Pseudo-Static Seismic Stability Analysis of Geosynthetic-Reinforced Soil Retaining Walls

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Introduction

Seismic stability of retaining walls using pseudo static limiting equilibrium method for earth pressure calculations is one of the interesting problems in geotechnical engineering. Most of the literature on this subject is on walls without any reinforcement (Sano, 1916; Mononobe, 1929; Okabe, 1926; Madhav and Kameswara Rao, 1968; Steedman and Zheng, 1990; Ling et al., 1999; Ling, 2001; Saran and Gupta, 2003). In recent years some studies on the seismic stability of GRS (Geosynthetics-Reinforced Soil) retaining walls has also been reported (Saran et al, 1992; Ismeik and Guler, 1998; Ling and Leshchinsky, 1998; Ausilio et al., 2000).

Saran et al. (1992) considered rigid wall retaining a reinforced cohesionless backfill with a uniform surcharge load and carried out limit equilibrium analysis assuming a planar failure surface. Non-dimensional design charts were presented for computing the resultant active earth pressure. They further compared the theoretical findings with results obtained from two different sets of model tests on a rigid wall with dry backfill reinforced with aluminum and bamboo strips and observed good agreement between the predicted and experimental values.

Using limit equilibrium method and assuming two-wedge planar failure mechanism Ismeik and Guler (1998) carried out seismic stability analysis on geosynthetics-reinforced soil (GRS) retaining walls subjected to different

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seismic loading conditions. They investigated the effect of full height concrete facing on the amount of geosynthetic reinforcement and presented the results as design charts. It was observed that the facing thickness plays an important role on the wall stability and economy in designs.

Ling and Leshchinsky (1998) studied the effect of vertical and horizontal seismic acceleration using limit equilibrium method with curved failure surface on the stability and permanent displacements of GRS retaining structures without facing wall. Thus, it is seen that only a few studies that consider the effect of vertical seismic acceleration are available so far. These studies highlighted the importance of these aspects and determined the tensile force in the geosynthetics reinforcement the required length of reinforcement considering different possible modes of failures e.g. tie back failure, oblique compound failure, direct sliding and pull out. It is observed that vertical seismic acceleration coefficient acting in the downward direction has a greater effect in stabilizing a steep slope in comparison to a flatter slope especially when the horizontal seismic acceleration coefficient exceeds 0.2. It has also been observed that when the vertical seismic acceleration is in the downward direction there is an increase in the required tensile reinforcement.

Using upper-bound limit analysis and assuming different failure modes like rotation, translation and direct sliding and linear slip surface Ausilio and Dente (2000) conducted pseudo static seismic stability analysis of geosynthetics reinforced slopes subjected to only horizontal seismic acceleration. It is observed that both reinforcement force and the required reinforcement length increase significantly during earthquakes.

Bathurst and Alfaro (1997) made an extensive review of the studies made on the analysis and performance of geosynthetics reinforced walls, slopes and embankments.

Thus there is a scope to extend the above works to study the seismic stability of GRS walls with concrete facing considering planar and composite failure surface for general c-ϕ soils. In this paper such an attempt is made in this direction extending the studies made by Ismeik and Guler (1998) to analyze the seismic stability, considering both vertical and horizontal earthquake acceleration, of a rigid retaining wall using pseudo-static limit equilibrium method. Computation of the geosynthetic reinforcement force coefficient and the corresponding reinforcement length required to stabilize the rigid retaining wall is also undertaken as a part of the study.

Analysis

Assumptions

The following assumptions are made in the analysis:

1. General cohesive-frictional soil backfill

2. Plane failure surface

3. Earthquake imposes a horizontal as well as a vertical acceleration of sinusoidal variation with amplitudes that are equal to some given percentages of gravity (seismic coefficients K_h and K_v) and the same are well below the critical acceleration.

In the present analysis K_v is assumed to be positive when acts in the downward direction.

4. There is no tension crack with in the slope.

Design Approach

A limit equilibrium based method for pseudo static seismic stability computations of a GRS retaining wall has been developed as follows. The method also enables to estimate contribution of the facing to the wall stability, forces in the reinforcements and the corresponding reinforcement length. The wall with a facing of thickness, t is assumed to have rectangular shape with a sufficient amount of bending rigidity (Fig.1). Figure 2 shows the free body diagram of the GRS retaining wall.

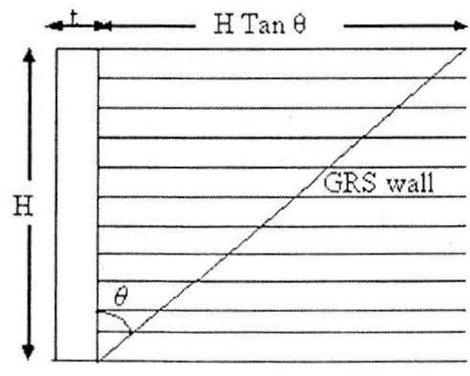

FIGURE 1 : Cross-section of a Geosynthetic Reinforced Soil Retaining Wall with Concrete Wall Facing

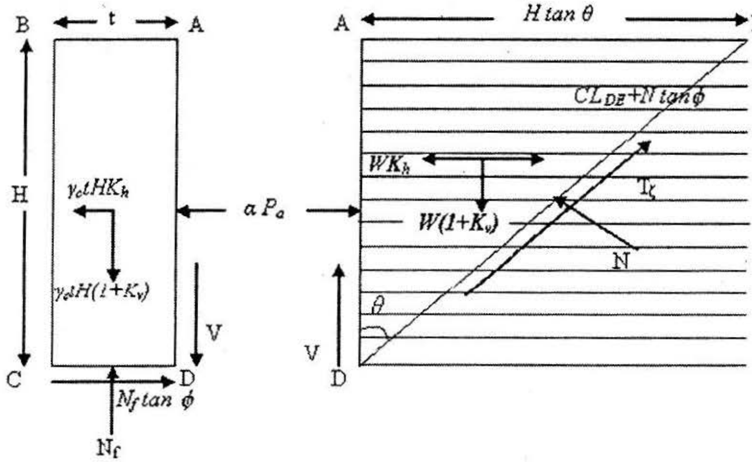

FIGURE 2 : Forces Acting on the Facing System and Wedge based on Pseudo-static Limit Equilibrium Analysis

Wall Forces and Wall Stability

The empirical equation as suggested by Guttenberg and Richter (1956) to calculate maximum seismic acceleration coefficient is adopted in the analysis. Following them the earthquake accelerations in horizontal and vertical directions are:

$$a_{0h} = K_h g$$

$$a_{0v} = K_v g$$

The horizontal earthquake acceleration as a function of earthquake magnitude M is,

$$\log_{10} a_{0h} = -2.1 + 0.81M - 0.027M^2$$

Horizontal forces acting on the facing are:

$$P = \alpha P_a \tag{1}$$

$$H_f = K_h W_f \tag{2}$$

$$S = N_f \tan\phi_f \tag{3}$$

where P = force contributed by the facing to the wall stability expressed as a fraction, α of the active force P_a;

H_f = horizontal seismic force acting on the facing;
S = shear force at the base of the facing; and
ϕ_f = friction angle between the wall facing and backfill.

The vertical forces acting on the facing are defined as:

$$V = P\tan\phi_w \tag{4}$$

$$W_f = t\,h\gamma_c \tag{5}$$

$$N_f = V + W_f \tag{6}$$

where V = shear force between the facing and the backfill;
W_f = self weight of facing;
N_f = normal reaction force at the base of the facing; and
φ_w = Friction angle between the wall facing and the backfill.

