Design of Veneer Cover Soils on Landfills

J.N. Mandal* and S.V. Suresh†

Introduction

Koerner and Soong (1998) have presented the slope stability analysis for different scenarios of cover soil in a landfill. They obtained the factor of safety (FS) as a result of the analyses and then they discussed its acceptability for different scenarios.

The geomembrane liner acts as the primary leachate barrier but is susceptible to mechanical damage during construction and subsequent waste placement and must be protected. In order to protect the geomembrane a soil layer is placed which also acts as a drainage layer for leachate.

The sliding of cover soils on slopes underlain by geosynthetics is obviously an unacceptable situation and will eventually reflect on the entire technology. Steeply sloped leachate collection layers are the situations where such slides can occur. The following are the reasons, which pose a major challenge;

- The barrier materials have a low interface shear strength with respect to the soil placed upon them.
- The liner posses a potential sliding.
- The shear planes are linear.
- Liquid (water or leachate) cannot percolate downward due to the barrier material.

---

* Department of Civil Engineering, Indian Institute of Technology, Powai, Mumbai-400 076, India

† Department of Civil Engineering, Indian Institute of Technology, Powai, Mumbai-400 076, India.

Here the geotechnical engineering consideration is presented leading to the goal of establishing a suitable factor of safety against the cover soil slope instability. A number of common situations are analyzed which have the tendency to decrease stability, and two numbers of design options are analyzed which increases or enhances the stability. For all the analysis the design charts are presented for typical soil characteristics.

Cover Soil on Geomembrane Lined Slopes

Cover soil is placed above the geomembrane in landfills to protect the geomembrane liner from the Municipal Solid Waste (MSW) that will be dumped. Cover soil on the geomembrane along with landfill caps and closures are shown in Fig.1. The stability of the overlying materials as well as tensile stresses of the geomembrane should be performed. Single or double liners are used beneath solid and liquid waste. In this analysis, a single geomembrane has been discussed here for the development of design model.

Interface friction considerations:

$$E_c = \left(\frac{C_a}{C}\right) * 100$$

$$E_\phi = \left(\frac{\tan\delta}{\tan\phi}\right) * 100$$

where E_C = efficiency on cohesion

c = cohesion of soil-to-soil

δ = friction angle of soil-to-geomembrane

C_a = adhesion of soil-to-geomembrane

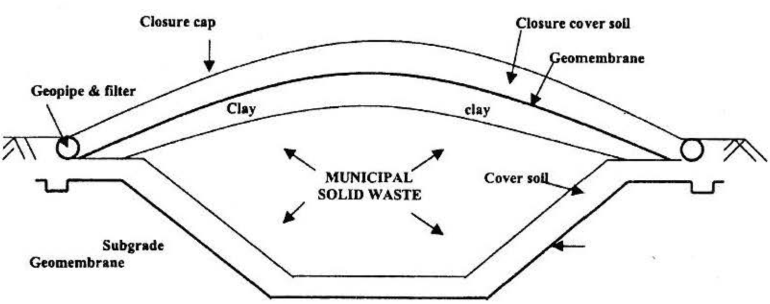

FIGURE 1 : Schematic Diagram of a Landfill showing Major Parts

E_ϕ = efficiency of friction

ϕ = internal friction angle of the soil

Stability of Cover Soil having Uniform Thickness

Geomembranes are to be covered with cover soil for the following reasons:

- Protection against oxidation, ultraviolet degradation, accidental damages intentional damage, ice puncture and tearing by sharp objects.
- Eliminating of wind uplift stresses.
- Minimization of temperature.

The covering is a thin layer of soil, which has the tendency when placed on side slopes to slide gravitationally downwards. The frictional resistance between soil and geomembrane is lower than the sub-grade soil. Fig.2 shows the subsoil, geomembrane and cover soil having uniform thickness and the limit equilibrium forces in an infinite slope. The potential failure surface for cover soil is usually linear with the cover soil sliding with respect to the lower interface friction layer in the underlying cross section. The potential failure plane being planer allows for a straightforward stability calculation without the need for trial center locations and different radii as with soil stability problems analyzed by rotational failure surfaces. Furthermore, full static equilibrium can be verified without any assumptions.

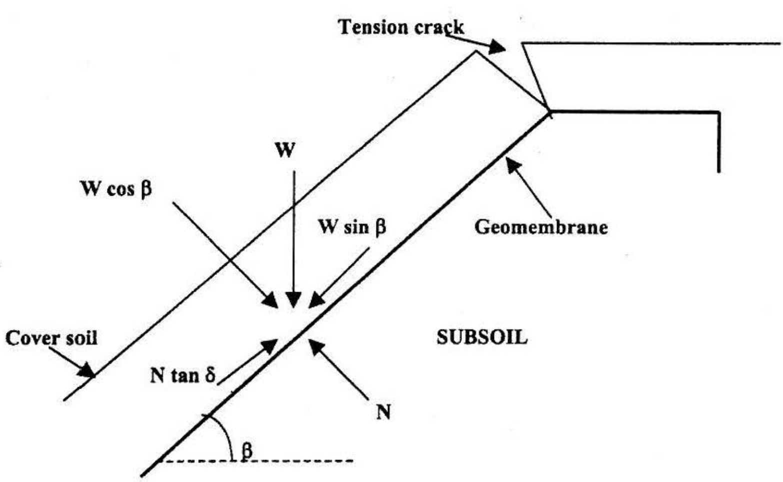

FIGURE 2 : Limit Equilibrium Forces involved in an Infinite Slope Analysis for a Uniformly Thick Cohesionless Cover Soil

Taking force summation parallel to the slope and comparing the resisting force to the driving or mobilizing force treats this situation quite simply. Thus, a factor of safety is determined.

$$
\begin{aligned}
FS &= \frac{\sum \text{Resisting Forces}}{\sum \text{Driving Forces}} \\
&= N \tan\delta / W \sin\beta \\
&= W \cos\beta \tan\delta / W \sin\beta \\
FS &= \tan\delta / \tan\beta
\end{aligned}
$$

where, W = weight of the cover soil

N = reaction normal to the geomembrane = $W\cos\beta$

δ = interface friction angle between geomembrane and cover soil

b = Side slope angle with respect to horizontal.

The interface shear strength of a cover soil with respect to the underlying material is critical to properly analyze the stability of the cover soil. This interface shear strength is obtained by laboratory testing. The values of peak shear strength τ_p for corresponding normal stress σ_n is plotted to obtain interface friction angle δ and cohesion intercept c_a. In case of design

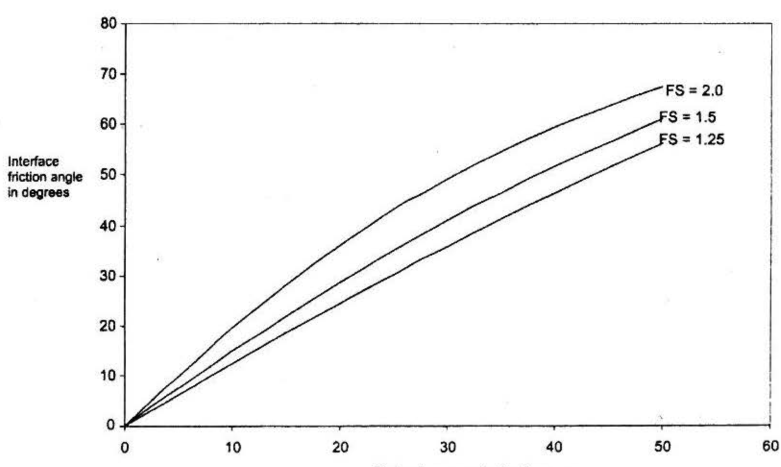

FIGURE 3 : Design Chart, Factor of Safety

we know the slope angle β. A factor of safety is selected (i.e. FS = 1.25, 1.5 or 2). The frictional angle between geomembrane and cover soil is obtained using the above equation or the design chart shown in Fig.3.

