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An analytical technique for estimation of seismic displacements in reinforced slopes based on horizontal slices method (HSM)

Ali Ghanbari*[1], Abbas Khalilpasha[1], Mohsen Sabermahani[2] and Babak Heydari[1]

[1]*Faculty of Engineering, Kharazmi University, No. 49 Mofatteh Ave., Tehran, I.R. Iran*
[2]*Faculty of Civil Engineering, Iran University of Science and Technology, Narmak, Tehran, I.R. Iran*

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Abstract. Calculation of seismic displacements in reinforced slopes plays a crucial role in appropriate design of these structures however current analytical methods result indifferent values for permanent displacements of the slope. In this paper, based on limit equilibrium and using the horizontal slices method, a new formulation has been proposed for estimating the seismic displacements of a reinforced slope under earthquake records. In this method, failure wedge is divided into a number of horizontal slices. Assuming linear variations for tensile forces of reinforcements along the height of the slope, the coefficient of yield acceleration has been estimated. The simplicity of calculations and taking into account the frequency content of input triggers are among the advantages of the present formulation. Comparison of the results shows that the yield acceleration calculated by the suggested method is very close to the values resulted from other techniques. On the other hand, while there is a significant difference between permanent displacements, the values obtained from the suggested method place somehow between those calculated by the other techniques.

Keywords: seismic displacement; reinforced slope; horizontal slices method; permanent deformation

1. Introduction

Reinforced slopes have been extensively used in recent years because of their satisfactory seismic performance and cost effectiveness. They also perform much better in earthquake conditions, compared with the other types of retaining structures (Sandri 1997, White and Holtz 1997, Tatsuoka *et al*. 1995, 1997, Ling *et al*. 2001).

As an example, although some displacements were observed in the reinforced earth walls, no catastrophic failure took place in the Hyogoken-nambu earthquake even for the magnitudes greater than the design value (e.g., Tatsuoka *et al*. 1997).

The seismic response of geosynthetic reinforced walls has recently been the subject of considerable studies, such as full-scale structures (Collin 2001, Kazimierowicz-Frankowska 2005, Lee and Wu 2004, Yoo 2004, Yoo and Jung 2004, Won and Kim 2007), reduced-scale models (Latha and Krishna 2008, El-Emam and Bathurst 2007, Nova-Roessig and Sitar 1999, Chen *et al*. 2007, Sabermahani *et al*. 2008) and numerical analyses (Al-Hattamleh and Muhunthan 2006,

*Corresponding author, Associate Professor, E-mail: Ghanbari@tmu.ac.ir

Rowe and Skinner 2001, Skinner and Rowe 2005, Hatami and Bathurst 2000, Huang *et al*. 2006). Furthermore, analytical models such as the homogenized analytical concept (Chen *et al*. 2000), limit analysis (Porbaha *et al*. 2000, Mojalall and Ghanbari 2012a, 2012b), limit equilibrium (Baker and Klein 2004a, 2004b, Nouri *et al*. 2006, 2008, Shekarian *et al*. 2008, Reddy *et al*. 2008, Ahmadabadi and Ghanbari 2009, Ghanbari and Taheri 2012) and the characteristics method (Jahanandish and Keshavarz 2005) have been developed.

Investigation of the obtained results from available methods shows that there is a considerable difference between the displacements reported by various researchers. On the other hand, seismic displacements are mostly calculated in available methods merely based on the maximum acceleration of the earthquake and regardless of the frequency content of input triggers. In this paper, an analytical procedure based on the horizontal slices method, limit equilibrium concept and Newmark (1965) sliding block analysis is used to evaluate the seismic displacements of reinforced slopes considering the effect of frequency content of input triggers.

2. Literature review

A pioneer study on the seismic displacement of soil retaining walls was performed by Richards and Elms (1979) using a pseudo static approach. They pointed out the importance of wall's inertia on the seismic displacements of gravity walls. Zonberg et al. (1998) have shown through centrifuge tests that limit equilibrium analyses provide valid indications of factor of safety and failure mechanisms for reinforced slops.

Displacement-based analyses have recently become more important as engineers focus on performance-based design methodology. That is why FHWA, 2001 recommends the well known Newmark (1965) sliding block analysis to be used for the estimation of the seismic displacement of walls subjected to PGA > 0.3 g. Development of the Newmark method for reinforced-soil slopes is described in various works (Bathurst and Alfaro 1996, Bathurst *et al*. 2002, Cai and Bathurst 1996, Huang *et al*. 2003, Kramer and Paulsen 2004, Huang and Wang 2005, Huang and Wu 2006, Huang and Wu 2007).