Substituting the values of P, V, and W_f in the above equations (Eqn.6 and Eqn.3) gives the following equation for N_f and S:

$$\begin{aligned} N_f &= \left(P\tan\varphi_w + t\,h\gamma_c\right) = \left(\alpha P_a\tan\varphi_w + t\,h\gamma c_c\right) \\ S &= \left(\alpha P\tan\varphi_w + t\,h\gamma_c\right)\tan\varphi_f \end{aligned} \tag{7}$$

Summing all of the forces in the horizontal direction that act on the facing gives

$$\begin{aligned} S &= H_f + P \\ \left(\alpha P_a\tan\varphi_w + t\,h\gamma_c\right)\tan\varphi_f &= K_h t\,H\,\gamma_c + \alpha P_a \end{aligned} \tag{8}$$

Using Mononobe (1929) and Okabe (1926) active earth pressure theory, P_a and K_a are defined as:

$$P_a = \gamma H^2\left[\frac{1}{2}N_{a\gamma} - \left(\frac{C}{\gamma H}\right)N_{ac}\right] \tag{9}$$

Let K_a (combined active earth pressure coefficient) as:

$$K_a = \left[\frac{1}{2}N_{a\gamma} - \left(\frac{C}{\gamma H}\right)N_{ac}\right]$$

where $N_{a\gamma}$ and N_{ac} are Mononobe and Okabe Active earth pressure coefficients.

Substituting Eqns.7 and 8, and solving for α we get

$$\alpha = \left[\left(\frac{1}{K_a}\right)\left(\frac{\gamma_c}{\gamma}\right)\left\{\frac{(1+K_v)\tan\varphi_f - K_h}{1-\tan\varphi_f \tan\varphi_w}\right\}\right]\frac{t}{H} \tag{10}$$

where
γ_c = unit weight of concrete (assumed as 24 kN/m^3);
γ = Unit weight of backfill material (assumed as 18 kN/m^3);
φ_w = Wall friction angle (assumed as 2/3 of back fill angle of friction φ); and
= Foundation friction angle (assumed as 2/3 of back fill angle of friction φ)

α is directly proportional to the variable t/H i.e. as the wall thickness increases α also increases, i.e. its capacity to carry active earth pressure increases.

Considering the force equilibrium of the wedge ADE (Fig.2) and summing up the forces in the horizontal and vertical directions and equating them to zero i.e. $\sum V = 0$ and $\sum H = 0$ we get the following equations,

$$P\tan\varphi_w + N\sin\theta + (cH\sec\theta + N\tan\varphi)\cos\theta = W(1+K_v) \tag{11}$$

$$WK_h + N\cos\theta = (cH\sec\theta + N\tan\theta)\sin\theta + T_r \tag{12}$$

Solving equations 11 and 12 for T_r we get,

$$\begin{aligned} T_r = \frac{1}{2}\gamma H^2 \tan\theta\left[K_h + (1+K_v)\cot(\theta+\phi)\right] \\ -\alpha P_a\left[\cot(\theta+\phi)\tan\phi_w + 1\right] - cH\left[\cot(\theta+\phi) + \tan\theta\right] \end{aligned} \tag{13}$$

Now let required geosynthetic reinforcement force coefficient be

$$\kappa = \frac{T_r}{\frac{1}{2}\gamma H^2}$$

it can be written as:

$$\mathrm{K} = \tan\theta\left[K_h + (1+K_v)\cot(\theta+\phi)\right] \\ -\alpha K_a\left[\cot(\theta+\phi)\tan\phi_w + 1\right] - 2\left(\frac{c}{\gamma H}\right)\left[\cot(\theta+\phi) + \tan\theta\right] \quad (14)$$

Now we need to find the value of θ which would give the minimum value of the reinforcing force T_r.

Minimizing T_r with respect to θ and setting it equal to zero, we get

$$\frac{dT_r}{d\theta} = 0$$

θ_{cr} is found out from the following equation by using Newton-Raphson Technique.

$$\left[K_h + (1+K_v)\cot(\theta+\phi)\right]\sin^2(\theta+\phi)\sec^2\theta + \alpha K_a\tan\phi_w \\ = (1+K_v)\tan\theta + 2\left(\frac{c}{\gamma H}\right)\left[\sin^2(\theta+\phi)\sec^2\theta - 1\right] \quad (15)$$

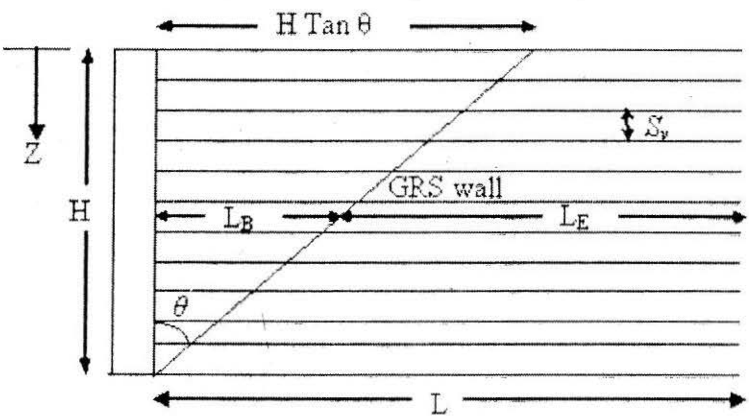

FIGURE 3 : Details showing the Length of Geotextile Reinforcement required for Stabilization and to Prevent Pull-out Failure

Length of the Geosynthetic Reinforcement Required for Stabilizing the Wall

Figure 3 shows the details of lengths of Geosynthetic reinforcements.

The vertical spacing of Geosynthetic reinforcement is:

$$S_v = \frac{T_{allow}}{\sigma_h FS}$$

The total length of Geosynthetic reinforcement required to stabilize the wall is

$$L = L_R + L_E$$

L_R ■☎H ✉Z ⓪tan □ (16)

The equation as suggested by Ling and Leshchinsky (1998) to calculate the length of embedment of the fabric layer in the anchorage zone L_E is adopted in the analysis.

$$L_E = \frac{T_{allow}}{(1+K_v)\gamma Z^2 C_i \tan\phi} \tag{17}$$

where

τ	=	soil-fabric interface shear strength;
FS	=	factor of safety (ranges from 1.3 to 1.5);
Z	=	$(H/2)$ = depth from the ground surface;
δ	=	angle of the friction between soil and fabric (assumed as $2/3$ of ϕ);
ϕ_h	=	total lateral pressure at the depth considered;
L_R	=	non-acting length of fabric behind the failure plane;
L_E	=	the length of embedment of the fabric layer in the anchorage zone; and
C_i	=	pull out coefficient, expressed as the ratio of soil-geosynthetic pull out strength to the soil strength (= 0.8 used in the present analysis).

The allowable stress in the fabric $T_{allow} = \dfrac{T_r}{FS} = \dfrac{T_r}{3.0}$

$$K_{allow} = \frac{T_{allow}}{\frac{1}{2}\gamma H^2} = \frac{1}{3}\left(\frac{T_r}{\frac{1}{2}\gamma H^2}\right) = \frac{K}{3}$$

$$L = (H - Z)\tan\theta + \frac{T_{allow}}{(1 + K_v)\gamma Z^2 C_i \tan\phi} \tag{18}$$

$$\frac{L}{H} = \frac{\left[\begin{array}{l} \tan\theta\left\{K_h + (1+K_v)\cot(\theta+\phi)\right\} \\ \quad -\alpha K_a\left\{\cot(\theta+\phi)\tan\phi_w + 1\right\} \\ \qquad -2\left(\dfrac{c}{\gamma H}\right)\left\{\cot(\theta+\phi) + \tan\theta\right\} \end{array}\right]}{6(1+K_v)\left(\dfrac{Z}{H}\right)^2 C_i \tan\phi} + \left(1 - \frac{Z}{H}\right)\tan\theta \tag{19}$$

Overturning Mechanism

After ensuring the internal stability of the wall it is essential to check the external stability. For overturning, moments are taken about the toe of the wall to form a factor of safety. Figure 4 shows wall overturning mechanism.

