

Seismic stability of reinforced-soil wall by pseudo-dynamic method

S. S. Nimbalkar¹, D. Choudhury² and J. N. Mandal³

¹Research Scholar, Department of Civil Engineering, Indian Institute of Technology Bombay, Powai, Mumbai 400 076, India, Telephone: +91 93267 12190, Telefax: +91 22 2576 7302, E-mail: sanjay_nimbalkar@iitb.ac.in

²Assistant Professor, Department of Civil Engineering, Indian Institute of Technology Bombay, Powai, Mumbai 400 076, India, Telephone: +91 22 2576 7335, Telefax: +91 22 2576 7302, E-mail: dc@civil.iitb.ac.in

³Professor, Department of Civil Engineering, Indian Institute of Technology Bombay, Powai, Mumbai 400 076, India, Telephone: +91 22 2576 7328, Telefax: +91 22 2576 7302, E-mail: cejnm@civil.iitb.ac.in

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ABSTRACT: Determination of the internal stability of reinforced soil walls under earthquake conditions is an important part of seismic design. The horizontal method of slices is used for determining internal stability or for tieback analysis of the reinforced soil wall. A pseudo-dynamic method is adopted in the present analysis, which considers the effect of phase difference in both the shear and primary waves travelling through the backfill due to seismic excitation. Reinforced soil walls with cohesionless backfill material have been considered in the analysis. Results are presented in graphical and tabular form to show the required tensile force and length of geosynthetic reinforcement to maintain the stability of the reinforced soil wall under seismic conditions. The effects of variation of parameters such as soil friction angle and horizontal and vertical seismic accelerations on the stability of the reinforced soil wall have been studied. With an increase of seismic accelerations in both the horizontal and vertical directions the stability of the reinforced soil wall decreases significantly, and thus greater strength and length of the geosynthetic reinforcement are required to maintain stability of the wall. The seismic vertical acceleration in an upward direction gives higher values of the required geosynthetic tensile strength, and the seismic vertical acceleration in the downward direction yields higher values of the length of geosynthetic reinforcement. Comparisons of the present results with available pseudo-static results are shown, and the limitations of the pseudo-static results are highlighted.

KEYWORDS: Geosynthetics, Earthquake, Reinforced soil, Retaining wall, Design, Internal stability, Modelling

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1. INTRODUCTION

Numerous methods are available for the seismic design of reinforced soil walls based on the pseudo-static method of analysis. The pioneering work on earthquake-induced lateral earth pressure under active and passive conditions acting on a retaining wall was reported by Okabe (1926) and Mononobe and Matsuo (1929). This pseudo-static approach, following Coulomb's earth pressure analysis under static conditions, is known as the Mononobe–Okabe method (Kramer 1996). The Mononobe–Okabe method can also be extended to the seismic analysis of reinforced soil walls. Ling *et al.* (1997) have proposed seismic design procedures based on pseudo-static limit equilibrium analysis, which considers only the horizontal seismic acceleration in the soil

medium. Ling and Leshchinsky (1998) have also studied the effect of pseudo-static vertical seismic acceleration on the seismic design of geosynthetic-reinforced soil walls. Shahgholi *et al.* (2001) used a horizontal slice method to analyse and investigate the seismic stability of reinforced soil walls. Kramer and Paulsen (2004) carried out seismic performance evaluation of reinforced slopes. El-Emam and Bathurst (2005) analysed the facing contribution to seismic response of reduced-scale reinforced soil walls subjected to a stepped amplitude sinusoidal base input acceleration. Huang and Wang (2005) presented a pseudo-static-based approach for evaluating the mechanical effects of facing components on the seismic displacement of reinforced soil walls backfilled with cohesionless soils.

However, all the above methods are based on the pseudo-static method of analysis, which considers the dynamic nature of earthquake loading in a very approximate way (Kramer 1996). These methods do not consider the effects of time and body waves travelling through the soil during the earthquake. Recently, Ling *et al.* (2005) conducted large-scale shaking table tests on modular-block reinforced soil-retaining walls. As proposed by Steedman and Zeng (1990), the phase difference due to finite shear wave propagation through a reinforced soil wall can be considered using a relatively new method, called the pseudo-dynamic method in this paper. Recently, Choudhury and Nimbalkar (2006) proposed a theory to compute the seismic active earth pressure by pseudo-dynamic method by considering both the shear and the primary waves propagating through the soil with variation in time by considering harmonic horizontal and vertical seismic accelerations. Choudhury and Nimbalkar (2005) also developed the theory to estimate the seismic passive earth pressure using the pseudo-dynamic approach. Here the authors considered both the shear and primary waves propagating through the soil with variation in time by considering harmonic horizontal and vertical seismic accelerations. However, the work by Choudhury and Nimbalkar (2005, 2006) was limited to un-reinforced rigid retaining walls. Hence in this paper an attempt has been made to investigate the stability of reinforced soil walls under seismic conditions by using the pseudo-dynamic method, which considers the time and phase change effects due to shear and primary waves propagating vertically through reinforced backfill, along with other seismic input parameters.

