Seismic stability analysis of reinforced soil structures using pseudo-static method

Analyse de la stabilité sismique des structures de sol renforcé pseudo-méthode statique

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ABSTRACT

The paper pertains to the pseudo-static seismic stability analysis of reinforced soil structures. Using limit equilibrium method and assuming the failure surface to be logarithmic spiral, analysis has been conducted to maintain internal stability against both tensile and pullout failure of the reinforcements. The influence of the intensity of the surcharge load placed on the backfill is also considered in the analysis. Studies have also been made regarding the influence of backfill soil friction angle, horizontal seismic accelerations, the tensile strength of reinforcement, pullout length of the reinforcement and number of reinforcement layers on the seismic stability against tension and pullout modes of failure.

RÉSUMÉ

Le document se rapporte à la pseudo-analyse de la stabilité statique sismique du renforcement de structures de sol. Utilisation de la méthode d'équilibre limite et en supposant que la surface à l'échec spirale logarithmique, l'analyse a été effectuée afin de maintenir la stabilité interne, à la fois contre la résistance à la traction et l'échec du retrait des renforts. L'influence de l'intensité de la surcharge de charge placée sur le remblai est également pris en considération dans l'analyse. Des études ont également été accomplis en ce qui concerne l'influence de l'angle de frottement du sol de remblai, les accélérations sismiques horizontales, la résistance à la traction de l'armature, l'arrachement de la longueur et le nombre de renforcement des couches de renforcement de la stabilité sismique contre les tensions et les modes de retrait de l'échec.

Keywords : limit Equilibrium method, pseudo-static method, factor of safety, reinforced earth walls, seismic stability

1 INTRODUCTION

Stability analysis of reinforced earth walls under earthquake loading is one of the most important topics in Geotechnical engineering and drawn the attention of researchers. Conventional earth structures are prone to catastrophic failure during earthquakes. One of the reasons of collapse is the softening behavior of the soil which results in accumulation of the strain energy in the shear bands during the earthquakes that ultimately causes large displacements. In contrast to the conventional retaining structures, reinforced soil structures (RSS) are well known for their improved performance during earthquakes. As such, design procedures are being evolved for such structures considering their stability under earthquake loading and still there exists scope for further development of a simple but improved method to analyze and design such mechanically stabilized walls with inclusions under seismic condition. In the following sections, review of the currently developed analyses procedures and the objective and scope of the present paper are presented.

The most common approach to seismic stability evaluation of reinforced earth structures is the pseudo-static analysis, which is based on limit equilibrium methods. In the 1920s, Mononobe and Okabe developed a method to estimate the lateral earth pressures acting on retaining structures during earthquakes (Kramer, 2003). Hence in this paper, pseudo-static method is adopted for the stability analysis of reinforced soil retaining structures. Using limit equilibrium method and assuming a planar failure surface, Saran et al. (1992) developed non-dimensional design charts for computing the resultant active earth pressure for a reinforced soil retaining wall carrying a uniform surcharge load. Many studies are reported in the literature regarding the seismic stability analysis of geosynthetics-reinforced soil structures based on a pseudo-static limit equilibrium analysis (Bathurst & Cai 1995, Ling et al. 1997; Ling & Leshchinsky 1998; Ismeik & Guler 1998; Ausilio et al. 2000, Basha & Basudhar 2005). Nouri et al. (2008) presented an approach for the design of reinforced soil structures for the internal stability based on limit equilibrium and pseudo-static methods of analysis, by employing a log-spiral failure mechanism and horizontal slices.

The focus of the paper is to examine the internal stability analysis of RSS when subjected to horizontal and vertical earthquake loading ensuring adequate safety factor against various types of possible failure modes viz. tension and pullout failure of all layers along the depth of wall. The efforts have been made to derive an analytical solution for the internal seismic stability of RSS considering log-spiral failure surface (Figure 1). Uniformly distributed surcharge load placed on the backfill soil is also considered in the analysis. In addition to the inertial forces due to backfill soil, the inertial force due to surcharge load is also calculated using the equivalent surcharge height method.