Defining factor of safety against overturning F_O as the ratio of the total resisting moments to the total driving moments and expressing it in a non-dimensional form we get.

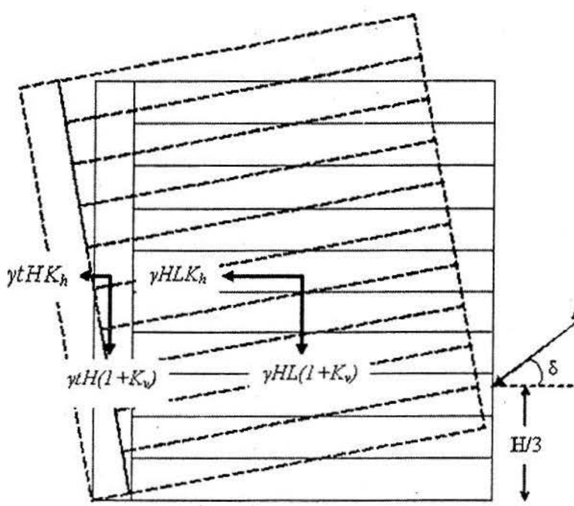

FIGURE 4 : GRS Wall is Overturning about the Tow of the Wall

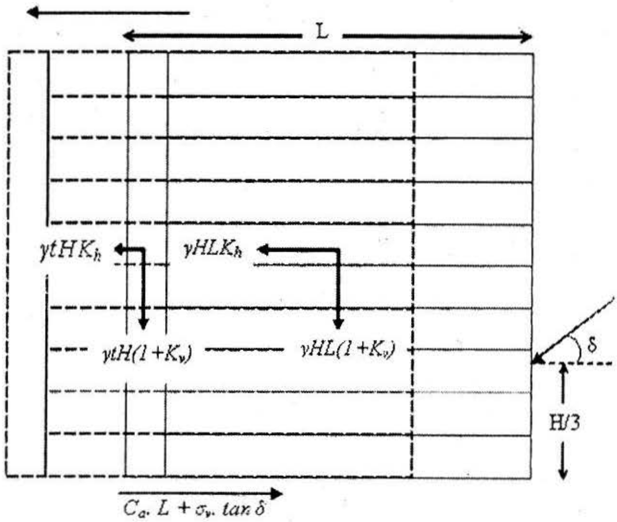

FIGURE 5 : GRS Wall is Sliding at the Base of the Wall

$$FS_{OT} = \frac{\left[\tan\theta\cdot(1+K_v)+K_a\cdot\sin\delta\cdot\left(\tan\theta+\frac{t}{H}\right)+\frac{\gamma_c}{\gamma}\left(\frac{t}{H}\right)^2(1+K_v)\right]}{\left[\frac{1}{3}K_a\cdot\cos\delta+\tan\theta\cdot K_h+\frac{\gamma_c}{\gamma}\frac{t}{H}K_h\right]} \tag{20}$$

Sliding Mechanism

Figure 5 shows the sliding mechanism. Defining factor of safety against sliding F_s as the ratio of the total resisting forces to the total driving forces and expressing it in a non-dimensional form we get.

$$F_S = \frac{\left(\frac{C}{\gamma H}\right)\tan\theta+\left[\tan\theta\cdot(1+K_v)+K_a\cdot\sin\delta+\frac{\gamma_c}{\gamma}\left(\frac{t}{H}\right)(1+K_v)\right]\tan\delta}{\left[K_a\cdot\cos\delta+\tan\theta\cdot K_h+\frac{\gamma_c}{\gamma}\frac{t}{H}K_h\right]} \tag{21}$$

Factor of safety values both against overturning (F_O) and sliding (F_S) are set to be either greater than or equal to 1.0.

Comparative Study and Validation of the Model

Using the above formulation the computed values of earth pressure coefficients are compared with reported solutions for validation. In the Eqn.13 if the values of $C/\gamma H$, t and K_V are set to zero expression for the active earth pressure coefficient in cohesionless soils can be obtained as,

$$K_a^{|} = \frac{\tan\theta\left[K_h + (1-K_v)\cot(\theta+\phi)\right]}{\left[\cot(\theta+\phi)\tan\phi_w + 1\right]}$$

From the above expression θ_{cr} is found out and the corresponding minimum value of $K_a^{|}$ is shown in Table 1 for different values of ϕ and K_h and compared with other solutions. It can be observed that the present solution is closest to those reported by Madhav and Kameswara Rao (1969) for both static and dynamic case. It is seen that till $K_h \leq 0.1$ for $\phi = 40^\circ$, the difference between the K_a values predicted by the present approach and by Madhav and Kameswara Rao (1969) is minimum and for $K_h \geq 0.1$ the difference is almost negligible. Values of earth pressure coefficient predicted by Saran and Gupta (2003) are always on the conservative side. However, the difference between these values and values predicted presently generally lies between 8 to 48 percent for static case; for dynamic case it is about 9 to 12 percent only.

Table 2 shows that the present solution and the solutions given by Ismeik and Guler (1998) for different t/H, ϕ and K_h values are in general either identical or differs by small amount. It is interesting to note that the discrepancy generally occurs for $t/H \geq 0.1$, $\phi \geq 25^\circ$ and $K_h \leq 0.1$. The study shows that the results obtained from the present analysis are correct and comparable to other solutions reported in literature. This further shows that results obtained with a single failure wedge are either identical or vary marginally for the two wedge failure reported by Ismeik and Guler (1998).

Results and Discussion

Parametric studies are made by varying the following parameters as follows:

$C/\gamma H$ = 0, 0.025, 0.05, 0.1, 0.2;
t/H = 0.10, 0.15, 0.20, 0.25, 0.3;
ϕ = 10°, 15°, 20°, 25°, 30°, 35°, 40°, 45°
K_h = 0, 0.1, 0.2, 0.3, 0.4, 0.5 and
$K_v = 0$, $K_v = K_h/2$, $K_v = K_h$.

