A Proposal for Seismic Design of Geosynthetic Reinforced Steep Slopes

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SUMMARY:
The main objective of the internal design of geosynthetic reinforced structures is the definition of the minimum required strength and reinforcement length of the geosynthetic layers. The need of reinforcement may be represented by required strengths along the structure height, those equivalents to an earth pressure distribution. Most design suggestions available in the literature for seismic design conditions assume a linear distribution of reinforcement force for the quantification of the earth pressure coefficient. In this work is suggested the assumption of a uniform dynamic earth pressures distribution along the structure height to estimate the earth pressure coefficient. The effect of seismic loading is reported and some design charts are presented.

Keywords: Reinforced soil, Geosynthetics, Seismic loading, Internal design, Earth pressure coefficients

1. INTRODUCTION

The main objective of the internal design of geosynthetic reinforced soil retaining walls and slopes is the definition of the required strength, and a minimum length of the geosynthetic layers. For a given vertical spacing between reinforcement layers, the internal stability analysis is used to determine a minimum value of the geosynthetic strength. The joint definition of the required strength and the vertical spacing between geosynthetic layers can also be achieved. Some methods have been proposed in the last decades and three different approaches can be distinguished. The first approach, limited to reinforced soil slopes, is an extension of the classical limit equilibrium slope stability methods (methods of slices) in which the reinforcement forces are included in the analysis (Wright & Duncan, 1991; FHWA, 2010). The second approach is based on considerations of limit equilibrium, such as two-part wedge or logarithmic spiral analyses (Schmertmann et al., 1987; Leshchinsky & Boedeker, 1989; Jewell, 1989). The third is a kinematic approach of limit analysis and can be performed as a continuum approach, where the soil and the reinforcements are homogenized, or a structural approach, in which the soil and the reinforcements are considered as two separate structural components (Sawicki & Lesniewska, 1989; Michalowski, 1997; Ausilio et al., 2000). This paper refers only to the second approach.

Limit equilibrium analyses are used to calculate the horizontal force due to lateral earth pressures which the reinforcement layers should support to reach the structure equilibrium. The failure surface associated with the maximum value of this horizontal force defines the critical surface. The reinforcement layers should extend beyond this critical failure surface and have design strength sufficient to maintain the equilibrium.

The horizontal force to be resisted by the reinforcement layers is equal to the resultant of the assumed earth pressure distribution. Thus, it is possible to define an earth pressure coefficient (or required coefficient) to be used to estimate the minimum value of the reinforcement tensile strength. This paper presents results from a developed computer program, based on limit equilibrium calculations, able to calculate required coefficients for static and seismic loading conditions, assuming distinct failure

mechanisms and earth pressure distributions (Vieira, 2008). Although not presented in this paper, the program also defines the minimum reinforcement length.

The effect of the seismic loading on the required geosynthetic strength is based on pseudo-static analyses. These analyses are, frequently, considered as conservative, since transitory earthquake acceleration is assumed to act permanently on the structure. A proper selection of the seismic coefficients used in the pseudo-static analyses shall compensate for the conservatism of these methods. On the other hand, this conservatism may counterbalance the acceleration amplification when not implicitly considered in the design.

2. DESIGN APPROACH

The design of geosynthetic reinforced soil structures are usually based on limit equilibrium considerations. The two-part wedge and the logarithmic spiral failure mechanisms are limit equilibrium approaches suitable for the design of these structures (Vieira et al., 2011). The present paper includes only results obtained with logarithmic spiral failure mechanisms. The logarithmic spiral failure mechanism has been used to calculate the horizontal required force needed to reach the equilibrium either under static (Jewell, 1989; Leshchinsky et al., 1995; Michalowski, 1997) or seismic loading conditions (Ling et al., 1997; Michalowski, 1998; Vieira et al., 2011).

Fig. 1 presents a logarithmic spiral failure mechanism for a general loading. The horizontal and vertical seismic inertial forces, acting at the centre of gravity of the potential failure soil mass, are considered through the horizontal and vertical seismic coefficients, k_h and k_v, expressed as fractions of the gravitational constant, g. The failure surface is described by the logarithmic spiral equation:

$$r = r_0 \, e^{-\theta \tan \phi} \tag{1}$$

where θ is the angle between the radius, r, and the vertical line that passes through the pole, P, r_0 is the radius of logarithmic spiral surface when $\theta = 0$ (Fig. 1) and ϕ is the soil internal friction angle. For a known failure surface, the force required for equilibrium, P_{ae}, can be evaluated by the moments equilibrium equation about the pole, given by (Fig. 1):

$$\sum M_P = \left(1 \pm k_v\right) W \left(X_C - X_P\right) + k_h W \left(Y_P - Y_C\right) - P_{ae} \left(Y_P - Y_{ae}\right) = 0 \tag{2}$$

As the resultant force of the normal and shear stresses over the logarithmic spiral surface, N and S in Fig. 1, passes through the pole, the corresponding moment is null.