Unstable situation may arise due to seepage forces on the cover soil. In addition seismic forces will further reduce the factor of safety. All such situations that cause instability are discussed in detail later. The soil covers on geomembrane-lined slopes are protected against scour by stone riprap, precast concrete blocks, erosion control geosynthetics, or other armoring system. The geomembrane-lined slope with cover soil may fail due to sliding failure of the soil or tension failure of the soil or tension failure of the geomembrane.

Situations causing Destabilization of Slopes

The following are the common situations, which cause destabilizing forces for the slope stability.

- Gravitational forces of the cover soil
- Gravitational forces of the cover soil and construction equipment on the slope
- Gravitational forces of the cover soil and seepage forces
- Gravitational forces of the cover soil and seismic forces

Gravitational Forces of the Cover Soil

A finite length slope with uniformly thick cover soil placed over a liner material at a slope angle 'β' is shown in Fig.4. The analysis that follows is after Koerner and Hwu (I 99 1).

The cover soil of unit weight γ and thickness h is placed on the geomembrane, which is at an angle of β with the horizontal. The shear strength parameters, ϕ and c and adhesion C_a and interface friction angle δ should be determined in the laboratory. The discrete zones of active and passive wedges are shown in Fig.4. A tension crack may appear at the top of the slope. For the slope stability analysis the factor of safety is obtained as follows:

$$W_A = \gamma h^2\left(\frac{L}{h} - \frac{1}{\sin\beta} - \tan\frac{\beta}{2}\right)$$

$$N_A = W_A \cos\beta$$

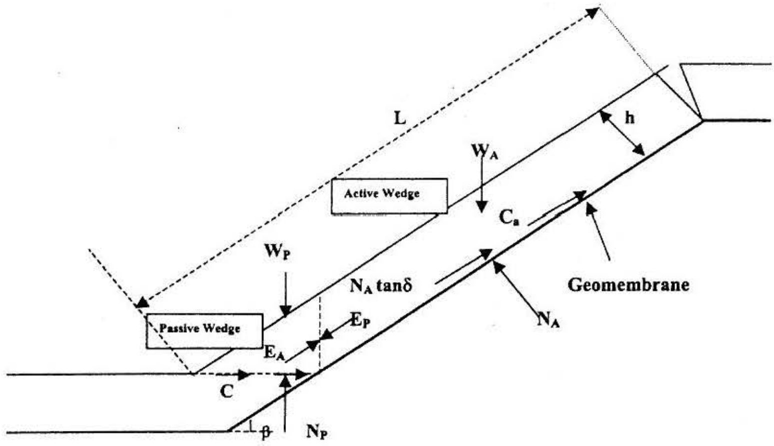

FIGURE 4 : Limit Equilibrium Forces involved in a Finite Length Slope Analysis for a Uniformly Thick Cover Soil

$$C_a = c_a \left[\frac{(L-h)}{\sin\beta} \right]$$

$$W_p = \frac{\gamma h^2}{\sin 2\beta}$$

$$N_p = W_p + E_p \sin\beta$$

$$C = \frac{c\,h}{\sin\beta}$$

where W_A = total weight of the active wedge

W_p = total weight of the passive wedge

γ = Unit weight of the cover soil

N_A = effective force normal to the failure plane of the active wedge

β = Soil slope angle beneath the geomembrane

ϕ = Friction angle of the cover soil

δ = interface friction angle between cover soil and geomembrane

h = uniform thickness of the cover soil

L = length of slope measured along the geomembrane

C_a = adhesive force between cover soil of the active wedge and geomembrane

c_a = adhesion between cover soil of the active wedge and geomembrane

C = cohesive force along the failure plane of the passive wedge

c = cohesion of the cover soil

E_A = inter-wedge force acting on the active wedge from the passive wedge and

E_p = inter-wedge force acting on the passive wedge from the active wedge

$$E_A = \frac{(FS)(W_A - N_A\cos\beta) - (N_A\tan\delta + C_a)\sin\beta}{\sin\beta(FS)}$$

$$E_p = \frac{C + W_p\tan\phi}{\cos\beta(FS) - \sin\beta\tan\phi}$$

On equating E_A and E_p a quadratic equation is formed from which FS is obtained as under

$$FS = \frac{\left(-b + \sqrt{(b^2 - 4ac)}\right)}{2a} \tag{1}$$

where $a = (W_A - N_A\cos\beta)\cos\beta$

$$b = -\left[\begin{array}{l}(W_A - N_A\cos\beta)\sin\beta\tan\phi \\ \qquad + (N_A\tan\delta + C_a)\sin\beta\cos\beta \\ \qquad + \sin\beta\left(C + W_p\tan\phi\right)\end{array}\right]$$

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

A FS value greater then 1.0 must be targeted. In the above analysis situations like seepage forces, seismic forces and construction equipment have not been considered.

A program was developed to obtain FS-values for typical soil

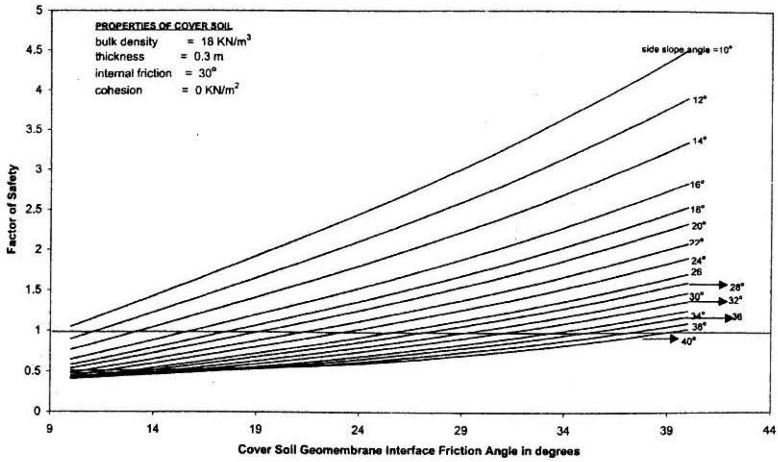

FIGURE 5 : Design Chart, Factor of Safety for Cover Soil Slope Stability (Gravitational Forces only)

characteristics for various slope angles β and interface friction angle δ. The results are given as design chart as per Fig.5.

Gravitational Forces of Cover Soil and Construction Equipment on Slope

Earth moving equipment-weighing W_b is used to place the cover soil on the liner material. It is desirable to place the cover soil from toe upward to the crest. This is because the gravitational forces of the cover soil and live load of the construction equipment are compacting previously placed soil and working with an ever-present passive wedge and stable lower portion beneath the active wedge. The reduction of FS value due to equipment on slope moving upward is relatively small compared to the equipment moving downward. The earth moving equipment and the equipment forces are shown in Figs.(6a) to (6d).

In the first case when the equipment moves up slope: the term 'W_e' is added to 'W_A' and the analysis is carried like that of gravitational forces. Fig.6c is used for the analysis as given in Poulos and Davis (1974).

In case of equipment moving down: slope is inevitable then a dynamic force per unit width at the cover soil to geomembrane interface, 'F_e', is taken into account as shown in Fig.(6d).