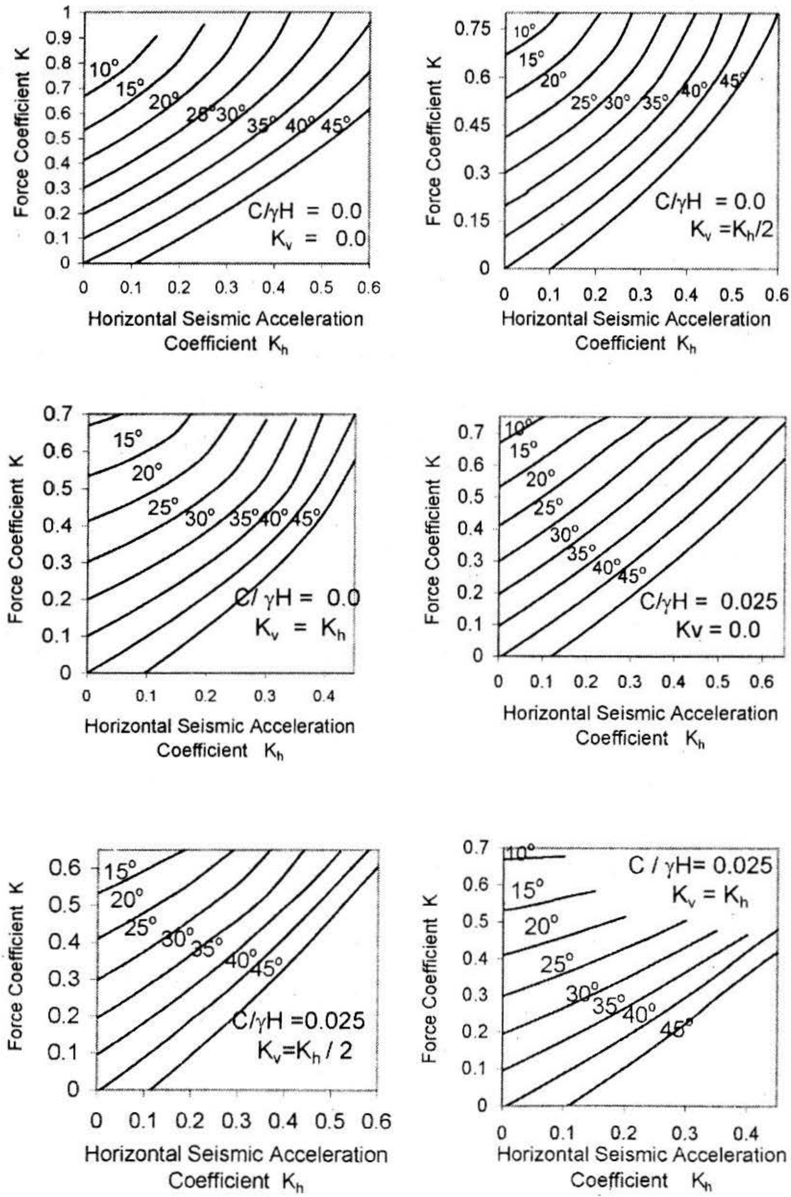

FIGURE 6 : Effect of K_h on Required Geosynthetic Tensile Reinforcement Force Coefficient, K for different K_h, K_v/K_h = 0, 0.5, 1.0 and for $C/\gamma H$ = 0, 0.025

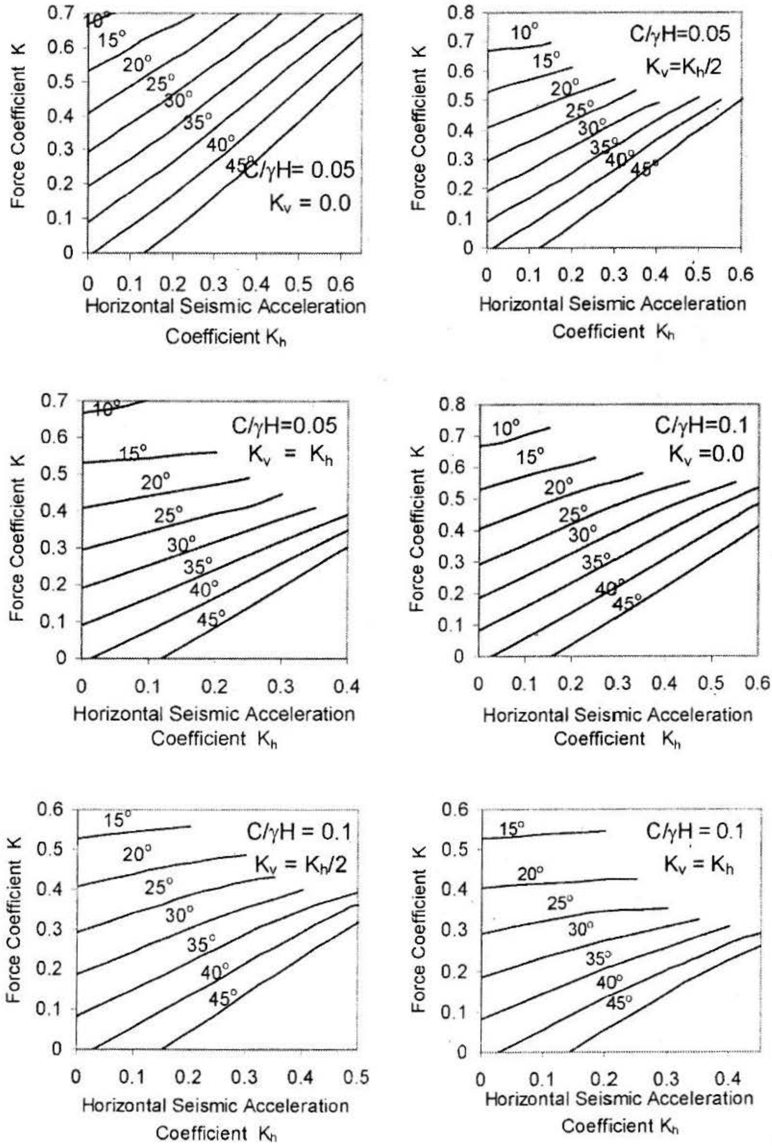

FIGURE 7 : Effect of K_h on Required Geosynthetic Tensile Reinforcement Force Coefficient, K for different K_h, K_v/K_h = 0, 0.5, 1.0 and for $C/\gamma H$ = 0.05, 0.1

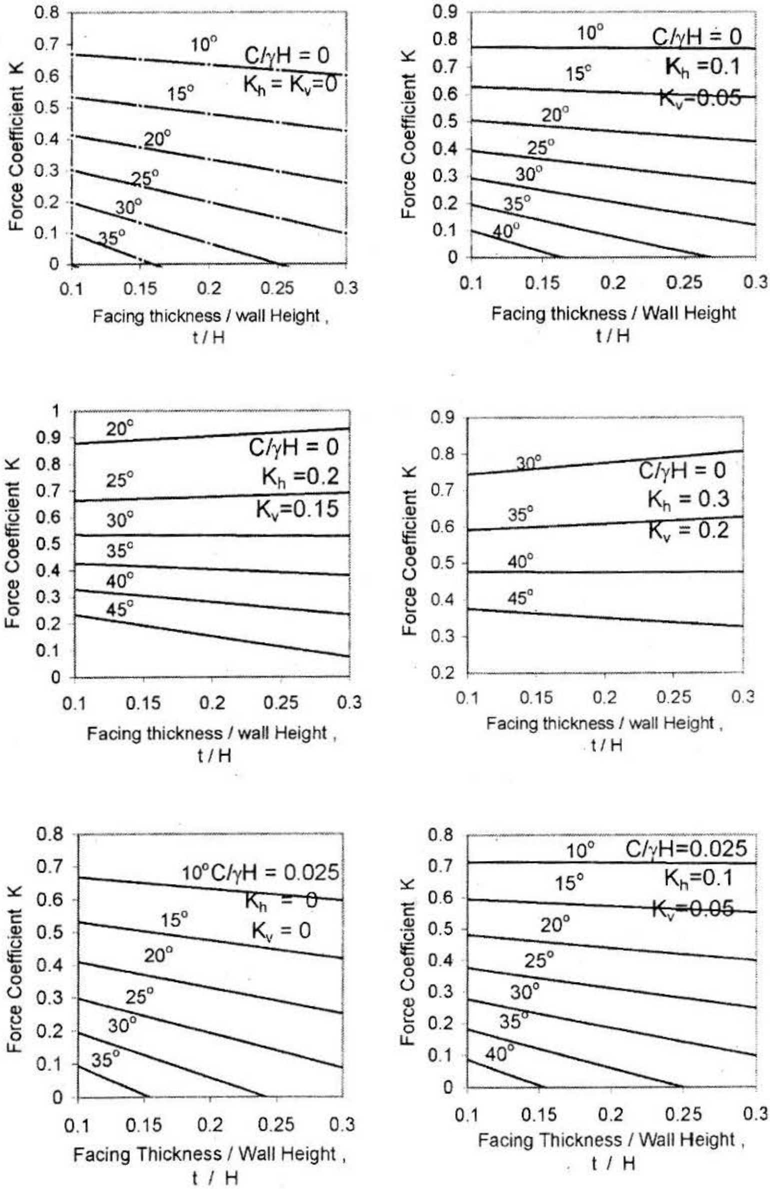

FIGURE 8 : Effect of Facing thickness, t on Required Geosynthetic Tensile Reinforcement Force Coefficient, K for different K_h, $K_v/K_h = -0.5$ and for $C/\gamma H$ = 0, 0.025