The critical failure mechanism corresponds to the maximum value of the force P_{ae} that satisfies Eqn. 2. The application point of the force P_{ae}, Y_{ae}, has influence on its magnitude. Two assumptions were considered: earth pressures increasing linearly with depth, which means $Y_{ae} = H/3$, and earth pressures uniformly distributed along the structure height, $Y_{ae} = H/2$. In practice, particularly when the structure height is less than 6-7 meters, it is common to adopt evenly spaced reinforcement layers of equal strength. That means that the available reinforcement strength is uniform trough the structure height and equal to the maximum value of required reinforcement strength, usually calculated assuming a linear distribution of reinforcement force. The design of these structures is conservative and could be more economical if the required reinforcement force was determined assuming a uniform distribution of earth pressures. These situations were one of the reasons for the previously mentioned second assumption. Furthermore, reduced-scale tests performed by El-Emam (2003) showed that the distribution of the static lateral earth force was more uniform than the triangular distribution assumed in current limit equilibrium design methods.

A computer code was developed to find the critical failure geometry and therefore to achieve the maximum required force, P_{ae}, for the structure equilibrium. The need of reinforcement may be

represented by required strengths along the structure height, those equivalents to an earth pressure distribution. Terzaghi (1943) demonstrated, by the similarity between critical potential failure surfaces, that the distribution of the earth pressure on the back of a wall increases like a hydrostatic pressure in simple proportion to depth. Based on this, Jewell (1989) states that the magnitude of the maximum required reinforcement force for equilibrium increases with the square of the slope height.
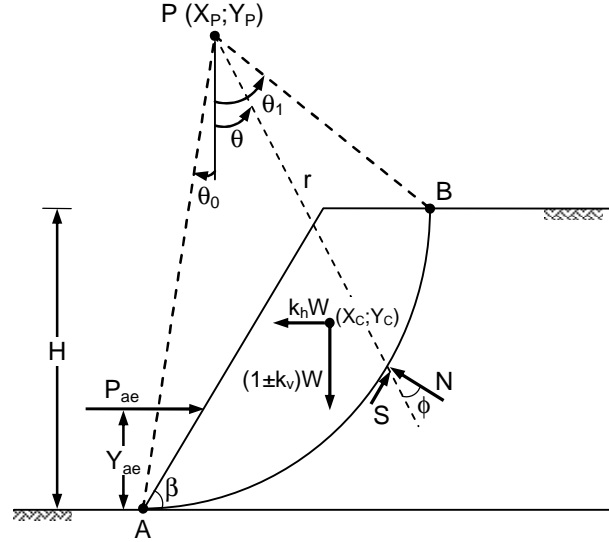

Figure 1. Logarithmic spiral failure mechanism

Most design suggestions available for seismic conditions (Ling et al., 1997; Ausilio et al., 2000; Nouri et al., 2008; Vieira et al., 2011) assume the linear (increasing with depth) distribution of reinforcement force. Assuming that the earth pressures (or reinforcement required strengths) increase linearly with depth, the equivalent earth pressure coefficient, K_{req}, for a structure with height, H, constructed with a backfill of unit weight, γ can be calculated as:

$$K_{req} = \frac{2P_{ae}}{\gamma H^2} \tag{3}$$

Note that, when vertical seismic acceleration is considered, the equivalent earth pressure coefficient, K_{req}, is not equal to the Mononobe-Okabe earth pressure coefficient (Okabe, 1924; Mononobe & Matsuo, 1929), K_{ae}, even for a vertical face structure. For the particular case of structures with vertical face, the critical failure surface degenerates to a single wedge surface, coincident with the one assumed by the Mononobe-Okabe approach, but the equivalent earth pressure coefficient, K_{req}, is equal to:

$$K_{req} = (1 \pm k_v) K_{ae} \tag{4}$$

where k_v represents the vertical seismic coefficient.

Assuming that earth pressures are uniform along the structure height, H, the equivalent earth pressure coefficient can be calculated as:

$$K_{req}^{uniform} = \frac{P_{ae}}{\gamma H^2} \tag{5}$$

Note that the earth pressure coefficients estimated by Eqn. 3 and Eqn. 5 are not directly comparable. For the same value of the required force for equilibrium, P_{ae}, the earth pressure coefficient calculated

by the Eqn. 5 is half of the value obtained with Eqn. 3.

In the design of retaining structures, it is usual the decomposition of the dynamic active earth force, P_{ae}, into two components representing the static earth force, P_a, and the incremental dynamic earth force, ΔP_{ae} (Seed & Whitman, 1970).