2 INTERNAL SEISMIC STABILITY ANALYSIS

Let us consider a RSS of height, H supporting horizontal cohesionless backfill and the developing failure surface can be represented by a logarithmic spiral. Logarithmic spiral portion of the failure surface (GH_1) is governed by height of the reinforced soil slope (EG) and the location of centre of the logarithmic spiral arc (A). AH_1 and AG are respectively the initial and final radius of the logarithmic spiral passing through the centre 'A'. Thus the location of the center of the log-spiral

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(A) can be accurately defined by the subtended angle 'θ_1' and 'θ_2' (Figure 1) where in the various terms are defined as follows: r_0 = initial radius of the log-spiral wedge (AH_1G), r_1 = final radius of the log-spiral wedge (AH_1G), θ_1 = subtended angle of log-spiral wedge (AH_1G), θ_2 = the angle of the initial radius of the log-spiral wedge (AH_1) with the horizontal, F = resultant force acting along the radial line of the logarithmic spiral, W_{SH_1G} = weight of the log-spiral wedge SH_1G, γ = unit weight of the backfill soil, ϕ = friction angle of the backfill soil, k_h = horizontal seismic acceleration coefficient, and k_v = vertical seismic acceleration coefficient.

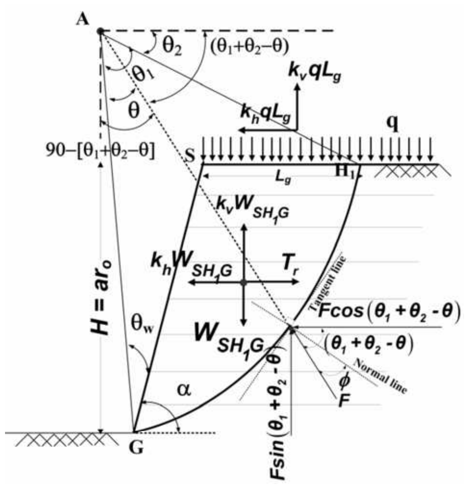

Figure 1. Log-spiral failure mechanism for internal stability

The reinforcement force (T_r) under seismic conditions can be obtained by resolving the forces acting on the wedge, SH_1G horizontally and vertically (Figure 1). An expression for the reinforcement force (T_r) under seismic conditions can be obtained as follows:

$$T_r = \left[k_h + (1-k_v)\cot(\theta_1/2+\theta_2)\right]\left(W_{SH_1G} + q\left(br_0 - H\cot\alpha\right)\right) \quad (1)$$

In order to locate the critical log-spiral failure surface to detect the maximum of force of the reinforcement required to ensure equilibrium (T_{or}), Improved Nelder-Mead simplex method is adopted in the study which can be stated as follows:

$$\text{Find } \theta_1 \text{ and } \theta_2 \text{ which } \begin{cases} \text{maximize} & T_r \\ \text{subjected to} & \begin{cases} 0^o < \theta_1 < 90^o \\ 0^o < \theta_2 < 90^o \end{cases} \end{cases} \quad (2)$$

2.1 *Estimation of factors of safety*

The design for internal seismic stability should be carried out such that there is an adequate margin of safety against the internal failure modes of RSS during the design life of the structure. Consideration should be given to local instability relating to rupture and pullout of the individual layers of reinforcement.

