SEISMIC STABILITY OF REINFORCED RETAINING WALL

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ABSTRACT:
The use of pseudo-static methods for the evaluation of soil thrust acting on retaining wall under seismic condition is well established in the design of such structures. But the most common design method is still based on the limit equilibrium of the soil wedge without taking into account the presence of the wall. This paper presents a new solution based on the pseudo-static equilibrium of the soil-wall-reinforcements system which considers horizontal acceleration. The developed solution takes into account the effect of the presence of the wall and reinforcements. Stability analyses are conducted to determine the required strength of reinforcements and critical inclination of the failure angle. Parametric studies illustrate the effects of seismic acceleration on the design of reinforced retaining wall and also the forces in the reinforcements. The force of reinforcements required to resist direct sliding increase rapidly as the seismic acceleration increases. A detailed design example is included to illustrate usage of proposed procedures.

KEYWORDS:
Seismic design; Reinforced retaining wall; Pseudo-static analysis

1. INTRODUCTION

Seismic designs of geotechnical earth structures, such as slopes, retaining walls, embankments and dams, are conducted routinely using a pseudo-static approach. The Mononobe (1924) and Okabe (1924) approach for retaining wall design, is the most well-known pseudo-static procedures. It is considered an earth pressure approach where the solution is obtained by extending Coulomb's analysis. Pseudo-static stability analysis that uses a mechanism at a prescribed failure plane has been addressed by several investigators (Seed and Goodman (1964), Sarma (1975) and Ling and Cheng (1997)).These studies all assume the inertia force due to an earthquake horizontal acceleration for a failure soil mass along a prescribed plane. The first seismic design procedure for metal strip reinforced soil structures was proposed by Richardson & Lee (1975). It was based on the Mononobe–Okabe analysis (Mononobe, 1924; Okabe, 1924). A planar failure surface was assumed and a dynamic earth pressure component was added to the static component in determining the required reinforcement force. Bonaparte et al. (1986) proposed a pseudo-static limit equilibrium approach for designing reinforced slopes. The geosynthetics length and strength required to resist these failure modes were presented in several design chart. This approach does not consider permanent displacement. Ling et al. (1997) conducted a seismic design for designing geosynthetics-reinforced slopes base on a pseudo-static limit equilibrium analysis, which considers horizontal acceleration and incorporates a permanent displacement limit. Internal and external stability analysis conducted to determine the required strength and length of geosynthetics, considering different modes of failure.

Motta (1994) proposed a general closed-form solution for the case of uniformly distributed surcharge applied on the unreinforced back- fill soil to a certain distance from the top of the wall. In this solution seismic effects were included pseudo-statically by the introduction of seismic coefficients. Caltabiano et al. (2000) used the pseudo-static methods for the computation of unreinforced soil thrust acting on retaining walls under seismic condition. Their solution takes into account the effect of the presence of the wall and it applied to soil-wall systems with surcharged backfills. Formulas are provided to calculate directly the yield acceleration and the inclination of the failure surface.

In this paper a modified limit equilibrium approach in seismic condition is described. The seismic inertia force is considered pseudo-static. In contrast to many other procedures, the two proposed in this paper is considered (1) instead of analyzing a structure with prescribed geosynthetics lengths and forces, these values are determined from the analysis to satisfy prescribed performance criteria and (2) the solution takes into account the effect of the presence of the wall. A parametric study to review the relative importance of some of the design parameters such as seismic coefficient, friction angle of soil was conducted.

2. THEORITICAL MODEL

The analysis of the seismic limit equilibrium condition of walls retaining surcharged backfill soil is based on the following assumptions:

- the system slides from the former to the latter condition;
- the soil-wall system is long enough for the end effects to be neglected (plane-strain conditions);
- the soil is homogeneous, coarse-grained cohesionless soil; as the effect of pore-water pressure is neglected and, therefore, liquefaction is not a concern;
- the failure wedge is a plane;
- the soil-wall system can be subjected only to horizontal displacements;
- the seismic action is at any instant, constant in the whole soil-mass and wall and is directed horizontally.