The position of the dynamic earth force has been shown to be variable and to depend on the magnitude of ground acceleration (Bathurst and Cai, 1995). For retaining walls, a general range for the point of application of the incremental dynamic force, ΔP_{ae}, has been reported to be 0.4H to 0.7H above the toe of the wall (Seed & Whitman, 1970). Based on experience with conventional gravity retaining walls, Bathurst and Cai (1995) suggested the dynamic earth pressure distribution illustrated in Fig. 2 for geosynthetic reinforced walls In this distribution the distance of the dynamic load increment above the toe of the wall normalized by the wall height (η in Fig. 2b) is equal to 0.6. For the dynamic earth pressure distribution proposed by Bathurst and Cai (1995), the point of application of P_{ae} (m in Fig. 2c) is limited to the range 1/3H to 0.6H.
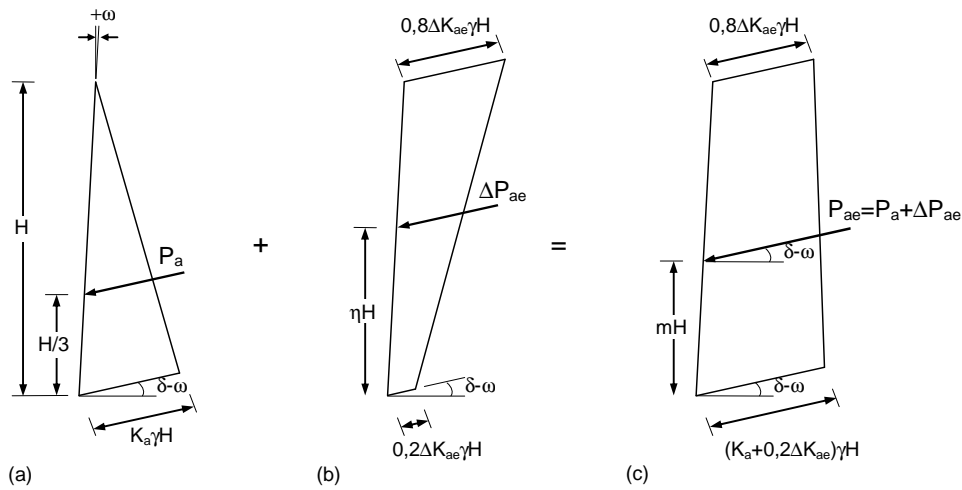

Figure 2. Dynamic earth pressure distribution due to soil self-weight proposed by Bathurst & Cai (1995): a) static component; b) dynamic increment; c) total pressure distribution.

As mentioned before, most design suggestions available for seismic design conditions assume a linear distribution of reinforcement force for the quantification of the earth pressure coefficient, used to estimated the resultant force that should be supported by the reinforcement layers (equal to P_{ae}). Bearing in mind the earth pressure distribution illustrated in Fig. 2(c) together with the evidence that in practice, particularly when the structure height is less than 6-7 meters, it is common to adopt evenly spaced reinforcement layers of equal tensile strength, it is suggested the assumption of a uniform earth pressures distribution along the structure height.

3. PARAMETRIC STUDY

3.1. General Aspects

In this parametric study, a purely frictional backfill material, with internal friction angle in the range of 20º-45º, slope inclination angles between 30º and 90º and zero pore water pressure ($r_u = 0$) were assumed. It was admitted that the structures rest on a competent and level foundation and the backfill surface is horizontal.

The effect of the seismic action on the required reinforcement tensile forces will be presented, in sequence, considering a logarithmic spiral failure mechanism and a uniform distribution of the earth pressures ($Y_{ae} = H/2 -$ Fig. 1).

3.2. Effect of Horizontal Seismic Action

Fig. 3 illustrates the effect, on the earth pressure coefficient, of the horizontal seismic coefficient, k_h, for reinforced structures with slope face inclined at 60º and 80º. As expected, the greater the horizontal seismic coefficient the larger the required reinforcement. Comparing the values of K_{req} for seismic loading with those obtained for static conditions ($k_h = 0$), the increase induced by the seismic action is higher for the flatter slope. For instance, assuming the backfill friction angle, ϕ, equal to 30º, the increment of K_{req} induced by a horizontal seismic loading with $k_h = 0.3$ is, approximately, 125% and 71% for $\beta = 60º$ and $\beta = 80º$, respectively.
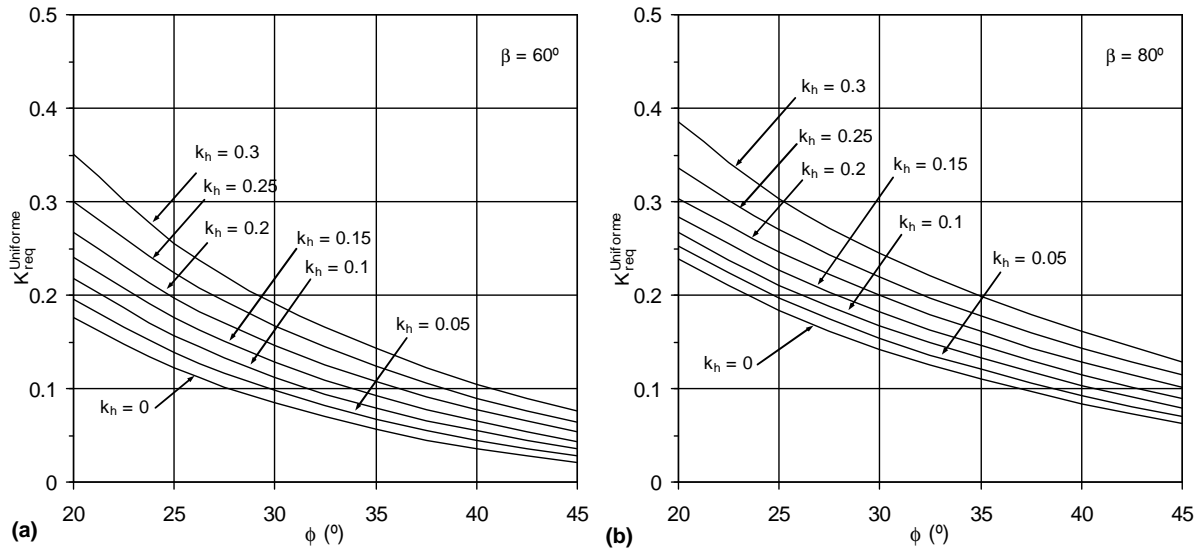