2. METHOD OF ANALYSIS

2.1. Background

The reinforcement in geosynthetic wall structures must be designed with adequate strength and length to resist different possible modes of failure, namely tieback, compound and pullout failure. The required geosynthetic tensile force is determined through tieback (internal stability) analysis, and the required geosynthetic length is obtained by considering potential rotational failures, extending both within and beyond the reinforced zone (compound failure) (Koerner 1994). For the seismic case, the internal stability analysis for reinforced soil walls can be carried out using pseudo-static forces as given by Kramer (1996).

In the present study, the pseudo-dynamic method considers the finite shear and primary wave velocities in the analysis. It is assumed that the shear modulus is constant with depth through the backfill, and that only the phase and not the amplitude of the acceleration is varying.

Consider a reinforced soil wall of height H with slope angle β , constructed of dry cohesionless free-draining soil as shown in Figure 1. Under earthquake condition, the shear wave velocity $V_s = (G/\rho)^{1/2}$ and primary wave velocity $V_p = [G(2 - 2\nu)/\rho(1 - 2\nu)]^{1/2}$ are assumed to act within the reinforced soil structure. For most geological materials, $V_p/V_s = 1.87$ (Das 1993) is considered. The

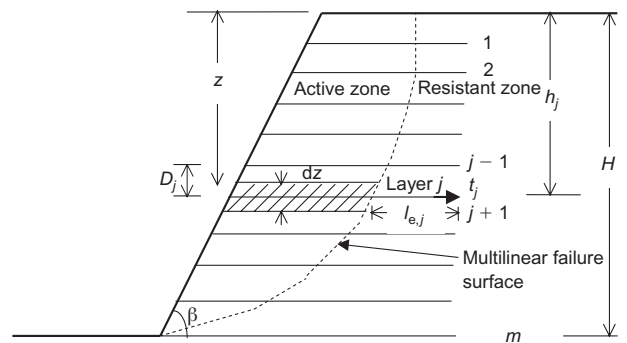


Figure 1. Configuration of model reinforced soil wall considered in the analysis

period of lateral shaking, $T = 2\pi/\omega = 4H/V_s$ (Kramer 1996), is considered in the analysis. In the present analysis, it is assumed that both the horizontal and vertical vibrations, with accelerations a_h and a_v , respectively, start at exactly the same time, and there is no phase shift between these two vibrations, thus giving the critical condition for design. The tensile force in the geosynthetic layer is considered to act horizontally, because its inclination has little effect on the results of analysis when cohesionless soils are used (Leshchinsky and Boedeker 1989; Wright and Duncan 1991). In the present method, the failure surface is assumed as multilinear, in which a number of failure planes are considered to identify the critical design criteria.

In the present method, the seismic accelerations acting on the reinforced-soil wall are considered as harmonic sinusoidal accelerations, which is one of the limitations of the pseudo-dynamic method as originally proposed by Steedman and Zeng (1990).

2.2. Calculation of pseudo-dynamic inertia forces

If the base is subjected to harmonic horizontal and vertical seismic accelerations of amplitudes a_h and a_v , the accelerations at depth z below the top of the wall can be expressed as

$$a_h(z, t) = a_h \sin \omega \left(t - \frac{H - z}{V_s} \right) \quad (1)$$

$$a_v(z, t) = a_v \sin \omega \left(t - \frac{H - z}{V_p} \right) \quad (2)$$

For a thin elemental slice of thickness dz at depth z , as shown in Figures 1 and 2, the mass of the elemental i th slice is given by

$$m_i(z) = \frac{\gamma}{g} \frac{H - z}{\tan \alpha_i} dz \quad (3)$$

The weight of the elemental i th slice is

$$W_i = \gamma dz \left(\frac{l_j + l_{j+1}}{2} \right) \quad (4)$$

The total horizontal inertia force q_{hi} acting on the i th slice can be expressed as

$$q_{hi} = m_i(z) \cdot a_h(z, t) \quad (5)$$

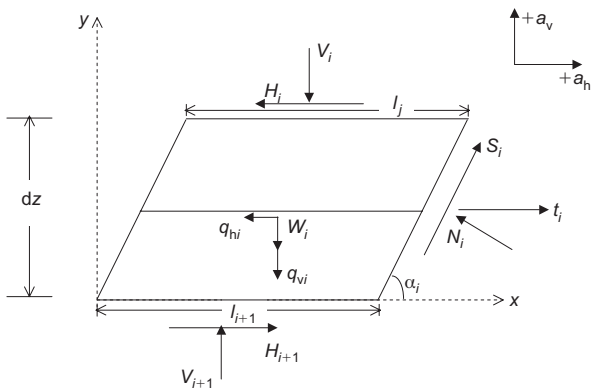


Figure 2. Forces acting on single horizontal elemental slice containing reinforcement

The total horizontal inertia force Q_h acting on the reinforced soil wall can be expressed as

$$Q_h = \int_0^H q_{hi} dz = \int_0^H m_i(z) \cdot a_h(z, t) dz$$

$$= \frac{\lambda \gamma k_h}{4\pi^2 \tan \alpha_i} [2\pi H \cos \omega \zeta + \lambda (\sin \omega \zeta - \sin \omega t)] \quad (6)$$