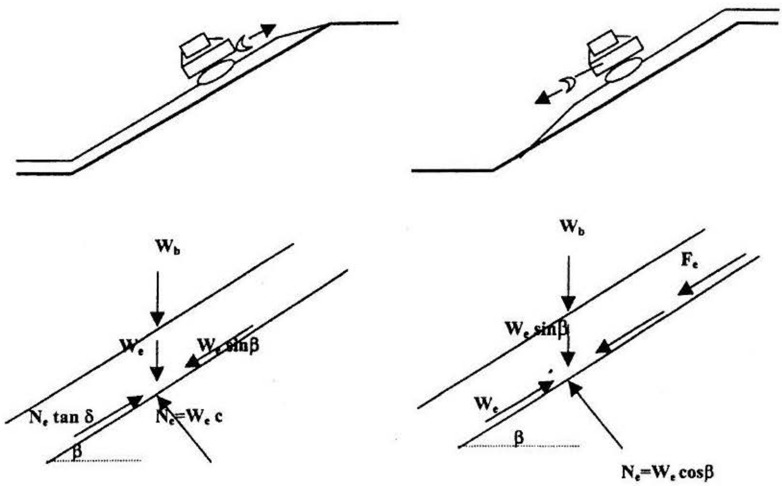

FIGURE 6 : (a) Equipment Back-filling Up Slope; (b) Equipment Back-filling Down Slope; Equipment Moving Up Slope; (d) Equipment Moving Down Slope

$$E_A = \frac{(FS)\left[(W_A + W_e)\sin\beta + F_e\right]}{FS} - \frac{\left[(N_e + N_A)\tan\delta + C_a\right]}{FS}$$

$$E_p = \frac{\left(C + W_p \tan\varphi\right)}{\left[\cos\beta(FS) - \sin\beta\tan\phi\right]}$$

where W_b = weight of the equipment

W_e = equivalent equipment force per unit width at the Geomembrane interface = qwI

q = $W_b(2wb)$, where b = width of track and w = length of track

I = influence factor at the geomembrane interface

N_e = effective equipment force normal to the failure plane of the active wedge

F_e = dynamic force per unit width parallel to the slope at the geomembrane interface

= $W_e(a/g)$, where a = acceleration of the equipment and g = acceleration due to gravity

Poulos and Davis (1974) gave values of 'I' and 'a'. Here the forces E_A and E_p are equated to find out the factor of safety. On equating,

$$a = \left[\left(W_A - W_e\right)\sin\beta + F_e\right]\cos\beta$$

$$b = -\left\{\begin{matrix}\left[\left(N_A + N_A\right)\tan\delta\right] \\ \qquad + \left[\left(W_A + W_e\right)\sin\beta + F_e\right]\sin\beta\tan\varphi + \left(C + W_p\tan\varphi\right)\end{matrix}\right\}$$

$$c = \left[\left(N_e + N_A\right)\tan\delta + C_a\right]\sin\beta\tan\phi$$

The factor of safety is calculated using Eqn. (1).

In this case of construction equipment on slope a program was developed to obtain FS-values for typical soil characteristics for various slope angles b and interface friction angle d. The results are given as design chart as per Fig.7.

Gravitational Forces of Cover Soil and Seepage Forces

If adequate drainage facility of the cover soil then seepage-induced slope stability problems will occur. This seepage builds up in two different ways a horizontal buildup from the toe upward or a parallel-to-slope buildup outward along the slope. Here analysis is done again using the free-body diagram of active and passive wedges including the pore water pressures. Fig.8 shows the both the seepage conditions.

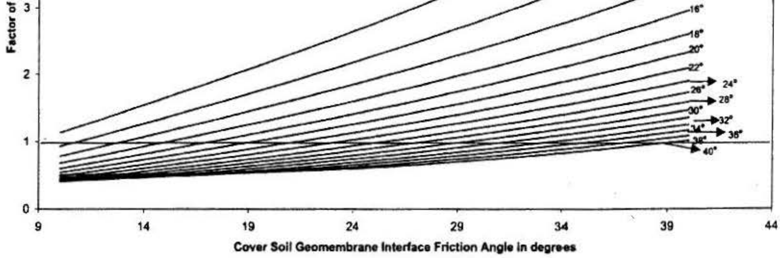

FIGURE 7 : Design Chart, Factor of Safety for Cover Soil Slope Stability with Earth Moving Equipment on Slope

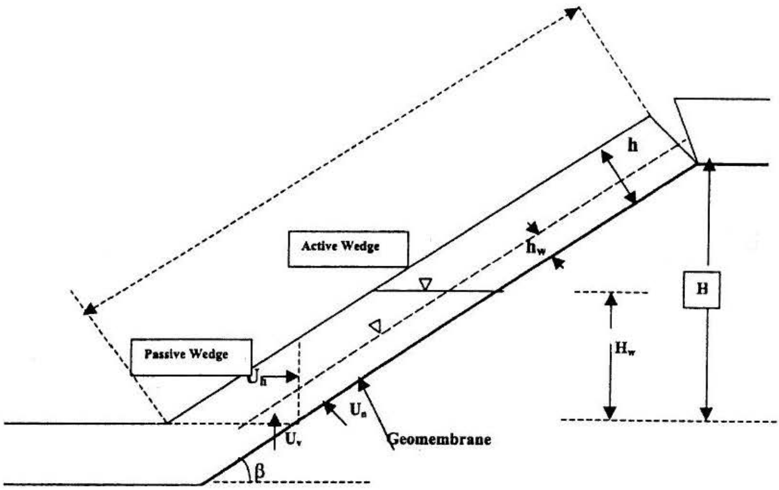

FIGURE 8 : Different Submergence Conditions for Seepage Forces Analysis

The situations which results in seepage induced slides are:

- Drainage soil with very low hydraulic conductivity
- Inadequate drainage capacity at the toe
- Clogging of drainage layer due to accumulation of fines at the toe of the slope
- Freezing of the drainage layer at the toe of the slope

Additional discussion on seepage induced slope failure is given by Soong and Koerner (1996).

Horizontal seepage buildup

Horizontal seepage buildup can occur when toe blockage occurs due to inadequate outlet capacity, contamination or physical blocking of outlets, or freezing conditions at the outlets. The expressions due seepage condition are

$$W_A = \frac{\gamma_{sat} h(2H_w \cos\beta - h)}{\sin(2\beta)} + \frac{\gamma h(H - H_w)}{\sin\beta}$$

$$U_n = \frac{\gamma_w h\cos\beta(2H_w \cos\beta - h)}{\sin(2\beta)}$$

$$N_A = W_A \cos\beta + U_h \sin\beta - U_n$$

$$U_h = \gamma_w h^2/2$$

$$U_v = U_h \cot\beta$$

where U_h = resultant of the pore pressures acting on the inter-wedge surfaces

U_n = resultant of the pore pressures acting perpendicular to the slope

U_v = resultant of the vertical pore pressures acting on the passive wedge

H = height of the landfill

H_w = vertical height of the slope measured from the toe

h_w = height of free water surface measured in the direction perpendicular to the slope

γ_{sat} = saturated unit weight of cover soil

The values of a, b and c to be used for calculation of FS are given in Parallel-to-slope seepage build up and the same is also applicable for horizontal seepage buildup.