Figure 3. Effect of the horizontal seismic coefficient, k_h, on the earth pressure coefficient: a) $\beta = 60º$; b) $\beta = 80º$.

The ratio between the earth pressure coefficient for seismic actions, K_{req}, and the earth pressure coefficient for static conditions, $K_{req}^{k_h=0}$, was used to illustrate the effect of the horizontal seismic loading on the required reinforcement tensile force. The values of the ratio $K_{req}/K_{req}^{k_h=0}$ for structures with face inclined in the range 40º-90º, backfill material with internal friction angle between 20º and 45º and horizontal seismic coefficient equal to 0.20 or 0.30, are represented in Fig. 4. Fig. 4 shows that, especially for structures with slope face lower than 70º, the ratio $K_{req}/K_{req}^{k_h=0}$ grows with the backfill internal friction angle, ϕ. As illustrated in Fig. 3, either for static ($k_h = 0$) or seismic conditions, the greater the backfill internal friction angle, the lower the requirement of tensile forces provided by the reinforcement. Notwithstanding, comparatively to the reinforcement needed to structure stability in static conditions, the increase of required tensile forces induced by the seismic action (expressed by the ratio $K_{req}/K_{req}^{k_h=0}$) grows with the backfill internal friction angle.

It should be noted that the increases of reinforcement tensile force induced by the seismic action, presented in Fig. 3 and Fig. 4, do not represent the real increases of required reinforcement strength obtained by designing for seismic loading conditions. The partial safety factors considered in the seismic design are lower than those used in static conditions.