Using the limit equilibrium technique, Cai and Bathurst (1996) calculated the coefficient of yield acceleration, k_y, and applied the Newmark method to determine the permanent sliding displacements of GRS-MB walls. In this study, the internal and external sliding mechanisms and shear failure at the block's interface were taken into account, and based on these three mechanisms, k_y and the wall's displacements were determined. Based on a pseudo static limit equilibrium analysis, Ling *et al*. (1997) proposed a seismic design procedure for geosynthetic-reinforced soil structures. Using their method, the reinforced zone is treated as a rigid block, and only the horizontal sliding along the base of the reinforced soil zone was considered in the analysis. Passive resistance against the outward movement of the wall is not considered in their method. This may result in erroneous analytical results because the earth retaining walls are usually buried up to some depth.

Kramer and Paulsen (2004) also used a one-block model whereby the soil in the entire reinforced zone is assumed to move as a rigid block. This means that the slope deformations estimated by all of these methods will be uniform across the slope's face and will not have distinct internal or external mechanisms to allow non-uniform deformations of the facing to occur along the height of the slope. Using a database of instrumented and monitored full-scale field and laboratory slopes, Bathurst *et al*. (2005) established a working stress method to calculate the internal force of reinforcements in slopes reinforced by geosynthetics. The proposed method captures the essential contributions of the different slope components and properties in reinforcement forces.

Assuming the spiral logarithmic mode for the failure wedge and using the limit analysis method, Michalowski and You (2000) proposed a formulation and design charts for calculating the seismic displacements of reinforced slopes. Two mechanisms of failure of reinforced slopes subjected to seismic conditions are considered: (1) Rotational collapse; and (2) sliding directly over the bottom layer of reinforcement. Also, using the limit analysis method, Ausilio *et al*. (2000) proposed a solution for calculating the seismic displacements of soil slopes with uniform and nonuniform distribution of reinforcements. These researchers considered the planar and spiral shapes for the failure wedge. In the recent years, analytical calculation of seismic displacements in reinforced slopes has been studied by some other scholars as well.

Zarnani *et al*. (2011) described a numerical model that was developed to simulate the dynamic response of two instrumented reduced-scale model reinforced soil slopes constructed on a 1-g shaking table. Both physical and numerical modelling results showed that the magnitude and distribution of reinforcement connection loads during static and dynamic loading are influenced by the toe boundary condition. Using the limit analysis theorem, Mojallal and Ghanbari (2012) and Mojallal *et al*. (2012) proposed an innovative approach for calculating the coefficient of yield acceleration and permanent displacements of geosynthetic-reinforced soil- retaining walls with full-height rigid concrete facing (GRS-FHR walls) and gravity walls. In this manner, based on the upper-bound theory of limit analysis and Newmark's sliding block theory, a group of charts has been proposed to estimate the permanent sliding displacements and the coefficient of yield acceleration for this type of wall.

On the other hand, the conventional vertical slices method is widely used for stability analyses of the slopes. Another commonly used method to handle these slopes was introduced by Shahgholi *et al*. (1999). The horizontal slices method (HSM) was expanded upon by Nouri *et al*. (2006, 2008). Ahmad and Choudhury (2008), Shekarian *et al*. (2008), Ahmadabadi and Ghanbari (2009) and Ghanbari and Ahmadabadi (2010a) employed the concept of HSM within the framework of static, pseudo-static and pseudo-dynamic methods to ascertain static and seismic active earth pressures on vertical retaining walls.

Moreover, based on Horizontal Slices Method a new formulation has been derived by Ghanbari and Ahmadabadi (2010b, 2010c) for determining the characteristics of inclined walls in frictional cohesive soils. The results of these studies show that active pressure distribution for inclined walls is non-linear along the height of the wall, which differs from the linear distribution resulting from previous studies. HSM permits determination of seismic active earth pressure distributions and the application point of the resultant earth pressures.

This paper presents the first analytical approach based on horizontal slices method for calculation of seismic displacements in geotechnical structures. It'll be the beginning for applying the capabilities of this method in simple analytical techniques for prediction of seismic displacements.

3. Proposed method

Fig. 1(a) shows a soil slope in which the reinforcements are arranged with a uniform pattern along the height. A planar slip surface passing through the toe of slope is depicted in this figure.

The tensile force of reinforcements has been assumed to be linearly distributed along the height with zero value at the crown and maximum value at the toe, as illustrated in Fig. 1(b). Fig. 2 shows the division of the slope into *n* horizontal slices. In order to calculate the coefficient of seismic acceleration in yield conditions, equilibrium equations have been written for all the slices. The relation between shear and normal stresses has been denoted by the Mohr-Coulomb yield criterion.