The reinforced soil-wall system considered in the analysis is schematically shown in Fig. 1, where H is the backfill height or height of the wall.

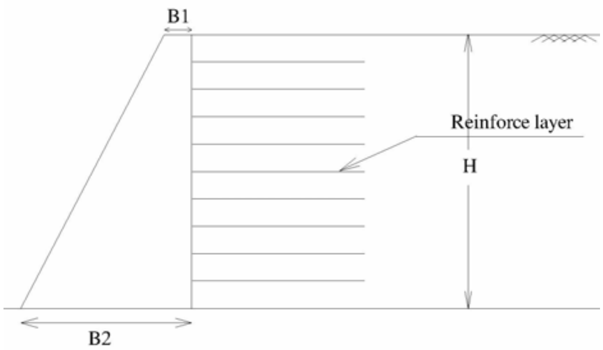

Figure 1 Reinforced soil-wall system

During earthquake the reinforced soil-wall system may either move together with the ground or move relatively respect to the ground. These two conditions are referred to as absolute motion and relative motion, respectively; the system shifting from the former to the latter condition depends on the value of the seismic horizontal acceleration $a_h = k_h g$, which k_h and g are the horizontal seismic coefficient and gravity acceleration, respectively. The geometry and acting forces of the system which considered in the analysis is shown in Fig. 2.

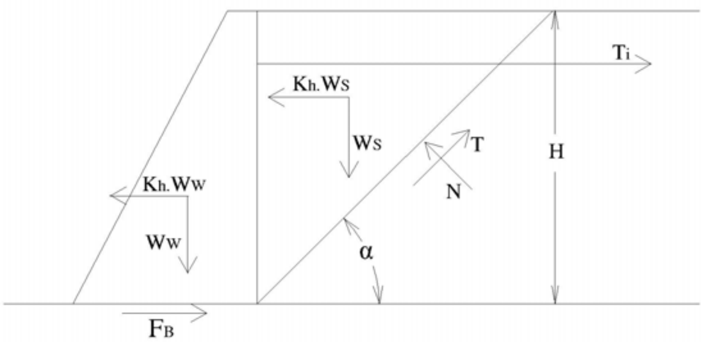

Figure 2 The geometry and acting forces of the reinforced soil-wall system

For this system, the dynamic equilibrium condition in X and Y direction are given in Eqs. (1) and (2); respectively.

$$\begin{aligned} &\sum F_X = 0 \quad \text{(for the whole system)} \\ \Rightarrow \quad & T\cos\alpha - N\sin\alpha - K_h.W_S - K_h.W_W + F_B + \sum T_j = 0 \end{aligned} \tag{1}$$

$$\begin{aligned} &\sum F_Y = 0 \, \text{(for the whole system)} \\ \Rightarrow \quad & T\sin\alpha + N\cos\theta - Ws = 0 \end{aligned} \tag{2}$$

The Equations (1) and (2) can be solved, simultaneously. Thus, the dynamic equilibrium condition obtains by the following expression:

$$F_B + \sum_{j=1}^{m} T_j - K_h.W_W = Ws\left[K_h - tg(\alpha - \varphi)\right] \tag{3}$$

where

$$F_B = \mu.W_W \tag{4}$$

is the resistance at the base of the wall, with $\mu = tg(\frac{2}{3}\varphi_B)$ soil-wall base friction coefficient, W_w and W_s are weight of the wall and weight of the soil failure wedge, respectively. W_s can be written as follow:

$$W_S = \frac{\gamma H^2}{2tg\alpha} \tag{5}$$

The forces and parameters used in Eqs. (1)– (5) are shown in Fig. 2, where T and N are, respectively, the shear (tangential) and normal forces acting on the failure plan of reinforced soil; Tj is the mobilized tension force in the reinforcement j, located in the soil failure wedge horizontally and m is the number of reinforcements. The Safety Factor (FS) is assumed to be equal one for the whole system wall. The soil internal friction angle (j) is obtained using Eq. (4). The parameter α is the angle formed by the failure surface with respect to the horizontal, γ is the unit weight of the soil and friction φ is the angle of shear strength of the soil. Introducing Eqs. (4) and (5), after simple calculation, Eq. (3) becomes:

$$\left(\Gamma(\mu - K_h) + K\right)(1 + Y\Phi)Y - \left[K_h(1 + Y\Phi) + Y - \Phi\right] = 0 \tag{6}$$