Again, the total vertical inertia force q_{vi} acting on the i th slice can be expressed as

$$q_{vi} = m_i(z) \cdot a_v(z, t) \quad (7)$$

The total vertical inertia force Q_v acting on the reinforced soil wall can be expressed as

$$Q_v = \int_0^H q_{vi} dz = \int_0^H m_i(z) \cdot a_v(z, t) dz$$

$$= \frac{\eta \gamma k_v}{4\pi^2 \tan \alpha_i} [2\pi H \cos \omega \psi + \eta (\sin \omega \psi - \sin \omega t)] \quad (8)$$

where $\lambda = TV_s$ is the wavelength of the vertically propagating shear wave, $\eta = TV_p$ is the wavelength of the vertically propagating primary wave, $\zeta = t - H/V_s$ and $\psi = t - H/V_p$. Only the critical directions of q_{hi} and q_{vi} acting on the elemental slice are considered, as shown in Figure 2: that is, the vertical seismic inertia force q_{vi} acting in the upward direction is considered in calculating the geosynthetic length required to resist tie-back, compound and pullout failure. Similarly, the vertical seismic inertia force q_{vi} acting in the downward direction is considered in determining the required geosynthetic tensile reinforcement (Ling and Leshchinsky 1998).

The special case of a rigid wedge is given, in the limit, as

$$\lim_{V_s \rightarrow \infty} (Q_h)_{\max} = \frac{\gamma H^2 a_h}{2g \tan \alpha} = \frac{a_h}{g} W = k_h W \quad (9)$$

$$\lim_{V_p \rightarrow \infty} (Q_v)_{\max} = \frac{\gamma H^2 a_v}{2g \tan \alpha} = \frac{a_v}{g} W = k_v W \quad (10)$$

which is equivalent to the pseudo-static forces assumed in the Mononobe–Okabe method without considering any phase change in the body waves travelling through the soil medium.

2.3. Tieback or internal stability analysis

The required tensile strength and minimum length of each geosynthetic reinforcement layer are determined from the pseudo-dynamic method of analysis using a multilinear failure surface. A horizontal method of slices is used in this analysis. The following assumptions are made in the present analysis.

- The vertical stress on an element in the soil mass is equal to the overburden pressure.
- The factor of safety (FS) is equal to the ratio of the available shear resistance to the required shear resistance along the failure surface.
- The factor of safety for all slides is equal.
- The failure surface does not pass below the toe of the wall.

From Figure 2,

$$\sum F_y = 0 \text{ (for each slice)} \quad (11)$$

$$V_{i+1} - V_i - W_i - q_{vi} + S_i \sin \alpha_i + N_i \cos \alpha_i = 0 \quad (12)$$

where V_i and V_{i+1} are the vertical interslice forces calculated by integration of overburden pressures on horizontal border of slice in a similar method to that used by Atkinson (1993) and Shahgholi *et al.* (2001).

Again,

$$\tau_r = \frac{\tau_f}{FS} \text{ (for each slice)}$$

Thus

$$S_i = \frac{1}{FS} (cb_i + N_i \tan \phi) \quad (13)$$

$$\sum F_x = 0 \text{ (for the whole wedge)} \quad (14)$$

Considering m layers,

$$\sum_{j=1}^m t_j + \sum_{i=1}^n S_i \cos \alpha_i - \sum_{i=1}^n N_i \sin \alpha_i - \sum_{i=1}^n q_{hi} = 0 \quad (15)$$

Substituting for S_i from Equation 13 into Equation 12,

$$N_i = \frac{V_i - V_{i+1} + W_i + q_{vi} - (cb_i/FS) \sin \alpha_i}{(\tan \phi / FS) \sin \alpha_i + \cos \alpha_i} \quad (16)$$

N_i is calculated using Equation 16, and S_i is calculated using Equation 13. We can determine t_j when FS is known.

The geosynthetic tensile reinforcement force t_j required to maintain local stability for layer j can be normalised to a parameter K , which is equivalent to the earth pressure coefficient

$$K = \frac{\sum t_j}{0.5\gamma H^2} \approx \frac{t_j}{\gamma h_j D_j} \quad (17)$$

where γ is the unit weight of the soil, and D_j and h_j are the tributary area and depth, respectively, of layer j .

For layer j under consideration, the linear failure surface making an angle α_i with the horizontal is selected to give the maximum force, t_j ; that is, the required geosynthetic tensile reinforcement force t_j for layer j is optimised with respect to the failure surface angle α_i . The search for the maximum forces is carried out using the Excel spreadsheet tool SOLVER.

The outermost critical multilinear failure surface obtained from the tieback analysis defines the active soil mass, as shown in Figure 1. It defines the boundary on which the geosynthetics anchor forces are applied so that an internally stable structure is produced. Each geosynthetic layer is extended into the stable backfill soil, i.e. tieback, so that the required geosynthetic tensile reinforcement force t_j can be mobilised.

2.4. Compound stability analysis

Potential slip surfaces emerging beyond the outermost multilinear failure surface obtained in the tieback analysis, either outside or within the effective anchorage length, may not allow sufficient geosynthetic force to be mobilised, and hence may not provide adequate stability. Some layers may require a greater length of geosynthetic than that obtained from tieback analysis: that is, these layers are needed to resist possible compound failures in which the slip surfaces pass through both retained and reinforced soils. The geosynthetic layers are then anchored beyond all critical failure surfaces determined from tieback and compound stability analyses.