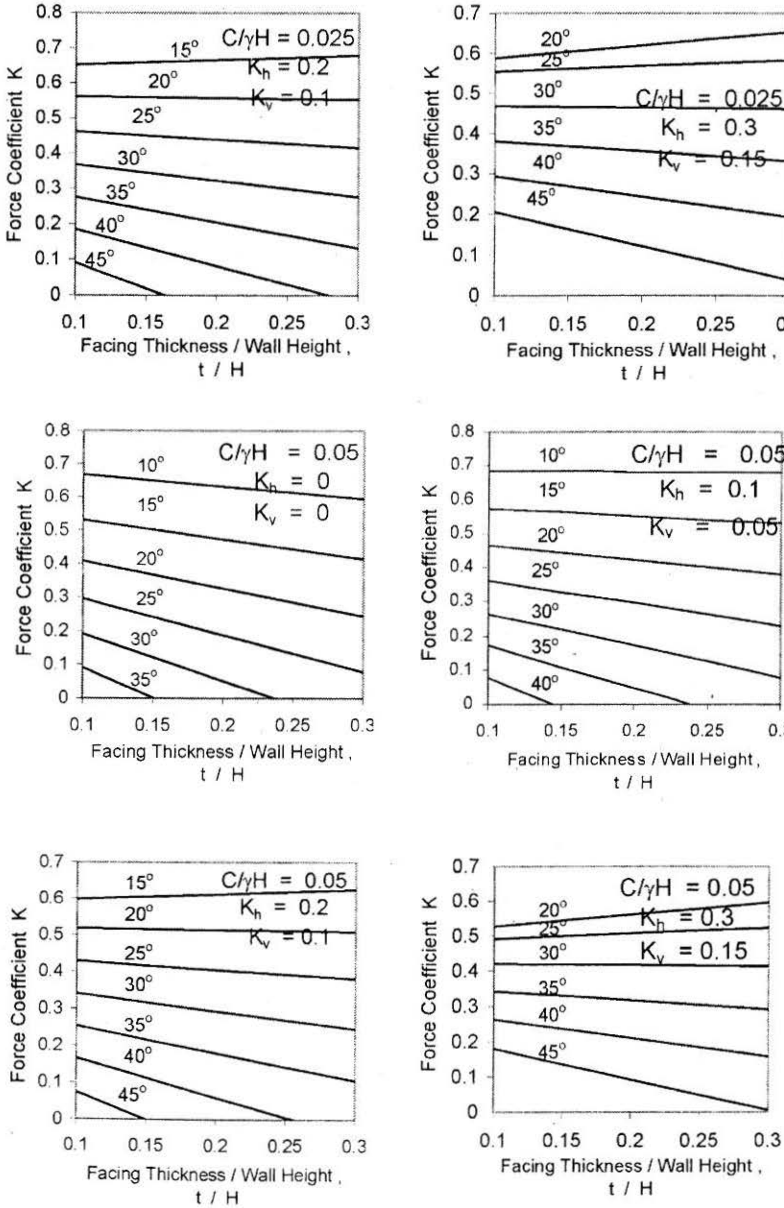

FIGURE 9 : Effect of Facing thickness, t on Required Geosynthetic Tensile Reinforcement Force Coefficient, K for different K_h, $K_v/K_h = -0.5$ and for $C/\gamma H$ = 0.025, 0.05

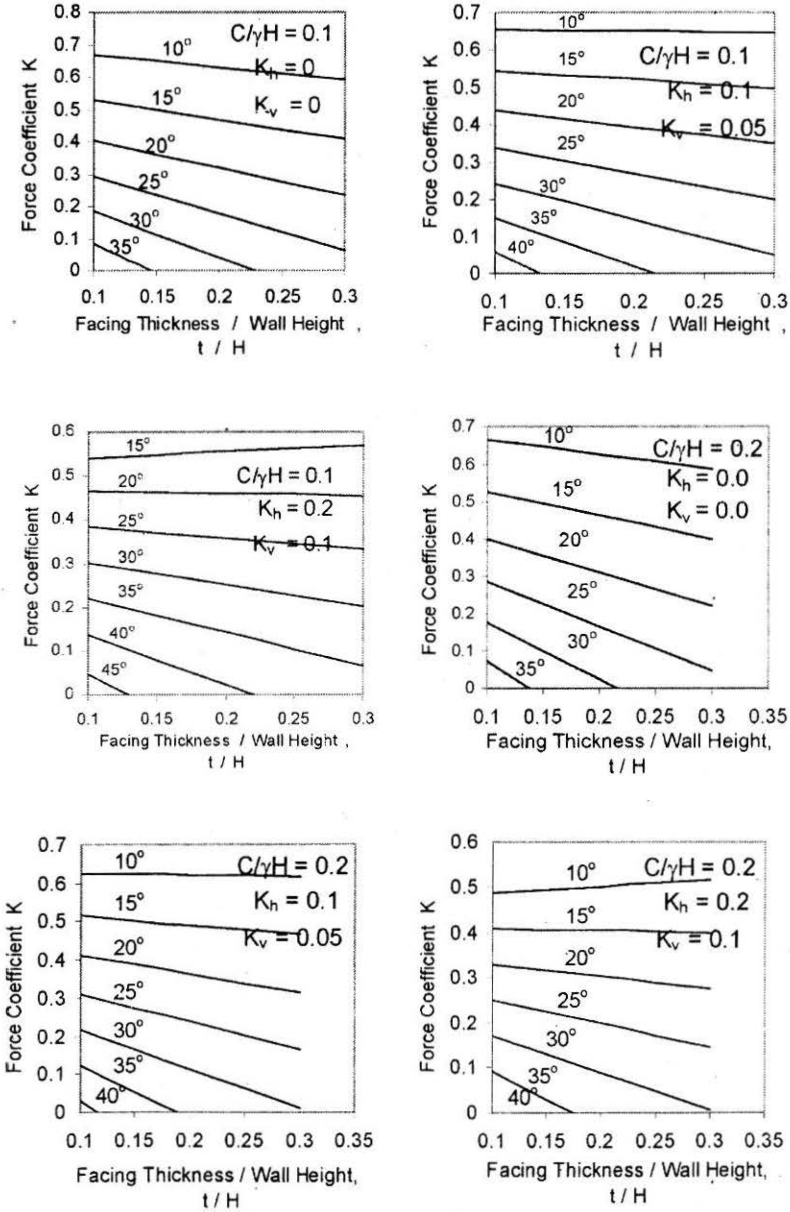

FIGURE 10 : Effect of facing thickness, t on Required Geosynthetic Tensile Reinforcement Force Coefficient, K for different K_h values, K_v/K_h = −0.5 and for $C/\gamma H$ = 0.1, 0.2