Where $Y = tg\alpha$, $\Phi = tg\varphi$, $\Gamma = \frac{2W_W}{\gamma H^2}$ and $K = \frac{2\sum_{j=1}^{m} T_j}{\gamma H^2}$ are all dimensionless quantities.

Eq. (6) can be solved with respect to $Y = tg\alpha$ obtaining a second power equation:

$$Y^2\left[\Gamma(\mu - K_h)\Phi + K\Phi\right] + Y\left[\Gamma(\mu - K_h) - (1 + K_h\Phi) + K\right] - (K_h - \Phi) = 0 \tag{7}$$

Also Eq. (6) can be solved with respect to K:

$$K = \frac{\{K_h(1 + Y\Phi) + (Y - \Phi)\}}{(1 + Y\Phi)Y} - \left[\Gamma(\mu - K_h)\right] \tag{8}$$

The maximum sum of the forces needed to maintain the stability of the reinforced retaining wall $(\sum_{j=1}^{m} T_j)_{\max}$ or ($K = K_{\max}$) corresponding to the most critical inclination of the failure plan angle, can be obtained to be set the first-order partial deviation of the K over the parameter $Y = tg\alpha$ equal zero as below:

$$\frac{\partial K}{\partial \alpha} = 0 \quad .OR. \quad \frac{\partial K}{\partial Y} = 0 \tag{9}$$

Then, Eq. (9) can be arranged with respect to $\alpha = \alpha_{cri}$ obtaining a second power equation:

$$Atg^2\alpha_{cri.} + Btg\alpha_{cri.} + C = 0 \tag{10}$$

with the positions:

$$A = \Phi(1 + K_h\Phi) \qquad B = 2\Phi(K_h - \Phi) \qquad C = K_h - \Phi \tag{11}$$

which solved gives the following expression for the critical inclination of the failure plan angle:

$$\alpha_{cri.} = tg^{-1}\left(\frac{-B \pm \sqrt{B^2 - 4AC}}{2A}\right) \tag{12}$$

Thus the maximum value of $K = K_{\max}$ can be derived with imposing the value of $\alpha = \alpha_{cri}$ in Eq. (8).

3. RESULTS

To illustrate the behavior of reinforced retaining wall under seismic inertia force, a computer program has been developed, which can be used to attain the critical inclination of the failure plan angle and required total geosynthetic force. The Geometric of soil-wall system (H, B1 and B2 in Fig. 1) utilized in the parametric analysis is that considered by Nadimand and Whitman (1983). A series of parametric study have been carried out in two cases (1) without presence of the wall and (2) with presence of the wall using the geotechnical, geometrical and design parameters detailed in Table 1. The obtained results of system, with presence of wall are compared to those obtained for the case of system without presence of wall.

Table 1.Geometric and geotechnical characteristics of reinforced retaining wall used for parametric study and comparison

Description	Value
Height of the wall (H)	8.0 m
B1	0.8 m
B2	5.0 m
Unit weight of the soil (γ)	18 kN/m^3
Unit weight of the wall (γ_{wall})	24 kN/m^3
Internal angle of soil friction (φ)	10, 15, 20, 25, 30, 35, 40, 45
Soil cohesion (C)	0
Coefficient of horizontal seismic acceleration (k_h)	0.0, 0.1, 0.15, 0.2, 0.25, 0.3
Coefficient of vertical seismic acceleration (k_v)	0.0

3.1. Results without Presence of Wall

3.1.1. Critical inclination of the failure plan angle

Fig. 3 shows the critical inclination of the failure plan angle; α_{cri} for different internal angle of soil friction (φ) under static and seismic loadings (k_h =0.0, 0.1, 0.15, 0.2, 0.25, and 0.3). The results show that for the certain value of k_h, the value of α_{cri} increases with increasing the internal angle of soil friction (φ). It means that when internal angle of soil friction (φ) increases, the volume of the critical sliding mass reduces. Also, for the certain value of φ, the critical inclination of the failure plan angle (α_{cri}) decreases with increasing the value of k_h. It means that when k_h increases, the weight of the soil failure wedge; W_s (the volume of the critical sliding mass) increases.