2.5. Anchorage length

The required anchorage length $l_{e,j}$ for layer j is determined as

$$l_{e,j} = \frac{t_j}{2\sigma_{v,j}C_i \tan \phi} \quad (18)$$

where C_i is the soil–geosynthetic pullout coefficient, expressed as the ratio of the soil–geosynthetic pullout strength to the soil strength (i.e. $\tan \phi$), t_j is the required tensile resistance of layer j , and $\sigma_{v,j}$ is the overburden pressure acting on the anchored portion of this layer.

The maximum length required to resist tieback and

compound failure (l_c), can be normalised to parameter L_c , which is equal to l_c/H .

3. RESULTS AND DISCUSSIONS

3.1. Parametric study

To study the effects of pseudo-dynamic seismic inertia forces on the required geosynthetic strength and lengths, a parametric study was conducted on a reinforced soil wall with slope angle $\beta = 90^\circ$, height $H = 5$ m, unit weight of the soil $\gamma = 18$ kN/m³ and cohesion $c = 0$, by considering 20 equally spaced geosynthetic layers. The soil–geosynthetic pullout coefficient (C_i) is selected as 0.8 and the factor of safety (FS) is taken as 1.0 in the present analysis.

The results are presented in the form of tables and graphs. Variations of parameters considered in the present analyses are as follows.

- $\phi = 20^\circ, 25^\circ, 30^\circ$
- $k_h = 0.0, 0.1, 0.2, 0.3$
- $k_v = 0.0, 0.5k_h$ and $1.0k_h$
- $V_s = 100$ m/s, $V_p = 187$ m/s, $H = 5$ m, $T = 0.3$ s, $H/\lambda = 0.167$, $H/\eta = 0.09$, $\beta = 90^\circ$.

Table 1 shows the values of the required sum of geosynthetic tensile reinforcement force $\Sigma t_{j\max}$ for different values of k_h , k_v and ϕ . It is evident that the stability of the reinforced soil wall decreases as the values of k_h and k_v increase, thus showing that a greater $\Sigma t_{j\max}$ is required to maintain the stability of the wall.

3.2. Effect of soil friction angle ϕ on normalised reinforcement force K

Figures 3 and 4 show the effect of soil friction angle ϕ on the required normalised reinforcement force K with $k_h = 0.1$ and 0.2 respectively. It can be seen that the required value of K decreases with increase in ϕ . For $k_h = 0.2$ and $k_v = 0.5k_h$, when ϕ changes from 20° to 25° , the required value of K decreases by about 19%. Similarly, when ϕ changes from 25° to 30° , the required value of K decreases by about 22%.

Table 1. Required sum of geosynthetic reinforcement layer forces ($\Sigma t_{j\max}$) for different values of k_h , k_v and ϕ with $\beta = 90^\circ$, $H/\lambda = 0.167$, $H/\eta = 0.09$ and $H = 5$ m

k_h	k_v	Required geosynthetic reinforcement, $\Sigma t_{j\max}$ (kN/m)		
		$\phi = 20^\circ$	$\phi = 25^\circ$	$\phi = 30^\circ$
0.0	0.00	110	91	75
	0.05	137	113	93
0.1	0.05	174	141	106
	0.10	226	175	126
	0.00	164	135	109
0.2	0.10	211	169	132
	0.20	269	208	164
	0.00	196	160	130
	0.15	247	189	156

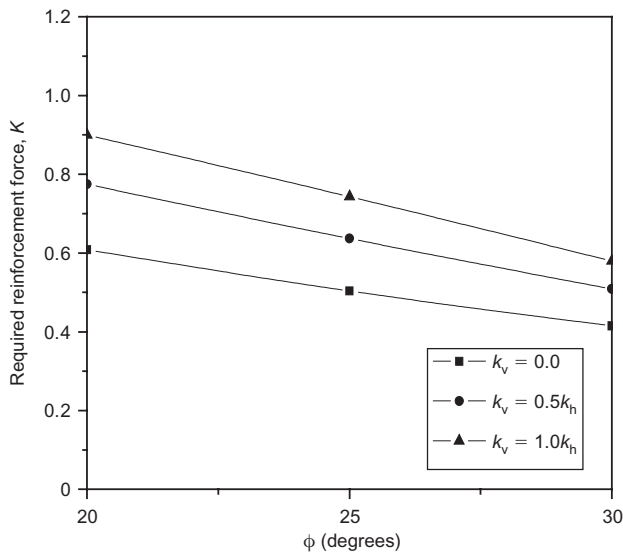


Figure 3. Required normalised geosynthetic tensile reinforcement force (K) against soil friction angle at different seismic coefficients with $k_h = 0.1$, $H/\lambda = 0.167$, $H/\eta = 0.09$, $H = 5$ m, $\beta = 90^\circ$