To determine the seismic displacement of reinforced slopes using HSM, following assumptions have been made in the present study

(1) The coordinate of the application point of the vertical inter slice force is the surface center of stress distribution derived from the succeeding equations.
(2) The failure surface is planar.
(3) The method is limited to homogeneous and granular soils.
(4) The failure surface is assumed to pass through the toe of the slope.
(5) The point where Ni acts on the slice base is located at the midpoint of that base.
(6) The point where Ti acts is located at the mid-height for each slice.
(7) The reinforcements are uniformly distributed along the height and the tensile force of reinforcements increases linearly from the crown to the bottom of the slope.

By analyzing all the slices, a system of 4*n* equations has been obtained. Solving this system, 4*n* unknowns of the problem can be determined. The used equations in this method are as follows:

Equilibrium of forces in horizontal direction for any slice

$$\vec{\Sigma} F_x = 0 \rightarrow S_i \cos\beta - N_i \sin\beta - H_i + H_{i+1} - k_y w_i + T_i = 0 \tag{1}$$

Equilibrium of forces in vertical direction for any slice

$$\overset{\uparrow}{\sum} F_y = 0 \Rightarrow V_{i+1} - W_i - V_i + S_1 \sin\beta + N_i \cos\beta = 0 \tag{2}$$

Equilibrium of moments around the point *O* (toe of the slope) for any slice

$$\begin{aligned} \sum M_O = 0 \Rightarrow H_i \sum_{j=i}^{n}(h_j) - H_{i+1} \sum_{j=i+1}^{n}(h_j) - V_i X_{v_i} + V_{i+1} X_{v_{i+1}} - W_i(\bar{X}_i) + k_y W_i(\bar{Y}_i) \\ + \frac{N_i}{\sin\beta}\left(\sum_{j=i+1}^{n} h_j + \frac{h_i}{2}\right) - T_i\left(\sum_{j=i+1}^{n} h_j + \frac{h_i}{2}\right) = 0 \end{aligned} \tag{3}$$

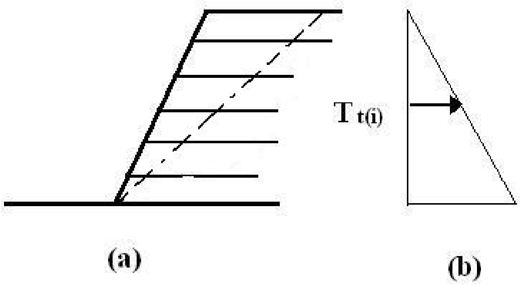

Fig. 1 (a) Reinforced slope and failure wedge; (b) force distribution in reinforcements along the height

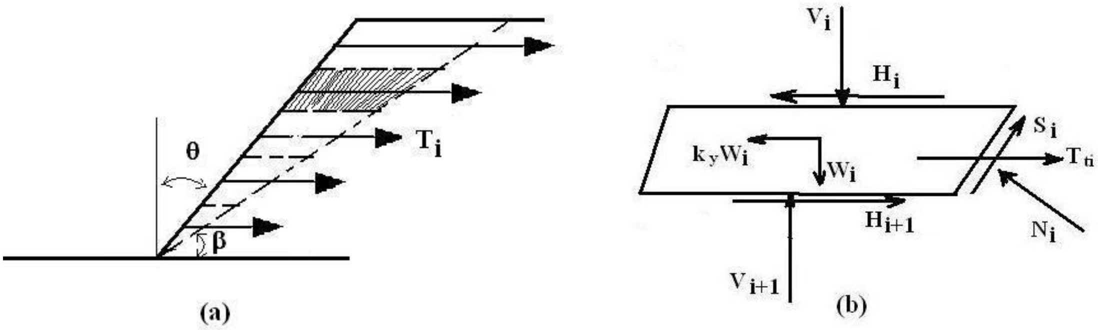

Fig. 2 (a) Divided slope into horizontal slices; (b) a separated horizontal slice and the forces acting on it

Mohr-Coulomb failure criterion for any slice

$$(S_i) - N_i \tan(\phi) = 0 \tag{4}$$

In the above equations, h_i is the height of the i^{th} slice. It has been assumed that all the slices have the same height. Also, X_{vi} is the moment arm of the vertical force, V_i, regarding the point O and can be calculated by following formula

$$X_{V_i} = \frac{\sum_{j=i}^{n} h_i \tan\theta}{2} + \frac{\sum_{j=i}^{n} h_j}{2\tan\beta} \tag{5}$$