Parallel-to-slope seepage build-up

Parallel seepage build-up can occur when soils placed above a geomembrane are initially too low in their hydraulic conductivity, or become too low due to long-term clogging from overlying soils, which do not have a filter. The expressions are,

$$W_A = \frac{\gamma(h-h_w)\left[2H\cos\beta-(h+h_w)\right]}{\sin(2\beta)} + \frac{\gamma_{sat} h_w (2H\cos\beta - h_w)}{\sin(2\beta)}$$

$$U_n = \frac{\gamma_w h_w \cos\beta(2H\cos\beta - h_w)}{\sin(2\beta)}$$

$$U_h = \frac{\gamma_w (h_w)^2}{2}$$

$$W_p = \frac{\left[\gamma\left(h^2 - h_w^2\right) + \gamma_{sat}(h_w)^2\right]}{\sin(2\beta)}$$

After obtaining the expressions the forces E_A and E_p are also obtained and equated to get the values for a, b and c as;

$$a = W_A \sin\beta\cos\beta - U_h \cos^2\beta + U_h$$

$$b = -W_A \sin^2\beta\tan\phi - U_h \sin\beta\cos\beta\tan\phi - N_A \cos\beta\tan\delta - \left(W_p - U_v\right)\tan\phi$$

$$c = N_A \sin\beta\tan\delta\tan\phi$$

As with previous solutions, the resulting FS value is obtained using Eqn (1).

In this case of seepage forces a program was developed to obtain FS-values for typical soil characteristics for various slope angles β and interface friction angle δ. The results are given as design charts as per Figs.9 and 10.

Gravitational Forces of Cover Soil and Seismic Forces

In areas with anticipated earthquake activity, the slope stability analysis of a cover soil must consider seismic forces. The analysis is a two-part process.

- The calculation of a FS value using a pseudo-static analysis by adding a horizontal force (seismic) acting at the centroid of the cover soil cross section.

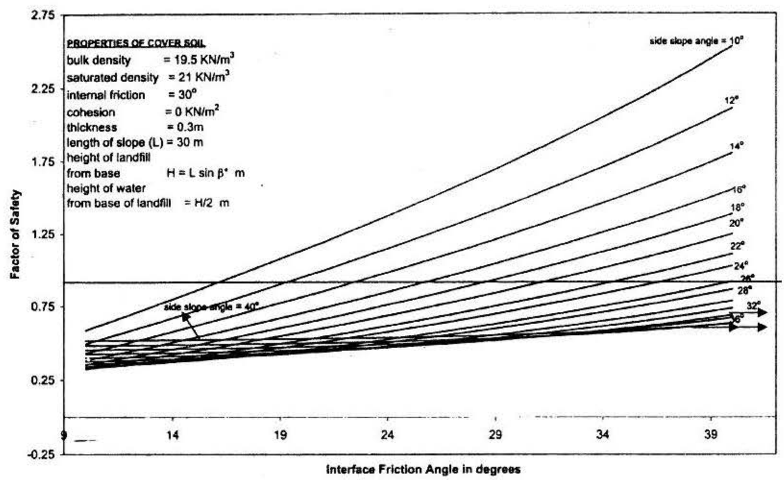

FIGURE 9 : Design Chart, Factor of Safety for Cover Soil Slope Stability with Horizontal Seepage Forces

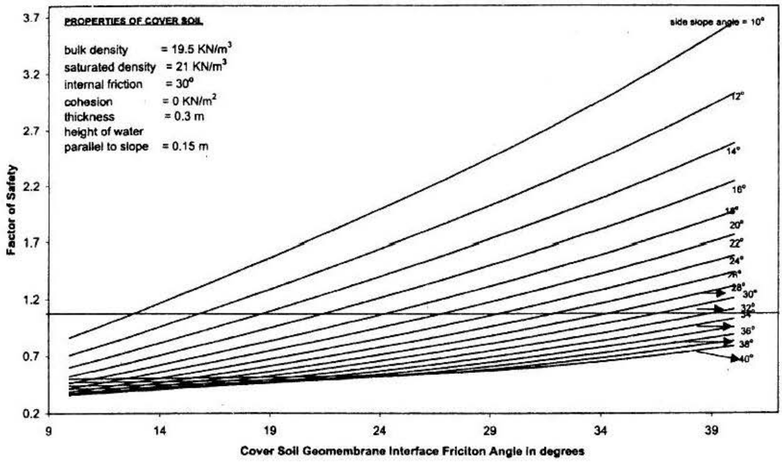

FIGURE 10 : Design Chart, Factor of Safety for Cover Soil Slope Stability with Seepage Forces Parallel to the Slope

- A permanent deformation analysis if the FS value calculated as above is less than 1.0.

In the first part: only a horizontal force at the centroid of the cover sol in proportion to the anticipated seismic activity is added. An average seismic coefficient (C_s) is obtained. The value C_s is a non-dimensional ratio of the bedrock acceleration to gravitational acceleration. The bedrock acceleration can be estimated from a seismic zone map as given by Algermissen (1991). The free body diagram for analysis of seismic effect is shown in Fig.11.

Only the value of C_s is multiplied with 'W_A' and 'W_p' and added to 'W_A' and 'W_p' respectively and the rest of the calculations are same as that of gravitational forces of the cover soil.

where

C_s = average seismic coefficient

$$E_A = \frac{(FS)(C_s W_A + N_A \sin\beta)}{(FS)\cos\beta} - \frac{(N_A \tan\delta + C_a)}{(FS)\cos\beta}$$

$$E_p = \frac{C + W_p \tan\phi - C_s W_p (FS)}{[(FS)\cos\beta - \sin\beta \tan\phi]}$$

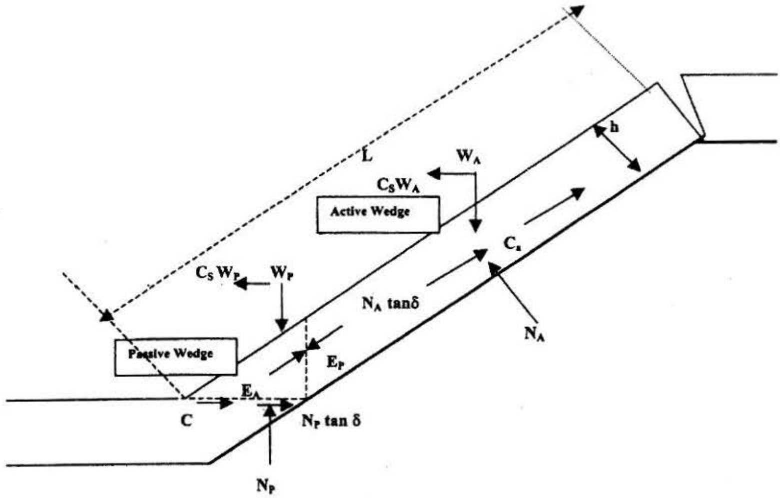

FIGURE 11 : Free Body Diagram for Analysis including Average Seismic Coefficient

$E_A = E_p$ gives a quadratic equation, and coefficients a, b and c are:

$$a = \left(C_s W_A + N_A \sin\beta\right)\cos\beta + C_s W_p \cos\beta$$

$$b = \left(C_s W_A + N_A \sin\beta\right)\sin\beta\tan\phi$$
$$+\left(N_A \tan\delta + C_a\right)\cos^2\beta + \left(C + W_p \tan\phi\right)\cos\beta$$

$$c = \left(N_A \tan\delta + C_s\right)\cos\beta\sin\beta\tan\phi$$

The resulting FS is calculated from Eqn (1).

In the second part: the analysis is directed towards calculating the estimated deformation of the lowest shear strength interface in the cross section under consideration. The deformation is then assessed in light of the partial damage that may be imposed on the system. The permanent deformation can be obtained using empirical charts given by Makdisi and Seed (1978).

In this case of seismic forces a program was developed to obtain FS-values for typical soil characteristics for various slope angles β and interface friction angle δ. The results are given in Fig.12.