2.1.1 *Tension failure of reinforcement*

When the stability against tension failure the reinforcement layer under consideration is assigned as the factor of safety, the ultimate tensile strength of the reinforcement layer under consideration (T_u) should be more than the maximum load in the soil reinforcement under consideration ($T_{i\max}$). The factor of safety against tension failure is given by,

$$FS_t = \frac{T_u}{T_{i\max}} \quad (3)$$

where $T_{i\max}$ based on vertical spacing ($S_v = H/n$) and horizontal spacing ($S_h = 1.0m$) can be calculated as

$$T_{i\max} = (z\gamma + q)K(S_v \times S_h) \quad (4)$$

where, K = reinforcement force coefficient which can be computed by dividing the optimum force of the reinforcement (T_{or}) with $0.5\gamma H^2$ as shown below:

$$K = \frac{T_{or}}{0.5\gamma H^2} \quad (5)$$

n = number of layers of the reinforcement and z is the depth of reinforcement layer under consideration from the top of slope.

2.1.2 *Pullout failure of reinforcement*

When the stability against pullout failure of the reinforcement is assigned as the factor of safety, the available resisting force (P_{ri}) on the embedded reinforcement length of layer under consideration should be more than the maximum load in the soil reinforcement under consideration ($T_{i\max}$). The factor of safety against pullout failure is given by,

$$FS_{po} = \frac{P_{ri}}{T_{i\max}} \quad (6)$$

where $P_{ri} = 2\sigma_{vi}L_{ei}\tan\delta_i$ (Figure 2)and L_{ei} = the embedded reinforcement length of each layer beyond the failure surface, $\sigma_{vi} = z\gamma + q$ = effective vertical stress acting on the embedded reinforcement length of layer under consideration and δ_i = soil-reinforcement interface friction angle.

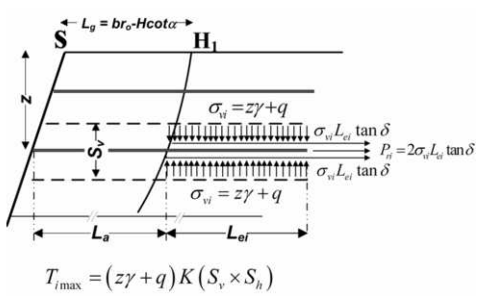

Figure 2. Pullout length of reinforcement (L_{ei})

Total length of the reinforcement (L) can be written as

$$L = L_a + L_{ei} \quad (18)$$

where L_a is the active length of the reinforcement

3 RESULTS AND DISCUSSIONS

The stability of RSS can be predicted by substituting the practically acceptable values of the different parameters involved in the factors of safety expressions derived in the above sections. Vertical seismic acceleration (k_v) acting in the downward direction is considered as positive in the present study. The values and ranges of these parameters for the present study are shown in Table 1. Effects friction angle of reinforced backfill and horizontal seismic acceleration on the tension, pullout stability modes are presented in Figures 3 and 4.

Table 3. Range of parameters considered in the analysis.

Parameter	Range
γ	18 kN/m^3
ϕ	30^o to 40^o
δ_i/ϕ	1.0
δ_b	2/3 ϕ_b
c	10 kN/m^2
k_h	0 – 0.3
k_v/k_h	0.5
$Q=2q/(\gamma H)$	0.0
T_u	45 kN/m
n	10 to 26 layers
α	90^o

3.1 *Influence of ϕ and k_h on factor of safety against tension failure (FS_t).*

Figures 3(a) and 3(b) show the variation of factor of safety against tension failure (FS_t) of all the layers of reinforcement along the depth of wall for ϕ = 30^o, 35^o and 40^o and k_h = 0.0, 0.1, 0.2 and 0.3 respectively for typical values δ_i/ϕ = 1.0, T_u = 45 kN/m, Q = 0.0, k_v/k_h = 0.5 and total length of the reinforcement (L/H) = 1.0. For the top layers where axial tensile force in the geosynthetic layer is significantly less, a very high value (more than 18) of factor of safety is observed. It can be seen from the figure that the bottom layers of reinforcement from the top of wall are more critical to the tension mode of failure due to overburden pressure and generally have lower factor of safety values. For this purpose, the number of reinforcement layers (n) required to maintain the factor of safety against tension failure (FS_t) $\geq$ 1.0 for all layers of reinforcement is computed and presented in Figure 3(a) and 3(b). As an illustration from Figure 3(b), for k_h = 0.0, 12 layers of geosynthetic reinforcement should be provided in the 9m height of wall with a vertical spacing (S_v) of 0.75m (i.e. $H/n=9/12$) to obtain FS_t $\geq$ 1.0 for all layers.