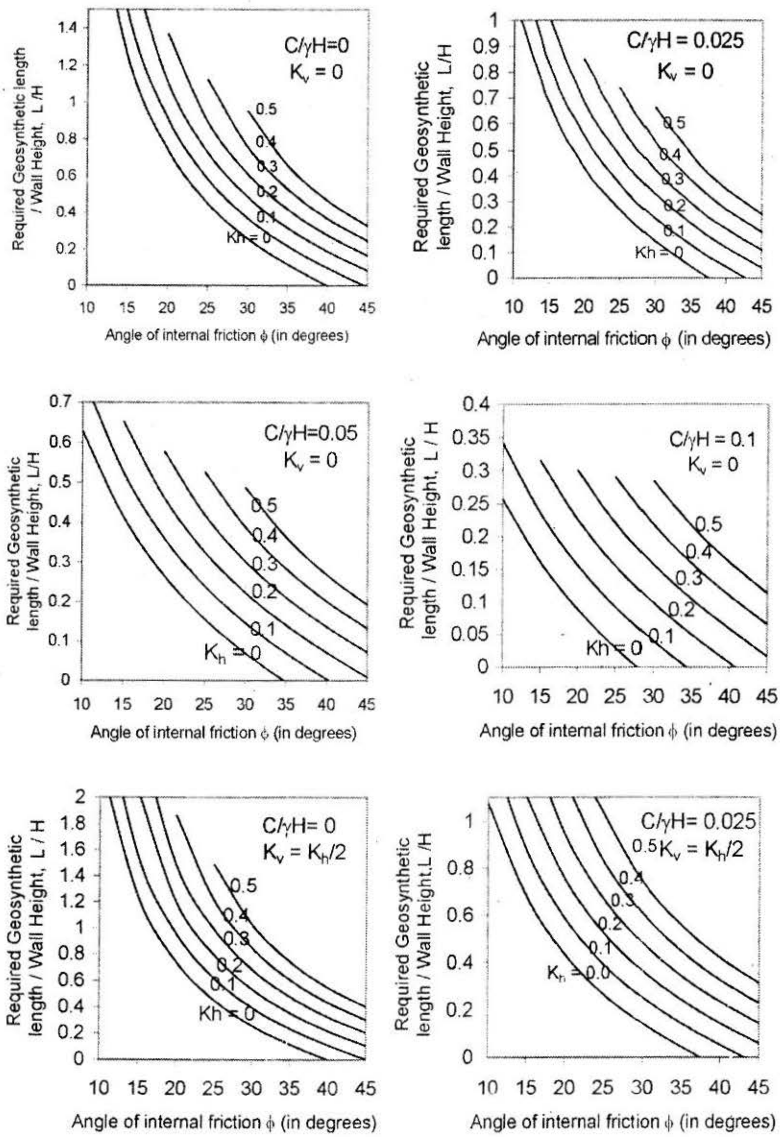

FIGURE 11 : Required Geosynthetic Length (L) versus Soil Friction Angle at different Sesmic Coefficients for $C/\gamma H$ = 0, 0.025, 0.05, 0.1 and for K_v = 0 and K_v = $K_h/2$

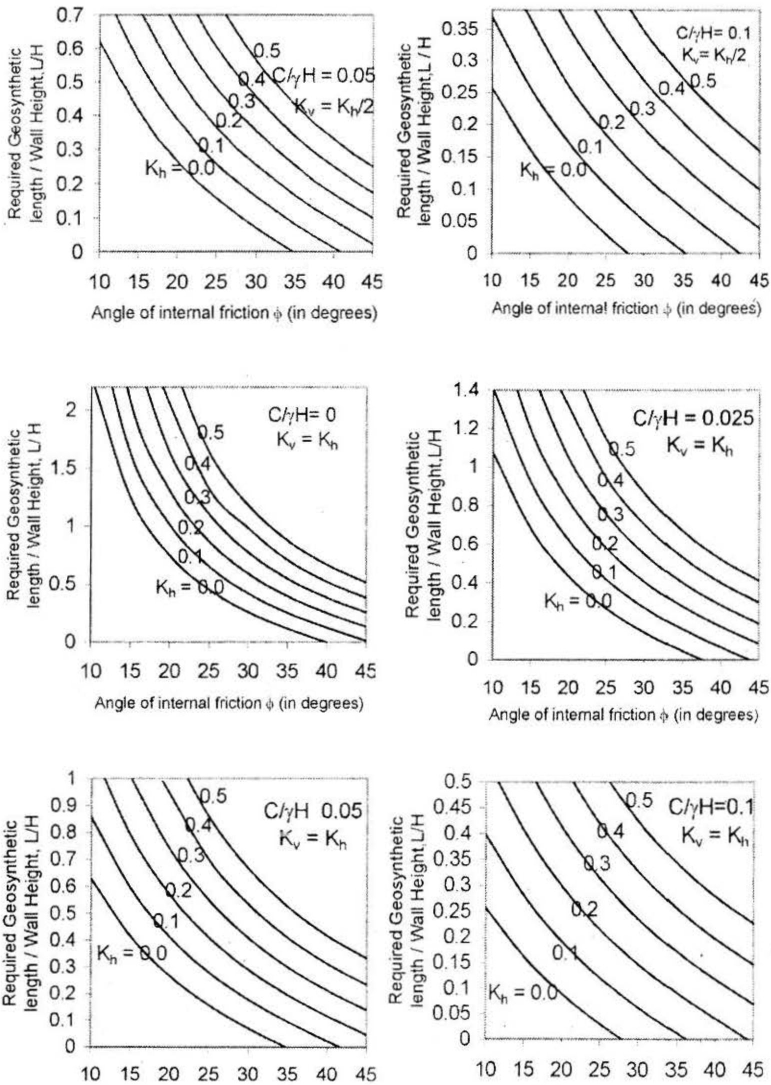

FIGURE 12 : Required Geosynthetic length (L) verses Soil Friction Angle at different Seismic Coefficient for C/γH = 0, 0.025, 0.1 and for $K_v = K_h/2$ and $K_v = K_h$