Fig. 5 presents the effect of the horizontal seismic coefficient on the potential failure surfaces for structures with slope angle, β, equal to 60º (Fig. 5a) and 80º (Fig. 5b), considering the backfill internal friction angle, ϕ, equal to 30º. As expected, the volume of soil potentially in failure increases with the horizontal seismic coefficient, although, the rate of increase of soil volume is greater than that of the horizontal seismic coefficient, k_h. For the steeper slope, with the increase of the horizontal seismic coefficient, the failure surface tends to become planar.
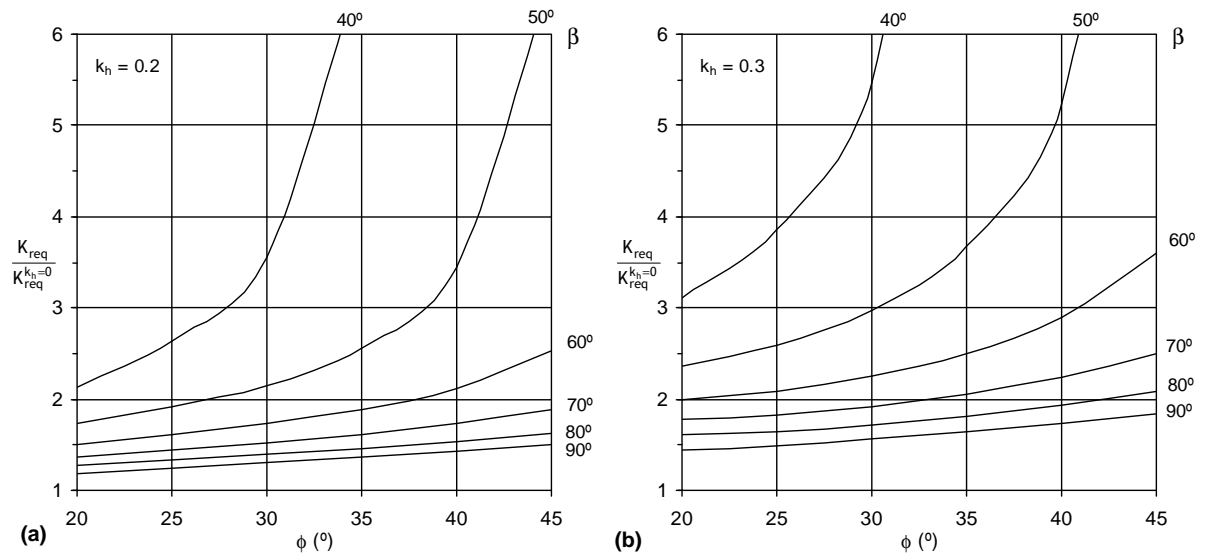
Figure 4. Increase in required reinforcement strength induced by the horizontal seismic action: a) $k_h = 0.20$; b) $k_h = 0.30$.
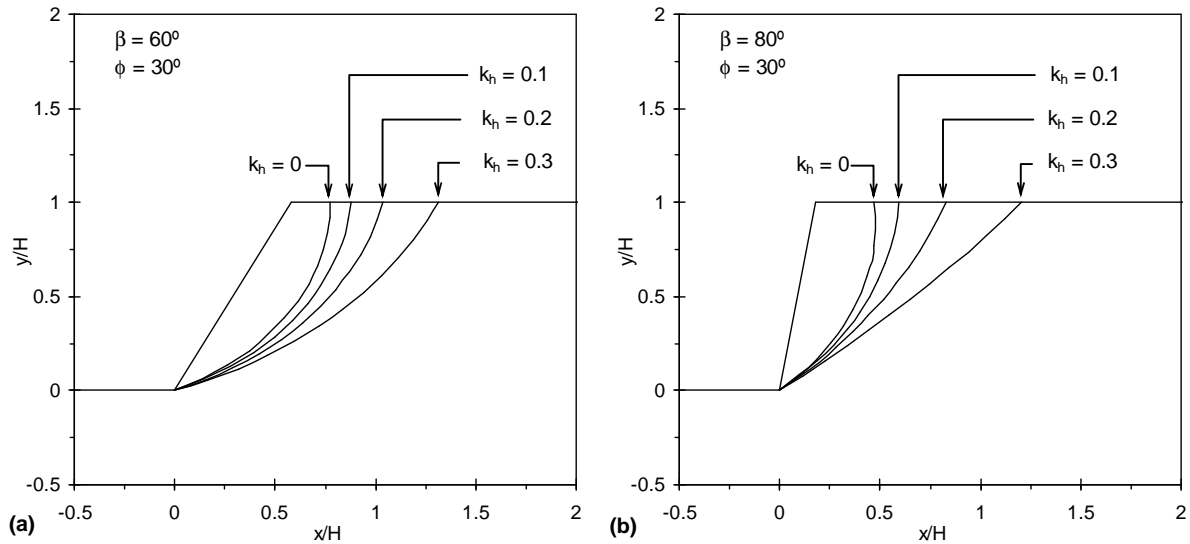

Figure 5. Effect of k_h on potential failure surfaces: a) $\beta = 60°$ e $\phi = 30°$; b) $\beta = 80°$ e $\phi = 30°$.

Fig. 6 summarizes earth pressure coefficients for horizontal seismic coefficients of 0 (static loading), 0.10, 0.20 and 0.30, assuming slope angles in the range 30°-90° and backfill material with an internal friction angle between 20° and 45°. It should be reminded that the coefficients presented in Fig. 6 were obtained assuming the reinforcement tensile forces uniform through the structure height.