Also $\overline{X}_i$ and $\overline{Y}_i$ are the horizontal and vertical distances between the center of gravity of the slice from the point O, respectively.

$$\overline{X}_i = \frac{\sum_{j=i}^{n} h_j}{2\tan\beta} + \frac{\sum_{j=i}^{n} h_i \tan\theta}{2} - \frac{h_i}{4\tan\beta} - \frac{h_i \tan\theta}{4} \tag{6}$$

$$\begin{aligned}\overline{Y}_i = & \left[\left(\frac{\sum_{j=i}^{n} h_j}{\tan\beta} - \sum_{j=1}^{n} h_i \tan\theta\right) + h_i \tan\theta\right] h_i \times \left(\sum_{j=i}^{n} h_j + \frac{h_j}{2}\right) \\ & - \frac{h_i^2 \tan\theta}{2}\left(\sum_{j=i}^{n} h_j + \frac{2h_j}{3}\right) - \frac{h_i^2}{2\tan\beta}\left(\sum_{j=i}^{n} h_j + \frac{h_i}{3}\right)\end{aligned} \tag{7}$$

Table 1 shows the equations and unknowns for the suggested formulation. The coefficient of yield acceleration (k_y) has been calculated by the proposed formulas. Applying the approach suggested by Newmark (1965) and double integration from the difference between the earthquake acceleration and the yield acceleration of accelerogram, permanent displacements have been

Table 1 Suggested formulas for calculating the coefficient of the critical seismic acceleration in reinforced slopes

Unknowns	Number	Equations	Number
H_i (Inter-slice shear force)	n-1	$\Sigma F_x = 0$ (For each slice)	n
N_i (Normal forces at base of each slice)	n	$\Sigma F_y = 0$ (For each slice)	n
S_i (Shear forces at base of each slice)	n	$\Sigma M_O = 0$ (For each slice)	n
V_i (Inter-slice vertical force)	n	$Si = N_i (\tan \phi)$ (For each slice)	n
k_y (Yield acceleration coefficient)	1		
Summation	4n		4n

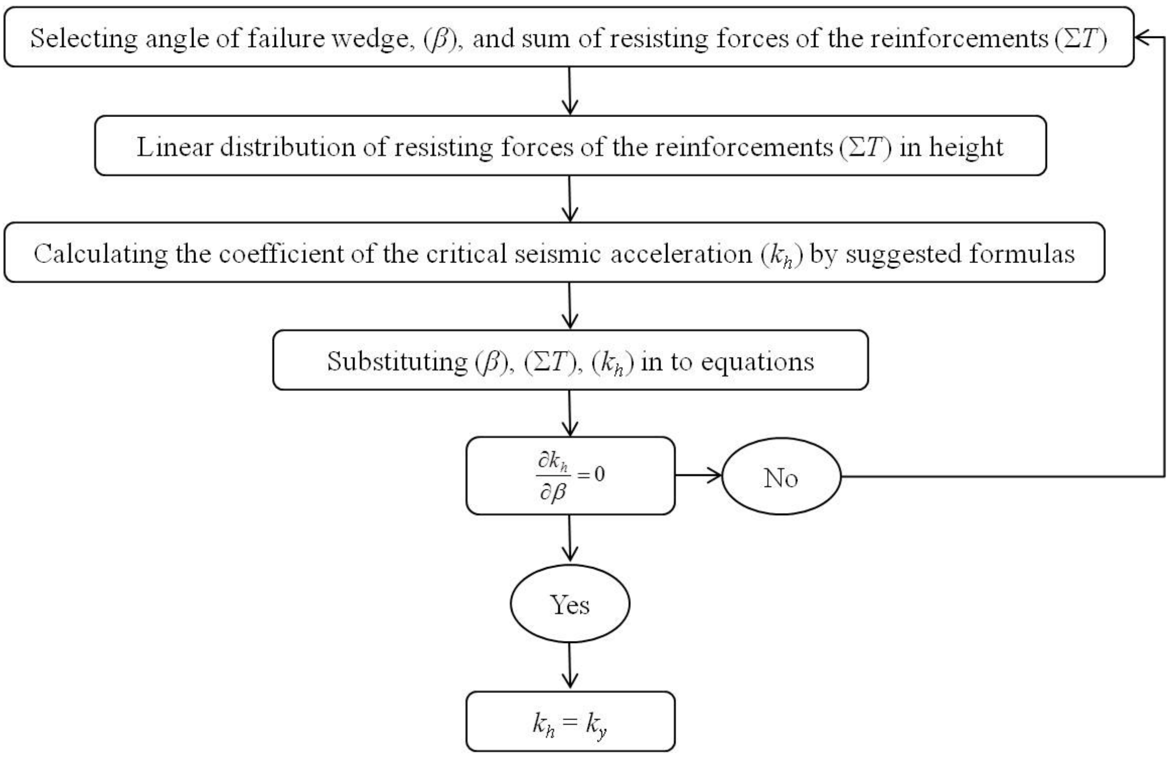

Fig. 3 Algorithm used to determine the yield acceleration

estimated. The above system equations have been solved by the Gauss-Jordan elimination method for 10 slices and the results are presented in the next sections. The algorithm used to determine the yield acceleration is shown in Fig. 3.