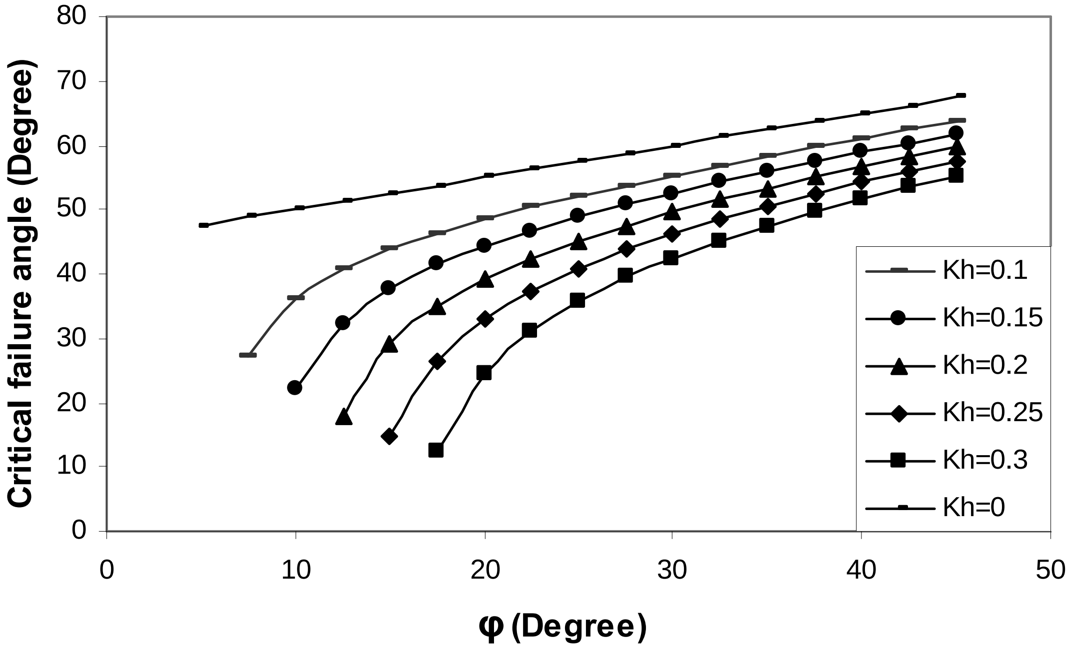

Figure 3 Critical inclination of the failure plan angle (α_{cri}) for different internal angle of soil friction (φ) under static and seismic loadings (k_h =0.0, 0.1, 0.15, 0.2, 0.25, and 0.3)

3.1.2. Required total geosynthetic force

Fig. 4 shows the required maximum total geosynthetic force $(\sum_{j=1}^{m} T_j)_{\max}$ for different internal angle of soil friction (φ) under static and seismic loadings (k_h =0.0, 0.1, 0.15, 0.2, 0.25, and 0.3). The value of $(\sum_{j=1}^{m} T_j)_{\max}$ is normalized to give $K_{\max}$, which is analogous to the earth pressure coefficient used in conventional retaining wall design. That is:

$$K_{\max} = \frac{2(\sum_{j=1}^{m} T_j)_{\max}}{\gamma H^2} \tag{10}$$

The results show that for the certain value of k_h, the value of $K_{\max}$ decreases with increasing the internal

angle of soil friction (φ). It means that due to increasing the internal angle of soil friction (φ), the stability of retaining wall increases and the total geosynthetic mobilized force reduces.
Also, for the certain value of φ, the value of $K_{\max}$ increases with increasing the value of k_h. It means that when k_h increases, the weight of the soil failure wedge; W_s (the volume of the critical sliding mass) increases. It means that due to increasing the seismic coefficient (k_h), in order to provide the stability of retaining wall, the total bearing capacity of geosynthetic should be enhance.