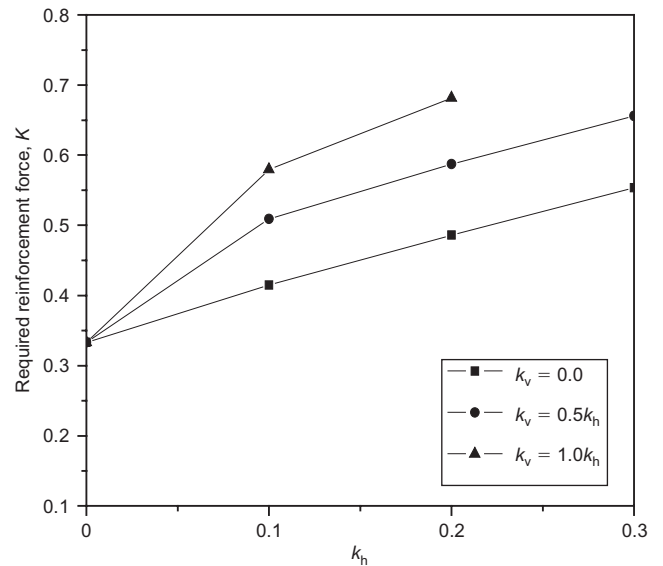


Figure 5. Effect of seismic coefficients on required geosynthetic tensile reinforcement force (K) with $\phi = 30^\circ$, $H/\lambda = 0.167$, $H/\eta = 0.09$, $H = 5$ m, $\beta = 90^\circ$

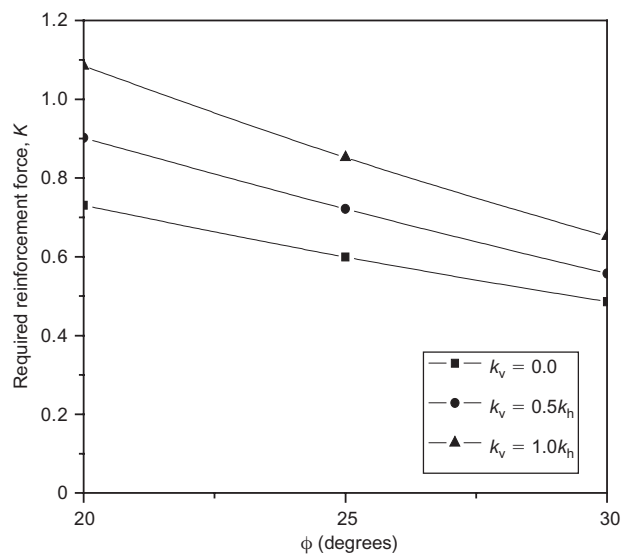


Figure 4. Required normalised geosynthetic tensile reinforcement force (K) against soil friction angle at different seismic coefficients with $k_h = 0.2$, $H/\lambda = 0.167$, $H/\eta = 0.09$, $H = 5$ m, $\beta = 90^\circ$

3.3. Effect of seismic acceleration coefficients k_h and k_v on required normalised reinforcement force K

Figure 5 shows the effects of the horizontal and vertical seismic acceleration coefficients k_h and k_v on the required normalised reinforcement force K for $\phi = 30^\circ$. It is evident that the required value of K shows a significant increase with increase in k_h and k_v . Also, the rate of increase is less for higher seismic accelerations. For $k_v = 0.5k_h$, when k_h changes from 0 to 0.1, the required value of K increases by about 41%. When k_h changes from 0.1 to 0.2, the required value of K increases by about 25%, and when k_h changes from 0.2 to 0.3, the required value of K increases by about 18%. For $k_h = 0.2$, when k_v

changes from 0 to $0.5k_h$, the required value of K increases by about 21%, and when k_v changes from $0.5k_h$ to $1.0k_h$, the required value of K increases by about 24%. Thus the effects of both k_h and k_v are significant in the computation of the required geosynthetic tensile reinforcement force as expressed by K .

3.4. Effect of period of lateral shaking T on required normalised reinforcement force K

Figure 6 shows the effect of the period of lateral shaking T on the required value of K with $k_v = 0.5k_h$, $\phi = 30^\circ$, $H = 5$ m and $\beta = 90^\circ$. It can be seen that, for $k_h = 0.2$, the value of K required to maintain stability of the reinforced-soil wall corresponding to $T = 0.2$ s is 3.9%, 10.2% and 16.8% smaller than that corresponding to $T =$

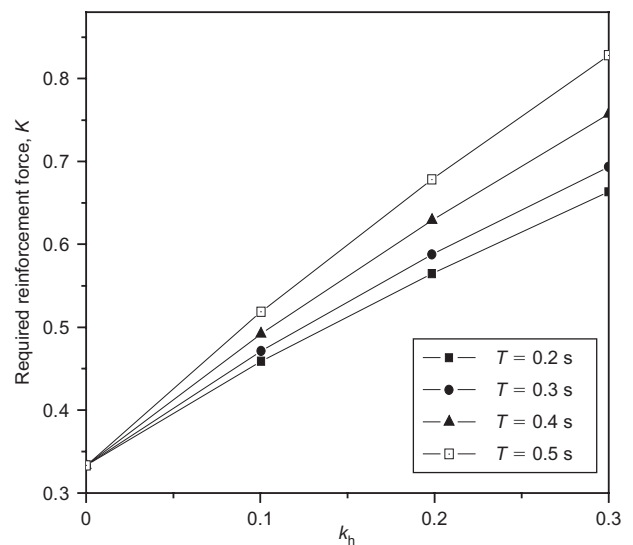


Figure 6. Effect of period of lateral shaking (T) on required normalised geosynthetic tensile reinforcement force (K) with $k_v = 0.5k_h$, $\phi = 30^\circ$, $H = 5$ m, $\beta = 90^\circ$

0.3 s, 0.4 s and 0.5 s, respectively. Thus it is evident that, as T increases, the seismic stability of the reinforced-soil wall decreases, requiring greater tensile reinforcement force. For most geotechnical structures $T = 0.3$ s is a reasonable value (Prakash 1981). Hence, for all other results reported in this paper, $T = 0.3$ s is used, and the results are expressed in terms of dimensionless parameters such as $H/\lambda = 0.167$ and $H/\eta = 0.09$.