3.3. Effect of Vertical Seismic Action

The effect of the vertical component of seismic action is not explicitly considered in the generality of the codes (AASHTO, 2007; FHWA, 2010). For some of them, the consideration of a vertical seismic coefficient infers that the maximum values of the vertical and the horizontal components of the seismic action occur simultaneously. A parametric analysis is presented to clarify the real effects, on the required geosynthetic tensile forces, of neglecting the vertical seismic action. In this parametric analysis, the vertical seismic loading is considered through a vertical seismic coefficient, k_v, defined as a function of the horizontal seismic coefficient, k_h. For the ratio k_v/k_h, values of +1.0, +0.5, -0.5 and -1.0 were considered. The sign of k_v is positive when the corresponding vertical inertial forces act downwards.
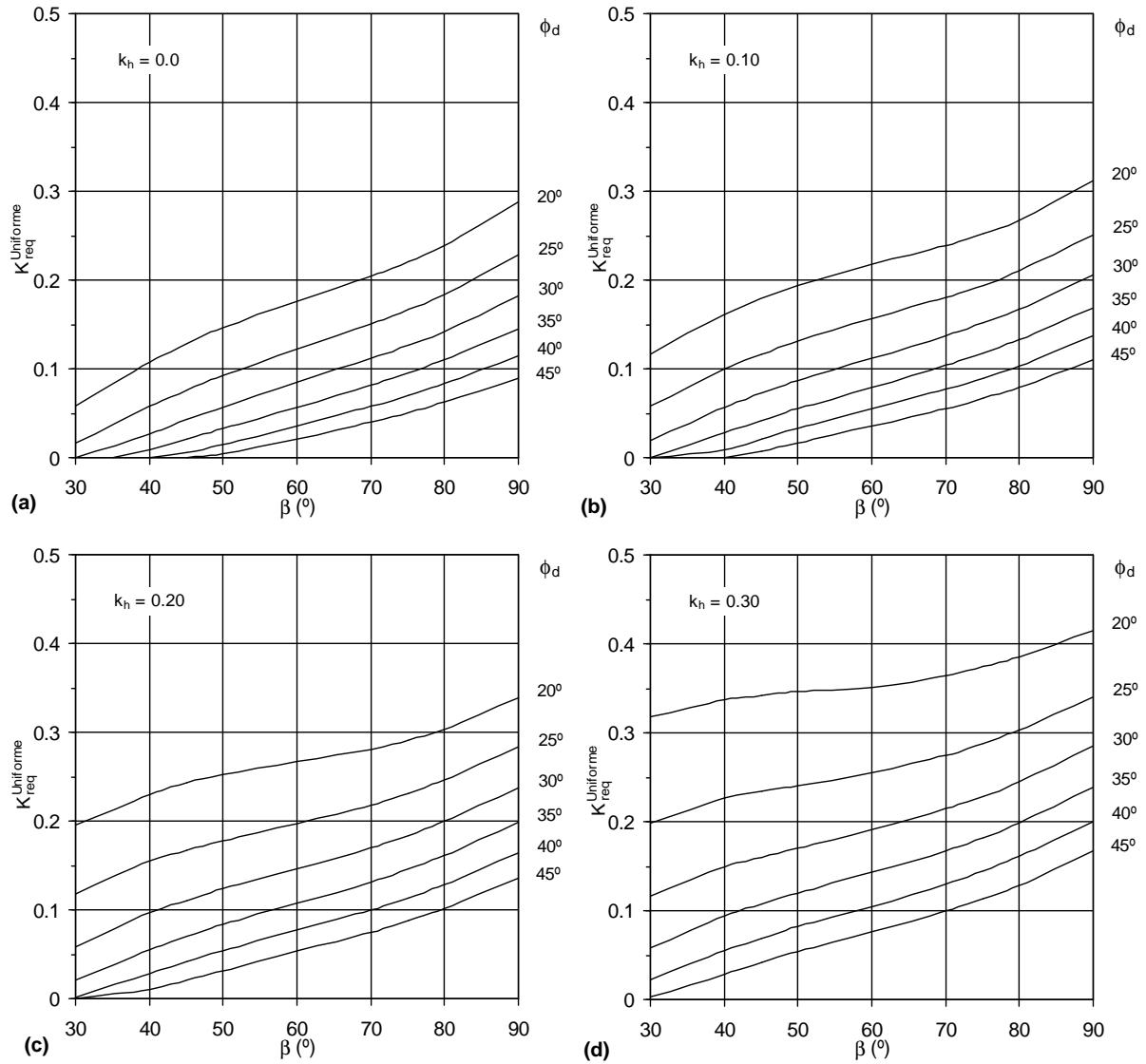
Figure 6. Effect of the horizontal seismic coefficient on the required coefficient K_{req} (uniform distribution): a) $k_h = 0$ (static loading); b) $k_h = 0.10$; c) $k_h = 0.20$; d) $k_h = 0.30$