Program and Results

A program was written to evaluate the factor of safety for side slope

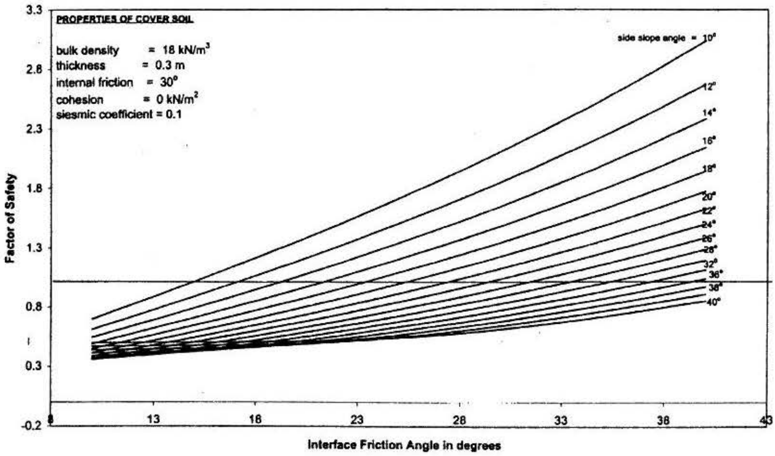

FIGURE 12 : Design Chart, Factor of Safety for Cover Soil Slope Stability with Seismic Forces

of landfill for all conditions analyzed as above and keeping all the values as variables. By taking a typical example of soil parameter values the results are obtained as design charts for varying slope angles (β) and interface friction angle (δ). The design charts are given along with each analysis. From the charts it can understood that as the inter face friction angle increases the FS increases and as the slope angle reduces the FS increases.

Situations Causing Enhanced Stabilization of Slopes

The following are the situations, which can be adopted to enhance the stabilization of the slopes:

- Slopes with tapered thickness cover soil
- Reinforcement of the cover soil with geogrid or high strength geotextile

Slopes with Tapered Thickness Cover Soil

The factor of safety can be considerably increased for a given slope by uniformly tapering the cover soil thickness from thick toe to thin crest (see Fig.13). The slope inclination 'β' is greater than the finished slope angle of the cover soil 'ω'. The earth pressure forces on the respective wedges oriented at the average of the slope and the cover soil angles.

h_c = thickness of cover soil at crest of the slope

w = finished slope angle of cover soil

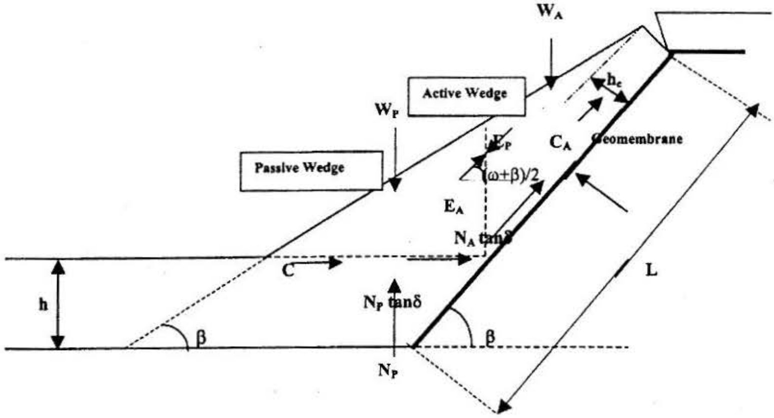

FIGURE 13 : Free Body Diagram for Analysis of Slope Stability with Tapered Cover SOil Thickness

$$y = \left(L - \frac{h}{\sin\beta} - h_c \tan\beta\right)(\sin\beta - \cos\beta\tan\omega)$$

$$N_A = W_A \cos\beta$$

$$W_A = \gamma\left[\left(L - \frac{h}{\sin\beta} - h_c \tan\beta\right)\left(\frac{y\cos\beta}{2} + h_c\right) + \frac{h_c^2 \tan\beta}{2}\right]$$

$$C_a = c_a\left(L - \frac{h}{\sin\beta}\right)$$

$$W_p = \frac{\gamma}{2\tan\omega\left[\left(L - \frac{h}{\sin\beta} - h_c \tan\beta\right)(\sin\beta - \cos\beta\tan\omega) + \frac{h_c}{\cos\beta}\right]^2}$$

$$N_p = W_p + E_p \sin\left(\frac{\omega+\beta}{2}\right)$$

$$C = \frac{\gamma}{\tan\omega\left[\left(L - \frac{h}{\sin\beta} - h_c \tan\beta\right)(\sin\beta - \cos\beta\tan\omega) + \frac{h_c}{\cos\beta}\right]}$$

After E_A and E_p are equated to get the values for a, b and c as;

$$a = \left(W_A - N_A \cos\beta\right)\cos\left(\frac{\omega+\beta}{2}\right)$$

$$b = -\left[\begin{array}{l}\left\{\left(W_A - N_A \cos\beta\right)\sin\left(\frac{\omega+\beta}{2}\right)\tan\phi\right\} \\ \qquad + \left\{\left(N_A \tan\delta + C_a\right)\sin\beta\cos\left(\frac{\omega+\beta}{2}\right)\right\} \\ \qquad + \left\{\sin\left(\frac{\omega+\beta}{2}\right)\left(C + W_p \tan\phi\right)\right\}\end{array}\right]$$

$$c = \left(N_A \tan\delta + C_a\right)\sin\beta\sin\left(\frac{\omega+\beta}{2}\right)\tan\phi$$

As with previous solution, the resulting FS value is obtained using Eqn.(1).

A program was developed to obtain FS-values for typical soil characteristics for various slope angles β and finished slope angle ω. The results are given as design chart as per Fig.14.

Reinforcement of the Cover soil

Another way of increasing a given slope's factor of safety is to

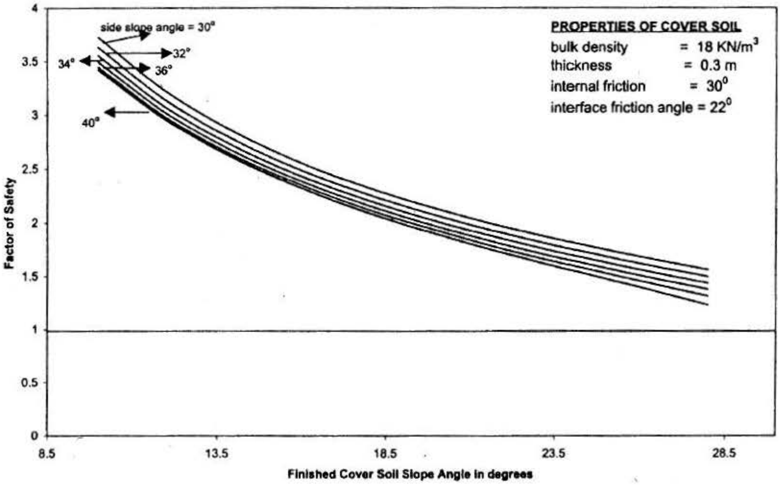

FIGURE 14 : Design Chart, Factor of Safety for Cover Soil Slope Stability with Tapered Thickness