3.5. Effect of soil friction angle ϕ on required geosynthetic length L_c

Figures 7 and 8 show the effect of soil friction angle ϕ on the required geosynthetic length L_c with $k_h = 0.1$ and 0.2 , respectively. Again, as before, $H/\lambda = 0.167$, $H/\eta = 0.09$, $H = 5$ m and $\beta = 90^\circ$. It can be seen that L_c decreases with increases in ϕ . For $k_h = 0.2$ and $k_v = 0.5k_h$, when ϕ changes from 20° to 25° , L_c decreases by about 53%, and when ϕ changes from 25° to 30° , L_c decreases by about 39%. Thus the effect of soil friction angle is more pronounced on the required geosynthetic length than on the required geosynthetic tensile reinforcement force as expressed by parameter K .

3.6. Effect of seismic acceleration coefficients k_h and k_v on required geosynthetic length L_c

Figure 9 shows the effect of horizontal and vertical seismic acceleration coefficients k_h and k_v on the required geosynthetic length L_c with $\phi = 30^\circ$, $H/\lambda = 0.167$, $H/\eta = 0.09$, $H = 5$ m and $\beta = 90^\circ$. It is evident that L_c shows an increase with increase in k_h and k_v , and the rate of increase is less for higher seismic accelerations. For $k_v = 0.5k_h$, when k_h changes from 0 to 0.1, L_c increases by about 111%; when k_h changes from 0.1 to 0.2, L_c increases by about 64%; and when k_h changes from 0.2 to 0.3, L_c increases by about 62%. Also, for $k_h = 0.2$, when k_v changes from 0 to $0.5k_h$, L_c increases by about 44%, and when k_v changes from $0.5k_h$ to $1.0k_h$, L_c increases by

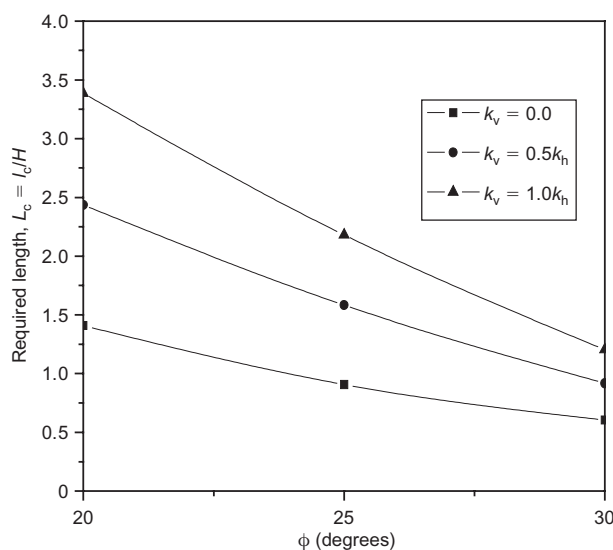


Figure 7. Required geosynthetic length (L_c) against soil friction angle at different seismic coefficients with $k_h = 0.1$, $H/\lambda = 0.167$, $H/\eta = 0.09$, $H = 5$ m, $\beta = 90^\circ$

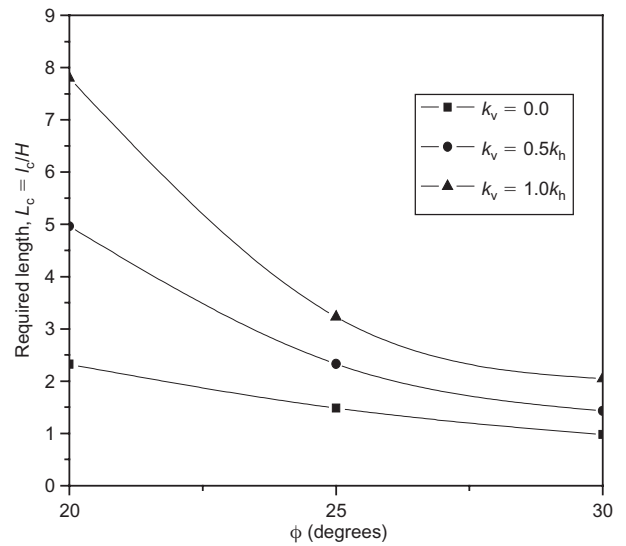


Figure 8. Required geosynthetic length (L_c) against soil friction angle at different seismic coefficients with $k_h = 0.2$, $H/\lambda = 0.167$, $H/\eta = 0.09$, $H = 5$ m, $\beta = 90^\circ$