It can also be seen from Figures 3(a) and 3(b) that number of reinforcement layers (n) required to maintain desired stability (FS_t $\geq$ 1.0) should be increased significantly with reduction in ϕ value from 40^o to 30^o and increase in k_h value from 0.0 to 0.30. For example, for ϕ = 40^o, 12 layers of reinforcement should be provided in the 9m height of wall with a vertical spacing (S_v) of 0.75m ($=9/12$) to obtain FS_t $\geq$ 1.0 for all layers. Similarly n = 14 layers for ϕ = 35^o, n = 18 layers for ϕ = 30^o should be accommodated in the 9m height of wall to avoid the tension failure of all layers of reinforcement. This behavior is attributed to decreasing shear resistance in the reinforced backfill soil. It can also be observed that for a constant values of ϕ and k_h, the factor of safety (FS_t) significantly reduces as the depth of the layer increases. For letting a constant value of k_h = 0.2, factor of safety (FS_t) decreases significantly from 20 to 1.2 when depth increases from the topmost to bottommost layer. Similar observations can also be made for the influence of friction angle. This important observation helps in judicious selection of the number of reinforcement layers as the increase in horizontal seismic accelerations adversely affects the wall stability.

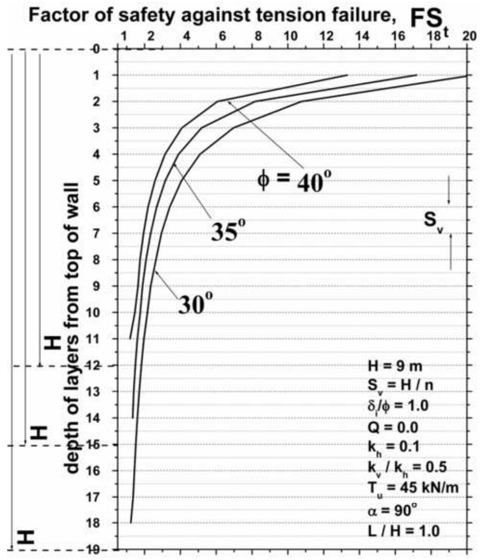

Figure 3(a). Effect of ϕ on factor of safety against tension failure (FS_t).

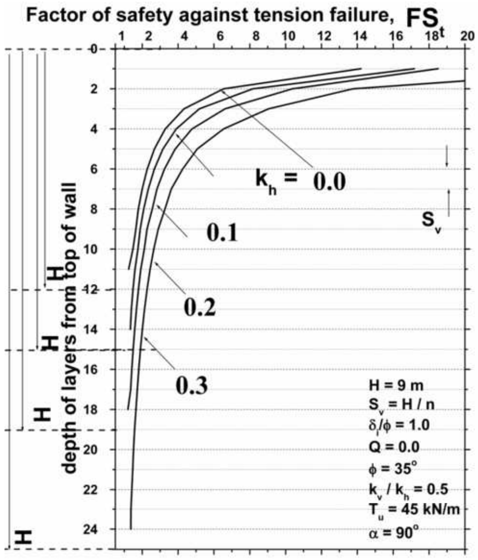

Figure 3(b). Effect of k_h on factor of safety against tension failure (FS_t).