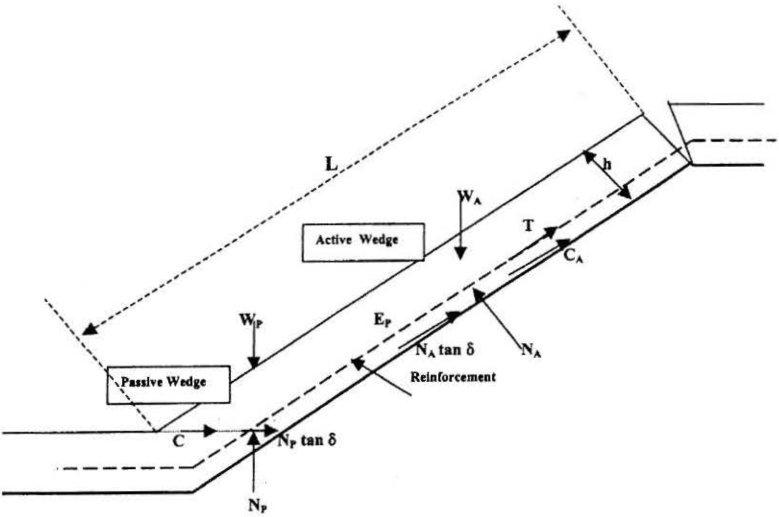

FIGURE 15 : Free Body Diagram for Slope Analysis with Reinforcement of Cover Soil

reinforce it with a geosynthetic material. Fig.15 shows the reinforcement force 'T'.

Here the term 'T' is the allowable strength of the geosynthetics reinforcement inclusion. The allowable 'T' is obtained by applying reduction factors as shown below.

$$T\ (\text{allowable}) = \frac{T\ (\text{ultimate})}{(RF1 * RF2 * RF3 *)}$$

RF1 = reduction factor for installation damages

RF2 = reduction factor for creep

RF3 = reduction factor for long term chemical /biological degradation

This force 'T' will enhance the resisting forces.

As usual the forces E_A and E_p are obtained and equated to get the values for a, b and c as;

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

$$b = -\left[\begin{array}{l}(W_A - N_A \cos\beta - T\sin\beta)\sin\beta\tan\phi \\ \qquad + (N_A \tan\delta + C_a)\sin\beta\cos\beta + \sin\beta\left(C + W_p \tan\phi\right)\end{array}\right]$$

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

Again, the resulting FS value can be obtained using Eqn.(1).

Program and Results

A program is made to estimate the factor of safety for the above mentioned two conditions. As an example, typical soil characteristics were chosen to run the program and the results are shown in Figs.14 and 16. It can be observed that in the charts the reinforcement provided does not substantially increase the FS value whereas the tapered thickness, even with 2 degrees of difference from top and bottom has improved the FS value substantially.

Design of Thickness and Anchorage of Geomembrane Liner

Thickness of the Geomembrane

Koerner (1998) proposed the thickness considerations for the geomembrane for landfills and liquid containment. The basic model along

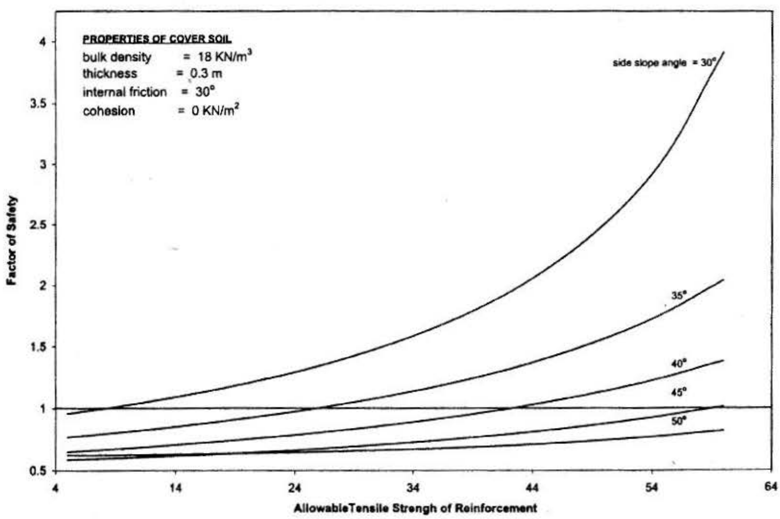

FIGURE 16 : Design Chart, Factor of Safety for Cover SOil Slope Stability with Geosynthetic Reinforcement

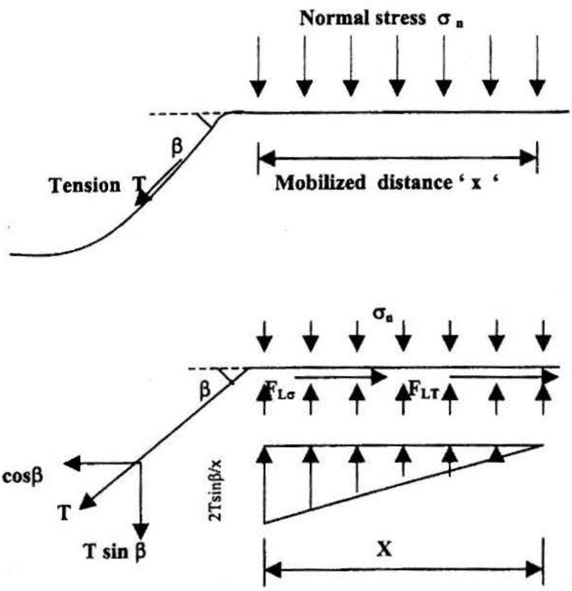

FIGURE 17 : Design Model and Related Forces used to Calculate Geomembrane Thickness

with the forces adopted by him was based on deformation-mobilised tensile force as shown in the Fig.17. The basic relationship is given by

$$T = \sigma_{allow} t$$

where T = tension mobilised in the geomembrane

σ_{allow} = allowable geomembrane stress, and

t = thickness of the geomembrane

After resolving the forces in the horizontal direction and taking $\sum F_x = 0$ we get

$$T\cos\beta = \sigma_n \tan\delta_u(x) + \sigma_n \tan\delta_L(x) + 0.5\left(2T\frac{\sin\beta}{x}\right)(x)\tan\delta_L$$

$$T = \frac{\sigma_n x(\tan\delta_U + \tan\delta_L)}{\cos\beta - \sin\beta\tan\delta_L}$$

$$\text{Therefore thickness 't'} = \frac{\sigma_n x(\tan\delta_U + \tan\delta_L)}{\sigma_{allow}(\cos\beta - \sin\beta\tan\delta_L)}$$

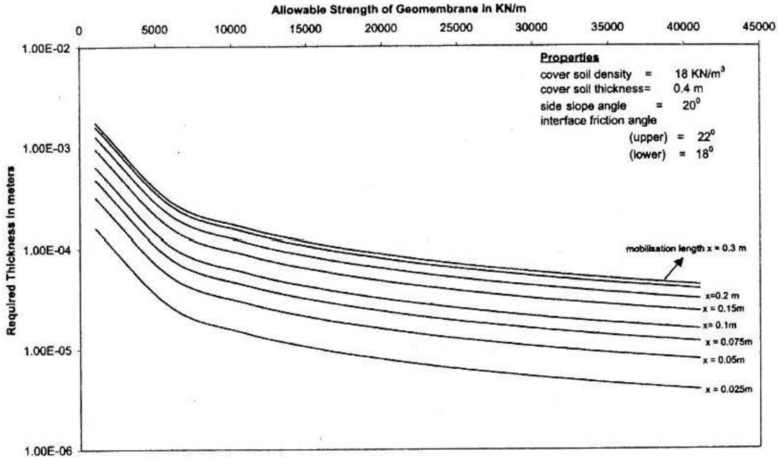

FIGURE 18 : Required Geomembrane Thickness for Various Slope Angles and Interface Angles