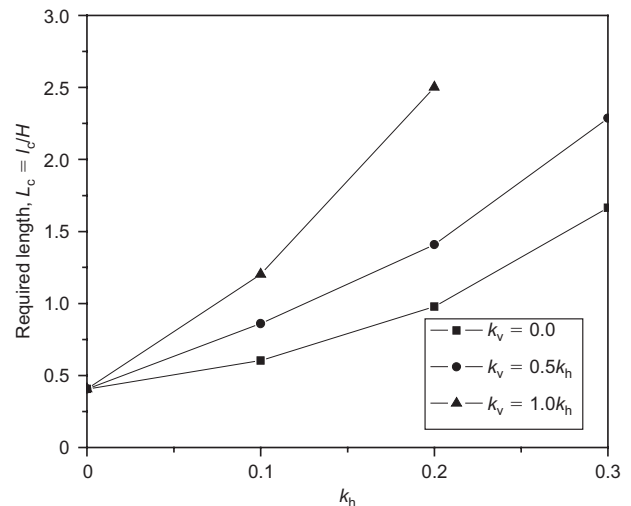


Figure 9. Effect of seismic coefficients on required geosynthetic length (L_c) with $\phi = 30^\circ$, $H/\lambda = 0.167$, $H/\eta = 0.09$, $H = 5$ m, $\beta = 90^\circ$

about 78%. Thus the magnitudes of both the horizontal and vertical seismic acceleration coefficients have an important influence in the computation of the required length of the geosynthetic layer.

3.7. Effect of period of lateral shaking T on required geosynthetic length L_c

Figure 10 shows the effect of the period of lateral shaking T on the required geosynthetic length L_c with $k_v = 0.5k_h$, $\phi = 30^\circ$, $H = 5$ m and $\beta = 90^\circ$. It can be seen that, for $k_h = 0.2$, the geosynthetic length L_c required to maintain stability of the reinforced-soil wall corresponding to $T = 0.2$ s is 4.6%, 11.9% and 17.3% smaller than those corresponding to $T = 0.3$ s, 0.4 s and 0.5 s, respectively. Thus it is evident from the plot that, as the period of lateral shaking increases, the seismic stability of the

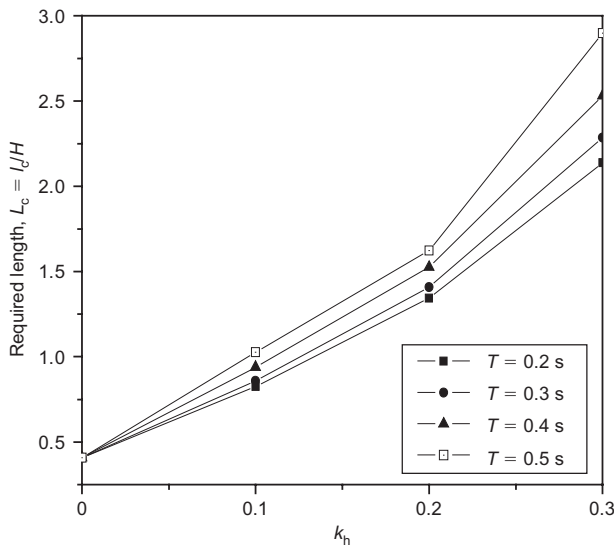


Figure 10. Effect of period of lateral shaking (T) on required geosynthetic length (L_c) with $k_v = 0.5k_h$, $\phi = 30^\circ$, $H = 5$ m, $\beta = 90^\circ$

reinforced-soil wall decreases, requiring longer geosynthetic layers.

4. COMPARISON OF RESULTS

The pseudo-dynamic analysis of a typical reinforced soil wall by the method of horizontal slices has been presented and compared with the pseudo-static results using the method of horizontal slices proposed by Shahgholi *et al.* (2001) and the analytical model implemented in the program ReSlope by Leshchinsky (1997) and Ling *et al.* (1997). In the ReSlope program, the slip surface is assumed to be a log-spiral, whereas a multilinear slip surface is assumed both in the horizontal slice method proposed by Shahgholi *et al.* (2001) and in the present study. Table 2 shows the values of the required geosynthetic reinforcement force Σt_{jmax} for different values of k_h and ϕ with $\beta = 90^\circ$, $k_v = 0.0$, $H/\lambda = 0.167$, $H/\eta = 0.09$ and $H = 5$ m. For $\phi = 20^\circ$, when k_h changes from 0.0 to 0.1, the required geosynthetic reinforcement force Σt_{jmax} calculated by the pseudo-dynamic method increases by 25%, compared with 16% as computed by the pseudo-static method. For the static case, values using the analysis presented in

this paper are exactly the same as those from Shahgholi *et al.* (2001), as expected. For the dynamic case, however, the pseudo-dynamic method originally proposed by Steedman and Zeng (1990) and further modified by Choudhury and Nimbalkar (2005, 2006) gives design values for required geosynthetic reinforcement strength and length by considering all probable dynamic factors in the design compared with the conventional pseudo-static approach. Also, in Table 3, the comparison of results for the required geosynthetic length L_c computed in the present study using pseudo-dynamic method with the pseudo-static results of Ling and Leshchinsky (1998) shows that the dynamic behaviour of a reinforced-soil wall system demands higher values of geosynthetic length than those obtained using the simplified pseudo-static method.