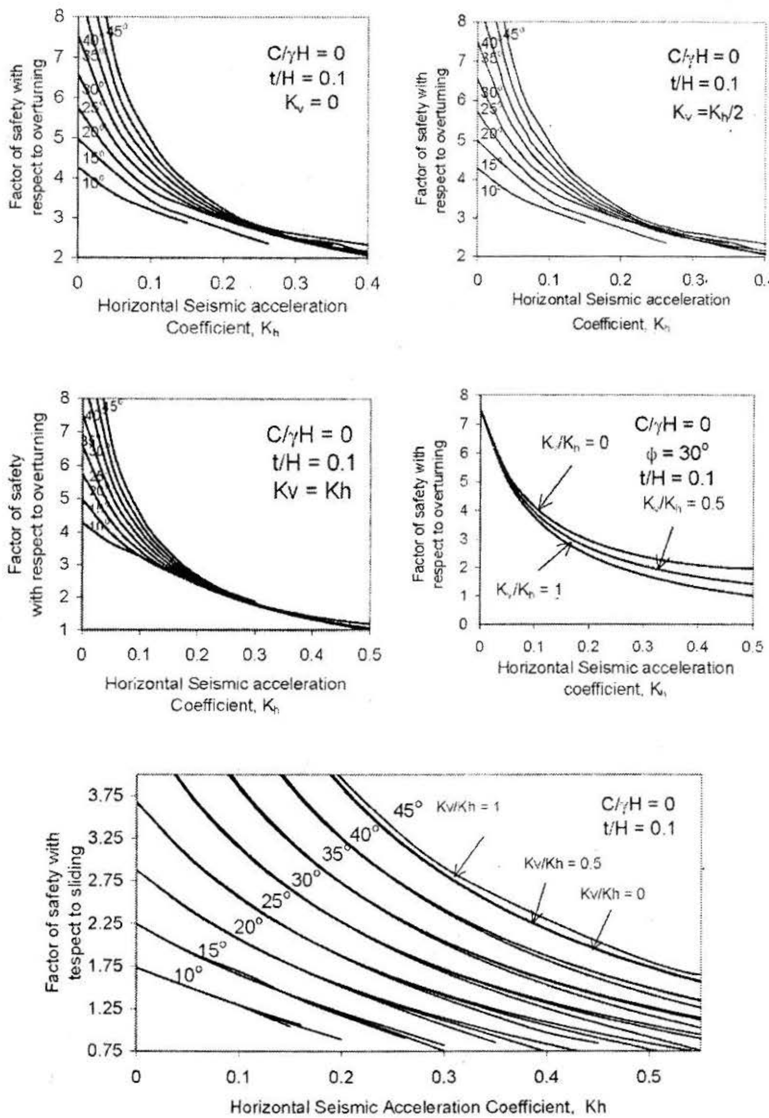

FIGURE 13 : Factor of Safety with respect to Overturning and Sliding for different Horizontal Seismic Acceleration Coefficients, Soil Friction Angle and for $C/\gamma H = 0.0$, $K_v/K_h = 0, 0.5, 1.0$

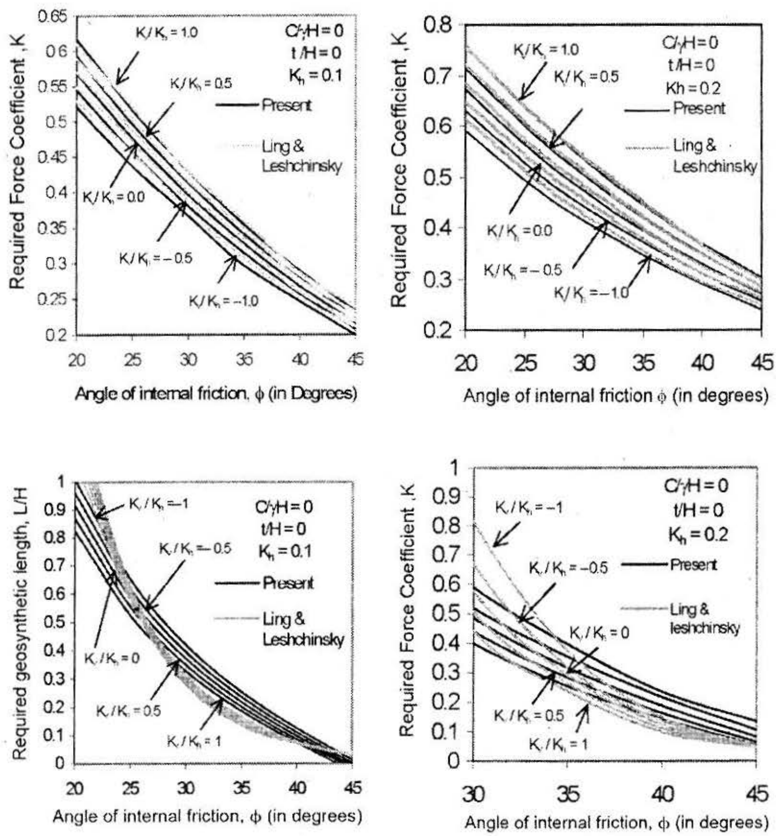

FIGURE 14 : Comparison of Present Results with Ling and Leshchinsky (1998)

The computed results are presented as design charts in Figs.6 to 15.

The results shown in Figs.6 and 7 indicate the variation of geosynthetics reinforcement force coefficient K with horizontal seismic acceleration coefficient K_h for different values of ϕ and stability number while K_v is acting in the downward direction. In those computations keeping all other parameters same K_v varies as 0, 0.5 K_h and K_h. It is seen from these figures that ϕ, $C/\gamma H$ and K_v remaining constant as K_h increases force coefficient also increases. It can also be observed that for any particular value of K_h as ϕ decreases the value of force coefficient K increases. This implies that for any earthquake force the stability of the wall with a less frictional soil can only be maintained by introducing more reinforcement. This is because in less frictional soil lesser internal shearing resistance will be available along the failure surface.

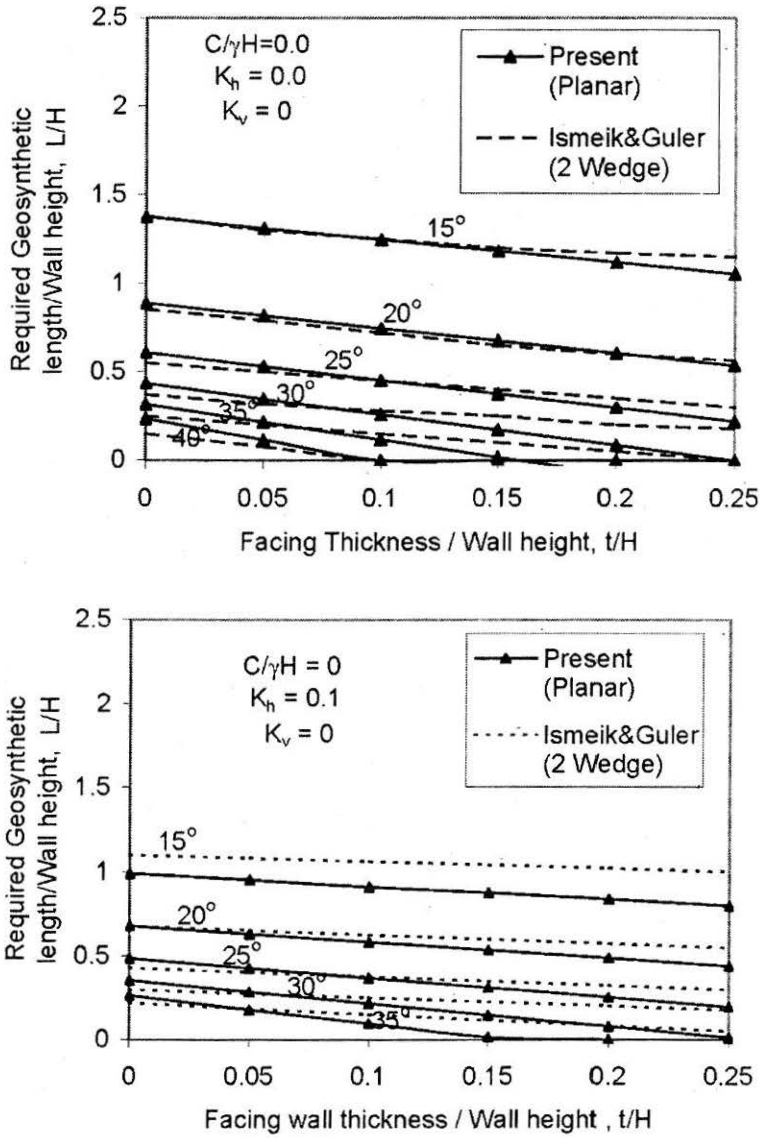

FIGURE 15 : Comparison of Present Results with Ismeik and Guler (1998)