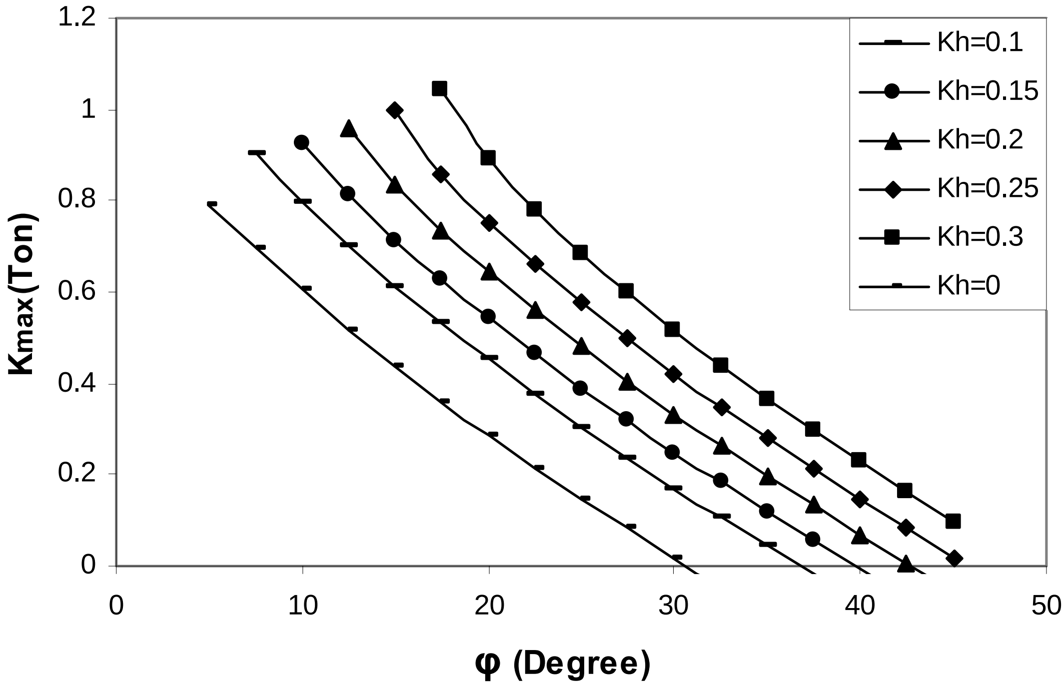

Figure 4 Required maximum total geosynthetic force ($K_{\max}$) for different internal angle of soil friction (φ) under static and seismic loadings (k_h =0.0, 0.1, 0.15, 0.2, 0.25, and 0.3)

3.2. Results with Presence of Wall

3.2.1. Critical inclination of the failure plan angle

Equations 10-12 show that the presence of wall is ineffective on the value of Critical inclination of the failure plan angle (α_{cri}) and therefore the results for values of this parameter are similar to Fig. 3. it should be noted that the effect of presence of wall is only on the required total geosynthetic force which discussed in sec. 3.2.2.

3.2.2 Required total geosynthetic force

To illustrate the effect of seismic inertia force (effect of considering the wall), the variation of the required maximum total geosynthetic force $(\sum_{j=1}^{m} T_j)_{\max}$ versus internal angle of soil friction (φ) under static and seismic loadings (k_h =0.0, 0.1, 0.15, 0.2, 0.25, and 0.3) are shown in Fig. 5. It should be noted that based on height of the wall (H) and unit weight of the soil (γ), the weight of the wall (W_w) obtains of 55 ton. It illustrates that for the value of $(\mu > K_h)$ presence of wall inertia decreases the value of $K_{\max}$. In contrast with, for the value of $(\mu < K_h)$ presence of wall inertia increases the value of $K_{\max}$.On the other hand considering the presence of wall in equilibrium equation in most research can be cause uncertainty in practical case.