3.2 *Influence of k_h on factor of safety, FS_{po}*

The pullout length of the reinforcement (L_{ei}) gives rise to the stability against pullout failure of RSS. Figure 4 shows the variation of factor of safety against pullout failure (FS_{po}) of all the layers of reinforcement along the depth of wall for k_h = 0.0 - 0.3 for typical value of total length of reinforcement (L/H) = 1.0 (uniform for all layers). It can be seen from Figure 4 that the upper layers of reinforcement from the top of wall are more critical to the pullout mode of failure and wall should have adequate pullout length to maintain the factor of safety against pullout failure (FS_{po}). Results presented in Figure 4 demonstrate that for a constant value of k_h and total length of the reinforcement (L/H) = 1.0, i.e. L = 9m, the factor of safety against pullout failure (FS_{po}) significantly increases as the depth of the layer increases. For letting a constant value of k_h = 0.1 (Figure 4), FS_{po} increases significantly from 1.5 to 15.5 when depth increases from the topmost layer (1st layer) to bottommost layer (13th layer). This is because, the available pullout length ($L_{ei} = L - L_a$) increases as depth increases from the uppermost layer (1st layer) to the bottommost layer (13th layer). An observation that can also be made is that the available pullout length (L_{ei}/H) reduces as the k_h value increases from 0.0 to 0.3.

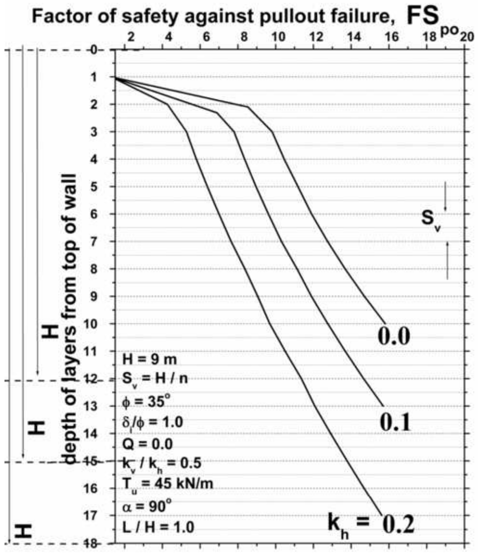

Figure 4. Influence of k_h on factor of safety (FS_{po}) along the depth of wall, computed for L/H = 1.0.

4 CONCLUSIONS

The present study gives a simple design methodology for analyzing the reinforced soil structures subjected to earthquake forces for the internal stability. A closed-form solution is obtained to estimate seismic active earth pressure acting on reinforced earth structures and the factor of safety against different modes of stability failure. From the present investigations, the following conclusions can be made:

1. The proposed method provides a closed form solution for the active earth pressure acting on reinforced soil structures under earthquake loading ensuring internal.
2. The design parameters such as horizontal seismic acceleration, number of reinforcement layers (n), total length of the reinforcement (L), angle of shearing resistance (ϕ), have significant influence on the internal seismic stability of the wall.
3. It is noted that the bottom layers of reinforcement from the top of wall are more critical to the tension mode of failure due to axial tensile force in the geosynthetic layer is significantly high and have lower factor of safety values.
4. It is observed that the upper layers of reinforcement from the top of wall are more critical to the pullout mode of failure and wall should have adequate pullout length of the reinforcement to maintain the targeted value of factor of safety in pullout mode.

5 APPENDIX

From Figure 1, the following equations can be derived.

$$H = KC = EG = r_1 \sin(\theta_1 + \theta_2) - r_o \sin\theta_2 = a r_0 \quad \text{(A1)}$$

$$a = \left[e^{\theta_1 \tan\phi} \sin(\theta_1 + \theta_2) - \sin\theta_2 \right] \quad \text{(A2)}$$

$$BC = DG = r_1 \sin(\theta_1 + \theta_2) - r_1 \sin(\theta_2 + \theta) \quad \text{(A3)}$$

$$GJ = F_1 I = EH_1 = L_g = r_o \cos\theta_2 - r_1 \cos(\theta_1 + \theta_2) = b r_o \quad \text{(A4)}$$

$$b = \left[\cos\theta_2 - e^{\theta_1 \tan\phi} \cos(\theta_1 + \theta_2) \right] \quad \text{(A5)}$$

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