5. CONCLUSIONS

The seismic stability of reinforced soil walls has been investigated using a pseudo-dynamic method of analysis that incorporates the method of horizontal slices. The effects of various parameters on reinforced soil wall design, such as soil friction angle and horizontal and vertical seismic accelerations, have been shown. Phase change in both the shear and the primary waves propagating through the reinforced soil wall influences the design of the wall. The seismic stability of the reinforced soil wall reduces with increases in both k_h and k_v . From the results of pseudo-dynamic analyses it is clear that both the horizontal and vertical seismic accelerations are important parameters for computation of the required length and tensile strength of the geosynthetic reinforcement, and, moreover, their importance increases as the earthquake intensity increases.

Table 3. Typical comparison of present results with pseudo-static results by Ling and Leshchinsky (1998)

k_v	Required length of geosynthetic layer, L_c	
	Method proposed by Ling and Leshchinsky (1998)	Present study
0.0	0.818	0.978
0.1	0.857	1.428
0.2	0.912	2.046

Data used: $k_h = 0.2$, $\phi = 30^\circ$, $H/\lambda = 0.167$, $H/\eta = 0.09$, $H = 5$ m, $\beta = 90^\circ$.

Table 2. Typical comparison of present results with pseudo-static results by horizontal slice method (HSM) using method of Shahgholi *et al.* (2001) and results of ReSlope program by Leshchinsky (1997), Ling *et al.* (1997).

k_h	Required geosynthetic reinforcement, Σt_{jmax} (kN/m)								
	$\phi = 20^\circ$			$\phi = 25^\circ$			$\phi = 30^\circ$		
	ReSlope	HSM	Present study	ReSlope	HSM	Present study	ReSlope	HSM	Present study
0.0	110	110	110	95	91	91	74	75	75
0.1	128	128	137	110	107	113	90	89	93
0.2	151	151	164	126	127	135	106	106	109
0.3	187	187	196	153	153	160	128	128	130

Data used: $k_v = 0.0$, $H/\lambda = 0.167$, $H/\eta = 0.09$, $H = 5$ m, $\beta = 90^\circ$

With increases in the horizontal and vertical seismic accelerations, the required geosynthetic tensile strength and length increase. It is also evident from the analysis that the effect of soil friction angle and seismic horizontal and vertical accelerations on geosynthetic length is more pronounced than that on the tensile strength of the geosynthetic reinforcement. Comparisons presented in the paper with available pseudo-static methods yielded satisfactory agreement. Moreover, the present study shows that greater length and tensile strength of the geosynthetic reinforcement is required for safe design of geosynthetic-reinforced soil walls in earthquake-prone areas using a dynamic analysis as compared with conventional pseudo-static design methods.

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NOTATIONS

Basic SI units are given in parentheses.

$a(z, t)$	acceleration at depth z , time (m/s ²)
a_h, a_v	amplitude of horizontal and vertical seismic acceleration (m/s ²)
b_j	length of base of slice (m)
c	cohesion of soil (N/m ²)
C_i	soil-geosynthetic pullout coefficient (dimensionless)
D_j	tributary area for geosynthetic layer j (m)
FS	factor of safety (dimensionless)
g	acceleration due to gravity (m/s ²)
G	shear modulus of soil (N/m ²)
h	vertical distance between any point in soil mass and external borders of soil mass (m)
H	wall height (m)
h_j	depth of geosynthetic layer j (m)
K	normalised geosynthetic tensile reinforcement force (equivalent earth pressure coefficient) (dimensionless)
k_h, k_v	seismic acceleration coefficient in horizontal and vertical directions (dimensionless)
l_c, l_e	geosynthetic lengths required to resist compound failure and pullout (m)
L_c, L_e	normalised geosynthetic lengths required to resist compound failure and pullout (dimensionless)
m	number of reinforcement layers (dimensionless)
n	number of slices (dimensionless)
N_i	normal force upon base of slice (N/m)
q_{hi}, q_{vi}	horizontal and vertical inertia force due to seismic acceleration acting at layer i (N/m)
S_i	shear force upon base of slice (N/m)
T	period of lateral shaking (s)
t	time (s)

t_j	required geosynthetic force in layer j (N/m)
V_p	primary wave velocity (m/s)
V_s	shear wave velocity (m/s)
W_i	weight of slice (N/m)
z_j	depth from top of wall to top of horizontal slice j
α_i	angle of base of elemental slice (degree)
β	slope angle (degrees)
γ	unit weight of soil (kN/m ³)
δ	wall friction angle (degrees)
ζ	$t - H/V_s$ (s)
η	wavelength of the vertically propagating primary wave, TV_p (m)
λ	wavelength of the vertically propagating primary wave, TV_s (m)
ν	Poisson's ratio (dimensionless)
ρ	density of soil (kg/m ³)
σ_v	overburden pressure (N/m ²)
τ_f	failure shear stress (N/m ²)
τ_r	required shear stress (N/m ²)
ϕ	soil friction angle (degrees)
ψ	$t - H/V_p$ (s)
ω	angular frequency of base shaking (rad/s)

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