. More data from the field practice as well as laboratory studies can help reduce variability in design results.

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