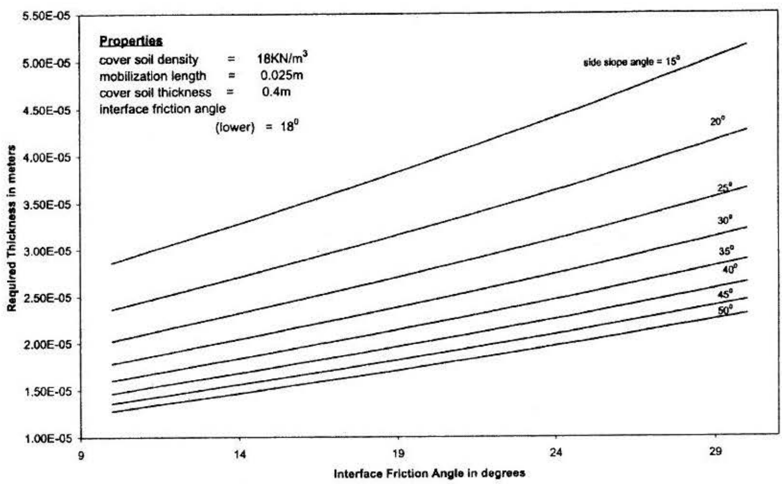

FIGURE 19 : Required Geomembrane Thickness for Various Allowable Strength and Mobilization Distances

where

β = Settlement angle mobilising the geomembrane tension (side slope angle),

σ_n = applied stress from the over lying cover soil,

x = distance of mobilised geomembrane deformation,

δ_L = interface friction angle between geomembrane and the subgrade, and

δ_U = interface friction angle between geomembrane and the cover soil.

Based on the derived equation a computer program was written and results were obtained for the required thickness of the geomembrane for the landfill for various side slope angles and interface friction angle. Also a similar program was run to obtain the required thickness of the geomembrane for various allowable strength of the geomembrane and mobilised distance 'x'. The results obtained for typical cover soil characteristics are shown in Figs.18 and 19.

Runout and Anchor Trench Design

For sufficient anchorage and holding of the geomembrane from slipping, the geomembrane should be extended to a distance (called the runout) away from the end of the slope or it should be taken vertically down into an anchor trench. Koerner (1998) analysed both cases i.e. runout and anchor trench considerations and obtained minimum runout length and depth required. The analysis as given below is based on the Fig.20.

From Fig.20 the horizontal force summation results, leads to the appropriate design equation.

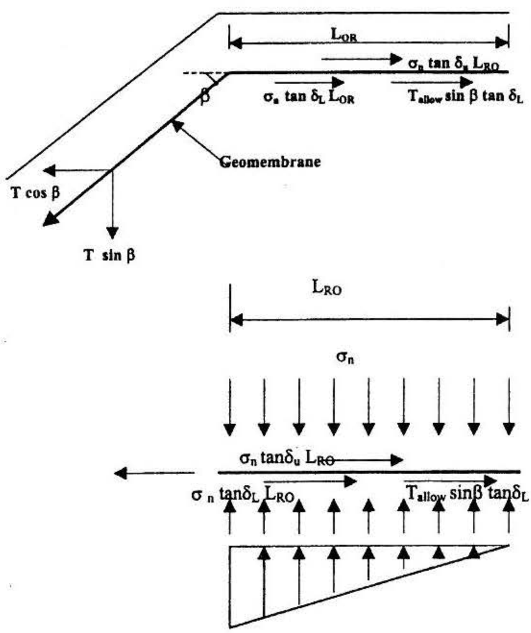

FIGURE 20 : Cross Section of Geomembrane Runout and Related Forces and Stresses

$$\sum F_x = 0$$

$$T_{allow} \cos\beta = \sigma_n \tan\delta_U (L_{RO}) + \sigma_n \tan\delta_L (L_{RO}) + 0.5\left(2\, T_{allow} \frac{\sin\beta}{L_{RO}}\right)(L_{RO}) \tan\delta_L$$

$$L_{RO} = \frac{T_{allow}(\cos\beta - \sin\beta \tan\delta_L)}{\sigma_n\, x(\tan\delta_U + \tan\delta_L)}$$

where L_{RO} = minimum required length of the geomembrane runout

T_{allow} = allowable force in the geomembrane i.e. $\sigma_{allow} t$

σ_{allow} = allowable stress in the geomembrane, and

t = thickness of the geomembrane

β = Settlement angle mobilising the geomembrane tension (side slope angle)

σ_n = applied stress from the over lying cover soil

x = distance of mobilised geomembrane deformation

δ_L = interface friction angle between geomembrane and the subgrade

δ_U = interface friction angle between geomembrane and the cover soil

h = cover soil thickness

The situation for anchor trench depth is shown in Fig.21. An active earth pressure (P_A) is tending to destabilize the situation, whereas a passive earth pressure (P_p) is tending to resist pullout.

Using the free body diagram forces $\sum F_x = 0$

$$T_{allow} \cos\beta = \sigma_n \tan\delta_U (L_{RO}) + \sigma_n \tan\delta_L (L_{RO}) + T_{allow} \sin\beta \tan\delta_L - P_A + P_p$$

$$P_A = 0.5(\gamma_{AT}\, d_{AT} + \sigma_n) K_{AT}\, d_{AT}$$

$$P_p = 0.5(\gamma_{AT}\, d_{AT} + \sigma_n) K_p\, d_{AT}$$

Thus an equation with two unknown is obtained. So either the runout is fixed and required depth is obtained or vice-versa.

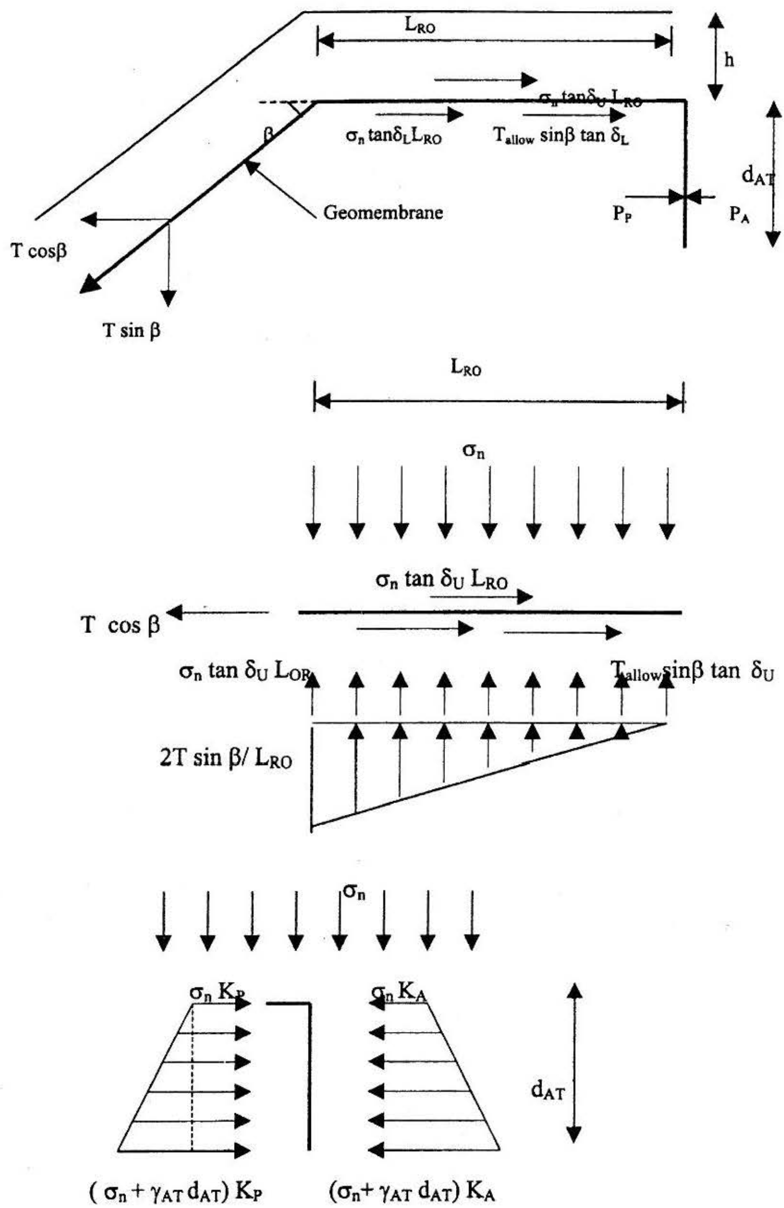

FIGURE 21 : Cross Section of Geomembrane Runout and Anchot Trench with Stress and Forces