Fig. 7 shows the effect of k_v on the required reinforcement strength, expressed by the earth pressure coefficient, K_{req}, for slope angles, β, equal to 60° and 80°, considering $k_h = 0.2$. For comparison purposes, it is also represented the earth pressure coefficients for static loading. When vertical inertial forces act downwards ($k_v > 0$) the required reinforcement strength increases (comparatively to the strength demanded when $k_v = 0$). The effect of k_v seems also to increase with the slope angle, β, and to decrease with the backfill internal friction angle, ϕ. The values represented in Fig. 7 are also plotted in Fig. 8 as the ratio between the earth pressure coefficient considering the vertical seismic coefficient, K_{req}, and the earth pressure coefficient for $k_v = 0$, $K_{req}^{k_v=0}$. For the steeper slope (Fig. 8b), the effect of k_v is not particularly influenced by the backfill internal friction angle. For $\beta = 60°$, the effect of k_v seems to slightly decrease with the internal friction angle of the backfill material. For the two slope angles analysed ($\beta = 60°$ and $\beta = 80°$), Fig. 8 shows that the increase in the required reinforcement strength induced by the vertical seismic action is lower than 15%.
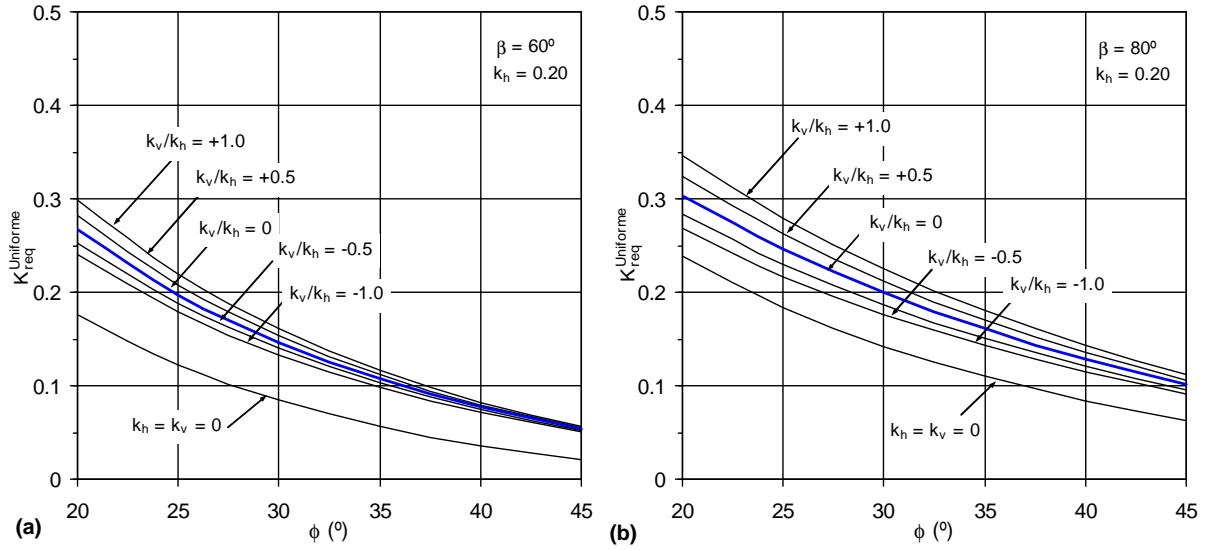
Figure 7. Effect of vertical seismic coefficient, k_v, on the earth pressure coefficient ($k_h = 0.2$):
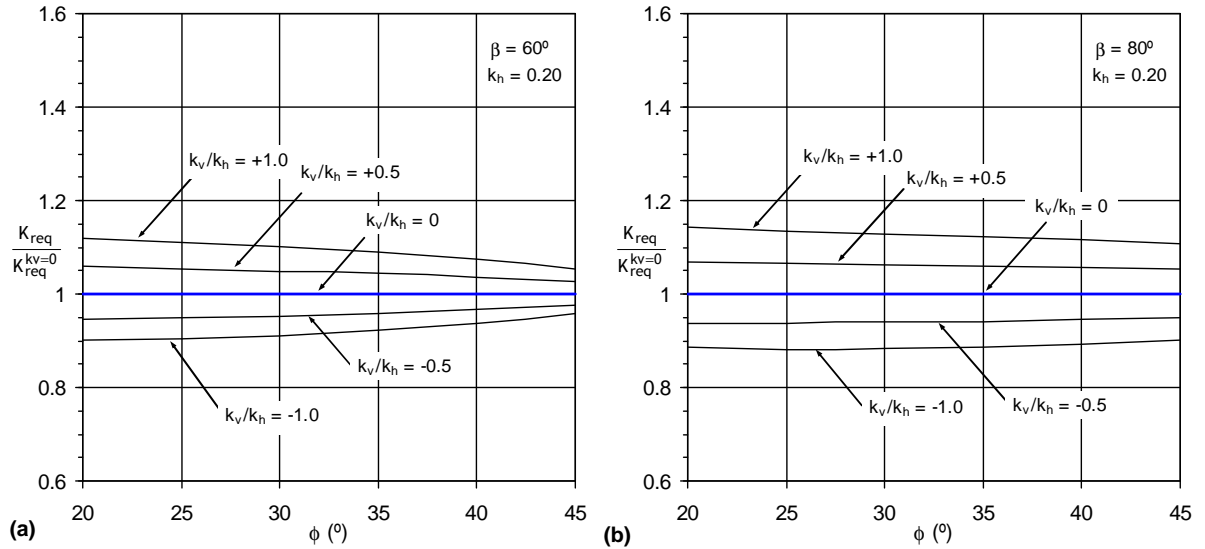a) $\beta = 60°$; b) $\beta = 80°$

Figure 8. Variation of required reinforcement strength due to the vertical seismic loading consideration
($k_h = 0.2$): a) $\beta = 60°$; b) $\beta = 80°$.

4. EFFECT OF ASSUMED TENSILE FORCE DISTRIBUTION

Most suggestions available for seismic design of geosynthetic reinforced structures assume the linear distribution of reinforcement force (increasing with depth) for the quantification of the required coefficient, K_{req}, used to calculate the reinforcement tensile forces. Notwithstanding there is a general consensus on a dynamic earth pressures distribution not triangular.