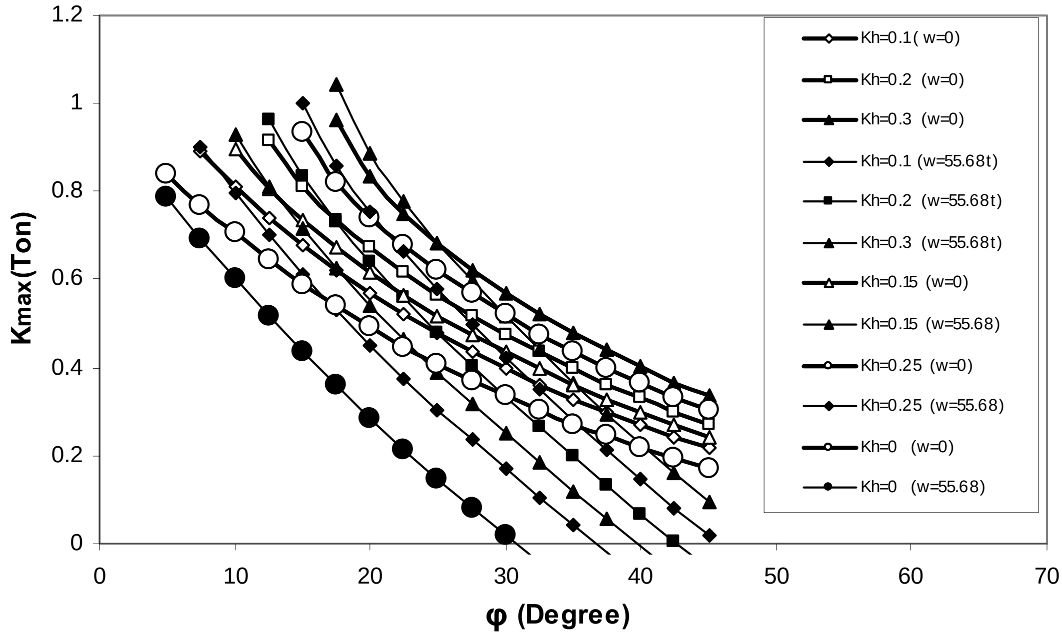

Figure 5 Effect of presence of the wall on required maximum total geosynthetic force ($K_{\max}$) for different internal angle of soil friction (φ) under static and seismic loadings (k_h =0.0, 0.1, 0.15, 0.2, 0.25, and 0.3)

4. SUMARRY AND CONCLUSIONS

In this paper a modified limit equilibrium approach for the seismic stability of reinforced gravity retaining wall has been described. The main difference with respect to the traditional solutions is that the presence of the wall is considered in the equilibrium equations. This approach allows the required maximum total geosynthetic force and the associated critical inclination of the failure plane angle for a given reinforced soil-wall system to be determined directly. Based on analyses performed in this study, the following general remarks can be made:

1. The results show that for the certain value of seismic loadings k_h , with increasing the internal angle of soil friction (φ) the value of the critical inclination of the failure plan angle (α_{cri}) increases and the value of $K_{\max}$ decreases. It means that due to increasing the internal angle of soil friction (φ), the stability of retaining wall increases and the total geosynthetic mobilized force reduces.

2. Also for the certain value of φ , the value of $K_{\max}$ increases with increasing the value of k_h . It means that when k_h increases, the weight of the soil failure wedge; W_s and hence in order to provide the stability of retaining wall, the total geosynthetic force should be enhance.

3. Considering the effect of wall illustrates that for value of $(\mu > K_h)$ presence of wall inertia decreases the value of $K_{\max}$. In contrast for value of $(\mu < K_h)$ presence of wall inertia increases the value of $K_{\max}$ .On the other hand, considering the presence of wall in equilibrium equation in most research can be cause uncertainty in practical case.

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