where LRO = minimum required length of the geomembrane runout

T_{allow} = allowable force in the geomembrane i.e. $\sigma_{allow} t$

σ_{allow} = allowable stress in the geomembrane, and

t = thickness of the geomembrane

β = settlement angle mobilising the geomembrane tension (side slope angle)

σ_n = applied stress from the over lying cover soil

x = distance of mobilised geomembrane deformation

δ_L = interface friction angle between geomembrane and the subgrade

δ_U = interface friction angle between geomembrane and the cover soil.

γ_{AT} = unit weight of soil in anchor trench

d_{AT} = depth of the anchor trench

K_A = coefficient of active earth pressure = $\tan^2(45-\phi/2)$

K_p = coefficient of passive earth pressure = $\tan^2(45+\phi/2)$

P_A = active earth pressure against the backfill side of the anchor trench

P_p = passive earth pressure against the backfill side of the anchor trench.

Based on the derived equation a computer program was written and results were obtained for the required runout of the geomembrane for the landfill for various side slope angles and interface friction angle. Also a similar program was run to obtain the required runout of the geomembrane for various allowable strength of the geomembrane and anchor trench depth. The results obtained for typical cover soil characteristics are shown in Figs.22 and 23.

Summary

Here the slope stability of the cover soil on top of the geomembrane barrier liner was studied for various conditions and enhancements of the slope stability were also suggested. This analysis also can be applied to drainage soils placed on lined slopes beneath the waste. All the analysis are based on the concept of limit equilibrium with different assumptions involving particular details like

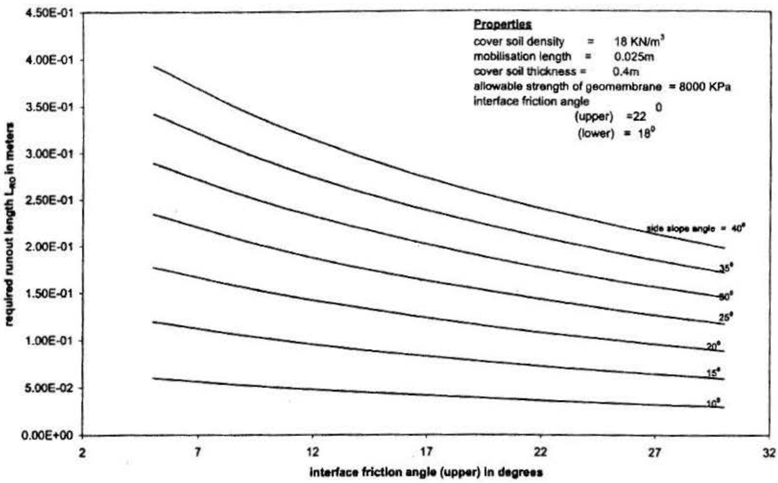

FIGURE 22 : Required Runout Length of Geomembrane for Landfill

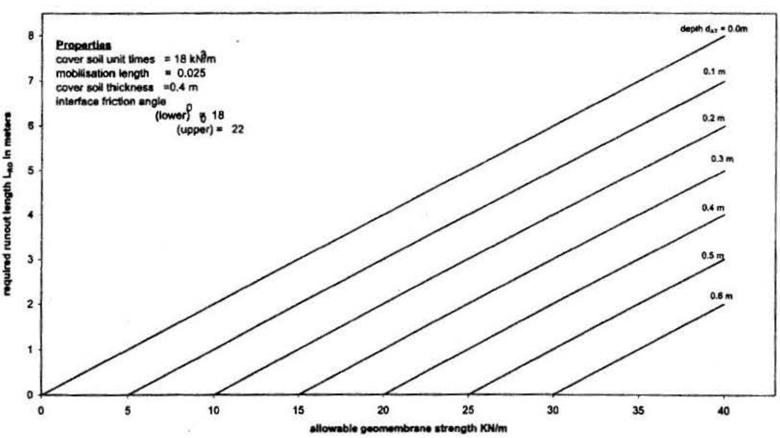

FIGURE 23 : Required Runout Lenth for Various Repths

- Existence of a tension crack at the top of slope (filled or unfilled)
- Orientation of the failure planes (linear) beneath the passive wedge
- Specific details of construction equipment movement on the slopes
- Specific details on seepage forces within the cover soil layer
- Specific details on seismic forces, particularly the magnitude and the selection of interface strengths

- Selection of strengths (allowable and ultimate) of the geomembrane for thickness design and anchor design
- Interface friction angle of geomembrane with subgrade and cover soil for thickness and anchor designs
- Soil characteristics for running all the programs

The design charts are made based on typical soil characteristics of the cover soil however the program presented can be used to obtain the FS value for any site condition and soil characteristics.

References

Algermissen, S.T. (1991) : "Seismic Risk Studies in the United States", *Proceedings 4th World Conference on Earthquake Engineering*, Santiago, Chile, Vol.1, pp.A1-14 to 27.

Druschel, S.J. and Underwood, E.R. (1993) : "Design of Lining and Cover System Side Slopes", *Proceedings Geosynthetics Conference*, Vol.3, pp.1341-1355.

Giroud, J.P. and Beech, J.F. (1989) : "Stability of Soil Layers on Geosynthetic Lining Systems", *Geosynthetic International*, Vol.2, No.6, pp.1115-1148.

Giroud, J.P., Bachus, R.C. and Bonaparte, R. (1995) : "Influence of Water Flow on the Stability of Geosynthetic-Soil Layered Systems on Slopes", *Geosynthetic International*, Vol.2, No.6, pp.1149-1180.

Koerner, R.M. (1998) : *Designing with Geosynthetics*, 4th Edition, Prentice Hall Inc. Publication, New Jersey.

Koerner, R.M. and Hwu, B.L. (1991) : "Stability and Tension Considerations Regarding Cover Soils on Geomembrane Lined Slopes", *Journal of Geotextiles and Geomembranes*, Vol.1, No.4, pp.335-355.

Makdisi, F.I. and Seed, H.B. (1978) : "Simplified Procedure for Estimating Dam and Embankment Earthquake Induced Deformation", *Journal of Geotechnical Engineering*, ASCE, Vol.104, No.GT7, pp.849-867.

Poulos, H.G. and Davis, E.H. (1974) : *Elastic Solutions for the Soil and Rock Mechanics*, John Wiley & Sons Inc., New York, USA, 411 pages.

Soong, T.Y. and Koerner, R.M. (1996) : "Seepage Induced Slope Instability", *Journal of Geotextiles and Geomembranes*, Vol.14, No.78, pp.425-445.