Fig. 9 compares the earth pressure coefficient, K_{req}, obtained with the linear increase, $Y_a = H/3$, and with a uniform distribution of required tensile strength along the structure height, $Y_a = H/2$. The values are presented as a function of the slope inclination, β, and the design value of backfill friction angle, ϕ_d.

As mentioned previously, for the same value of the force required for equilibrium, P_{ae}, the $K_{req}^{uniform}$ value, computed by the Eqn. 5, is half of the K_{req} value obtained with Eqn. 3. Thus, the earth

pressure coefficients presented in Fig.9 for uniformly distributed reinforcement are twice the value of $K_{req}^{uniform}$. From the analysis of Fig. 9 it can be concluded that the force required for equilibrium, P_{ae} (equal to the sum of reinforcement tensile forces) is greater when the uniform distribution of reinforcement is considered. This conclusion is obvious since the higher is the application point of P_{ae} the lower is the resistant moment about the logarithmic spiral pole. For the same failure surface, the reinforcement level needed to provide the same resistant moment to achieve the equilibrium is greater when P_{ae} is considered at middle of the structure height. The difference between the results tends to decrease with the soil internal friction angle.

A simplistic analysis of the Fig. 9 can lead to the conclusion that the uniform reinforcement distribution is more conservative than the triangular distribution (linear increase with depth), however, in practice, this conclusion may not be accurate. From the practical point of view, it is not realistic the design of a reinforcement system that gives rise to an available stress distribution increasing linearly with depth. In practice, it is common to divide the structure height in different parts, where evenly spaced reinforcement layers of equal strength is adopted. If only one geosynthetic type (with the same strength) will be used along the structure height, the use of the earth pressure coefficient estimated with a uniform required stress distribution is more economical.
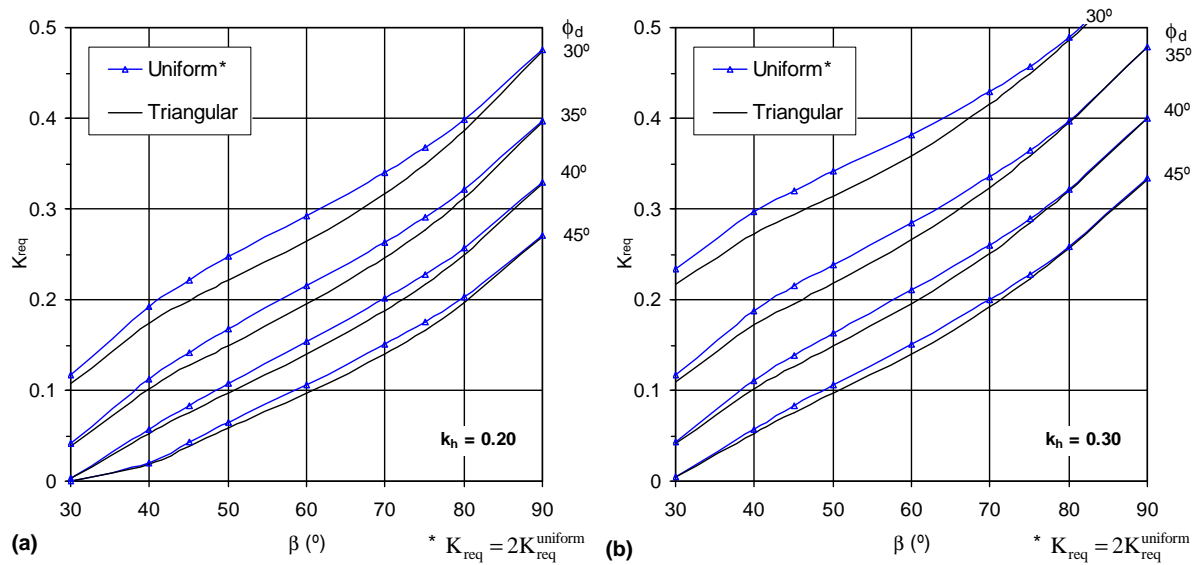

Figure 9. Effect of the reinforcement tensile forces distribution on the required strength for dynamic conditions (expressed by K_{req}): a) $k_h = 0.20$; b) $k_h = 0.30$

5. CONCLUSIONS

Some methods have been proposed in the last decades for the internal design of geosynthetic reinforced structures. The majority of them are based on limit equilibrium considerations, assuming bi-linear or logarithmic spiral failure surfaces. Based on these failure mechanisms, design charts have been presented by several authors most of them considering the linear increase of the dynamic earth pressure that should be supported by the reinforcement layers. In this work was suggested the assumption of a uniform dynamic earth pressures distribution along the structure height to estimate the earth pressure coefficient. The effect of seismic loading was reported and some design charts were presented.

Based on the parametric study presented in this paper, the following conclusions are drawn. The increase of required reinforcement strength induced by the seismic loading, comparatively to the required tensile forces in static conditions, grows with the backfill internal friction angle. The effects of the vertical component of seismic loading on the required reinforcement strength are not very significant.

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