The influence of the ratio of facing wall thickness to wall height on the reinforcement force coefficient is shown for different values of ϕ for a given number of stability number and vertical earthquake acceleration coefficient in Figs.8 through 10. It can be observed that as facing thickness ratios increases the force coefficient K in general decreases for all values of

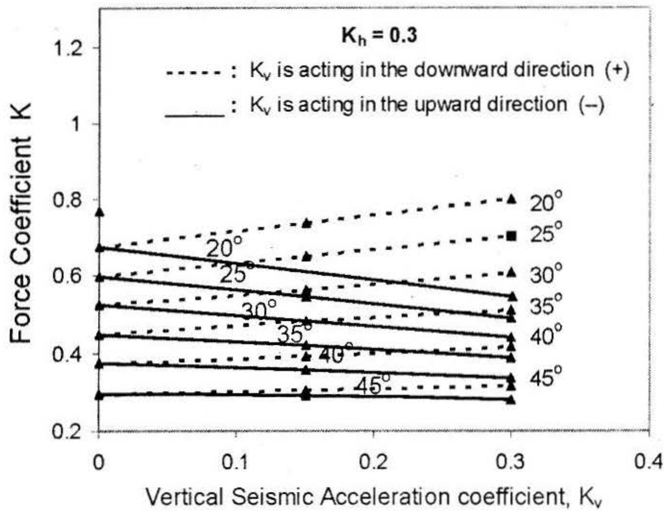

FIGURE 16 : Effect of Vertical Seismic Acceleration on Geosynthetic Reinforcement Force Coefficient for K_h = 0.3

ϕ. As the wall thickness increases, the wall is able to resist more of the superimposed earthquake force requiring less reinforcement force for the stability of the wall. But when K_h exceeds a value equal to 0.2, the reinforcement force required increases as wall facing thickness increases for soils having lower angle of internal friction (10°, 15°, 20° and 25°).

The ratio of the reinforcement length to the wall height is evaluated as a function of ϕ, K_h, $C/\gamma H$ and K_v. The results are presented as design charts in Figs.11 and 12. It can be seen from these figures that for the same value of K_h a greater Geosynthetic length is required if K_v is acting in the upward direction. It is observed that as ϕ increases and for the same K_h there is a sharp decrease in the required length. The required length ratio decreases with the increase in ϕ. For the same ϕ as K_h increases the required length of reinforcement also increases.

Figure 13 shows the variation of factor of safety of the GRS wall against overturning and sliding with horizontal seismic acceleration for various values of ϕ, K_v/K_h ratio and wall thickness ratio (t/H) for cohesionless soil. It can be seen from these figures that as K_h increase the values of F_s decrease indicating decreased wall stability with increasing seismic forces. For the same K_h as ϕ increases, and also increase indicating increased wall stability as with increased values of ϕ greater shearing resistance is available.

In Figs.14 and 15 a comparison of the results obtained by using the present approach with those obtained by Ling and Leshchinsky (1998) and Ismeik and Guler (1998) are presented. The comparison shows that for $t/H = 0$, there is an excellent agreement between the present solution (Force coefficient K) to that of Ling and Leshchinsky (1998) when K_v acts in the downward direction. When K_v acts in the upward direction significant variation occurs in the estimated required reinforcement length. For higher values of ϕ the predictions of Ling and Leshchinsky (1998) are in general lower than the present solutions. The present results (variation of L/H with t/H) are in close agreement with those of Ismeik and Guler (1998) for static case. Some deviation is observed for ϕ greater than 30°; under dynamic condition the predictions of Ismeik and Guler (1998) are in general higher than the present solution for t/H greater than 0.1. The above charts are quite handy in designing a GRS wall when subjected to seismic forces.

Conclusions

The following conclusions are made from the present study.

(a) The results for GRS retaining walls from the present method using planar failure surface when compared with the existing methods show the following:

   (i) Difference of the predictions from that Ling and Leshchinsky (1998) using curved failure surface is negligible.

   (ii) Predictions made by Ismeik and Guler (1998) are generally higher than the present predictions.

(b) The required amount of Geosynthetic reinforcement force and the corresponding reinforcement length for the seismic stability of a vertical retaining wall can be estimated using the expressions that have been developed as a function of the facing thickness, shear strength parameters, C and Φ, and seismic loads.

(c) When vertical component of earthquake acceleration is acting in the downward direction in addition to a horizontal component of earthquake acceleration, for the same wall thickness Geosynthetic reinforcement force (T_r) increases as the horizontal seismic acceleration increases; but the same decreases as wall-facing thickness increases. With the same value of horizontal earthquake acceleration a greater Geosynthetic length is required if vertical component of earthquake acceleration acts in the upward direction.

(d) Factor of safety with respect to overturning and sliding decreases as the horizontal component of earthquake acceleration increases. With K_h remaining the same, if K_v (acting in the downward direction) increases, factor of safety values decreases further. If K_h is small the effect of K_v is negligible. But when K_h exceeds 0.2, both reinforcement force and geosynthetic length are affected greatly by K_v. This effect is more pronounced for soil having low angle of internal friction.

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Notations

a_{0h} = Horizontal seismic acceleration (m/sec^2)

a_{0v} = Vertical seismic acceleration (m/sec^2)

C	=	Cohesion of the backfill material (N/m^2)
C_I	=	Pull out coefficient, expressed as the ratio of soil-geosynthetic pull out strength to the soil strength (dimensionless)
FS	=	Factor of safety (dimensionless)
F_O	=	Factor of safety with respect to Overturning (dimensionless)
F_S	=	Factor of safety with respect to Sliding (dimensionless)
g	=	Acceleration due to gravity (m/sec^2)
H	=	Wall height (m)
H_f	=	Horizontal seismic force acting on the facing (N/m)
K	=	Geosynthetic reinforcement force coefficient (dimensionless)
K_a	=	Combined active earth pressure coefficient (dimensionless)
K_h, K_v	=	Horizontal, vertical seismic acceleration coefficient (dimensionless)
L_R	=	Non-acting length of fabric behind the failure plane (m)
L_E	=	Length of embedment of the fabric layer in the anchorage zone (m)
N_f	=	Normal force on base of wall facing (N/m)
M	=	Earthquake intensity on Richter scale (dimensionless)
N	=	Normal force on base of wedge (N/m)
$N_{a\gamma}$, N_{ac}	=	Mononobe and Okabe active earth pressure coefficients due to unit weight, cohesion of the backfill material (dimensionless)
P	=	Force contributed by the facing to the wall stability (N/m)
P_a	=	Active earth pressure (N/m)
S	=	Shear force at the base of the facing;
t	=	Thickness of facing wall (m)
T_r	=	Amount of geosynthetic reinforcement force to

		stabilize the wall (N/m)
T_{allow}	=	Allowable geosynthetic reinforcement force to stabilize the wall (N/m)
V	=	Shear force between wall facing and backfill (N/m)
W	=	Weight of wedge (N)
Z	=	Depth from the ground surface (m)
τ	=	Shear strength of the soil to the fabric
δ	=	Angle of friction between soil and fabric (deg.)
ϕ_f	=	Friction angle between the wall facing and backfill (deg.)
φ_w	=	Friction angle between the wall facing and the backfill (deg.)
φ	=	Friction angle of the backfill material (deg.)
γ_c	=	Unit weight of concrete (N/m^3)
γ	=	Unit weight of backfill material (N/m^3)
α	=	Fraction of active earth pressure (dimensionless)
θ	=	Angle of the failure plane with vertical (deg.)
σ_h	=	Total lateral pressure at depth considered (N/m^2)