4. Results

In this section, the coefficient of yield acceleration and permanent displacements have been calculated, based on the suggested method, for a reinforced slope with 8 m height, slope face inclination angle of 30 degrees and specific weight of γ = 18 kN/m^3. For this purpose, four accelerograms related to some well-known earthquakes have been used. The records of these earthquakes are presented in following parts of this paper.

Variations of the coefficient of yield acceleration versus the angle of failure wedge are depicted in Figs. 4-6 for three cases of ΣT = 80, 100, 150 kN/m, respectively. The critical angle of failure wedge is the angle resulting in the minimum coefficient of yield acceleration. The minimum of the calculated coefficients of yield acceleration from analysis of different failure wedges has been taken as the real coefficient of yield acceleration (k_y). Fig. 4 shows the variations of the angle of failure wedge against the sum of reinforcing forces. Based on the obtained results, by increase in the sum of the reinforcing forces, the angle of failure wedge decreases with an almost linear pattern.

Variations of k_y against the internal friction angle of soil for different slopes are demonstrated in Fig. 5. With increase in the internal friction angle of soil, the coefficient of yield acceleration increases linearly as shown in Fig. 6. The results obtained from the suggested method show that

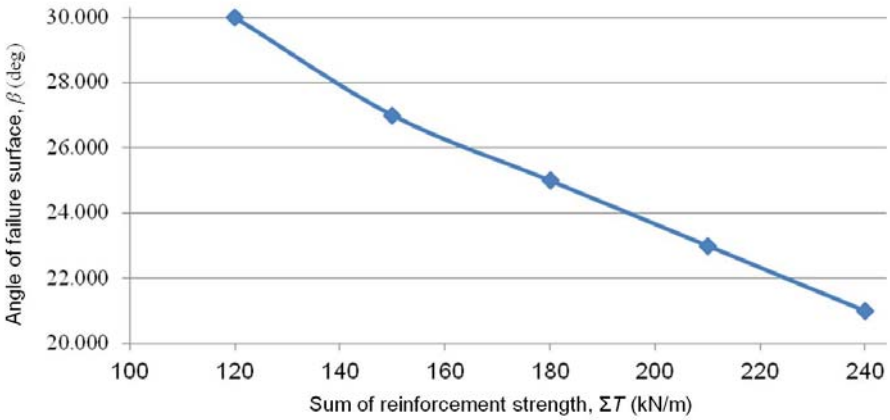

Fig. 4 Variations in the angle of failure wedge against ΣT (θ = 40°, ϕ = 20°)

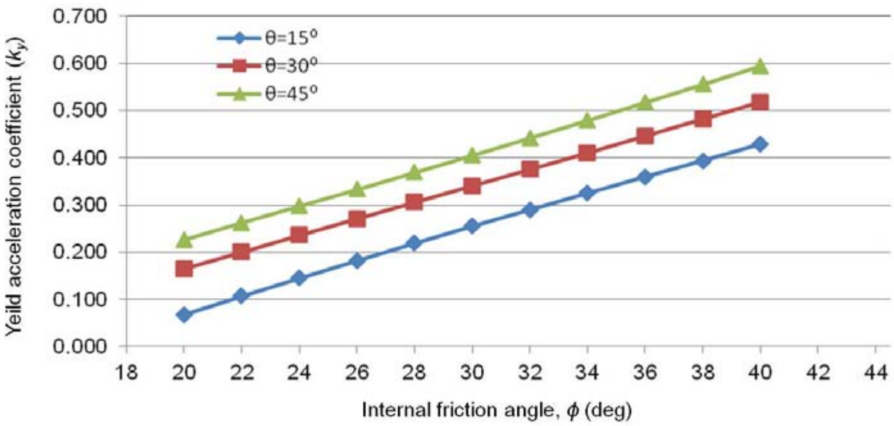

Fig. 5 Variations of the coefficient of yield acceleration against the internal friction angle of the soil (H = 5 m, ΣT = 90 kN/m)

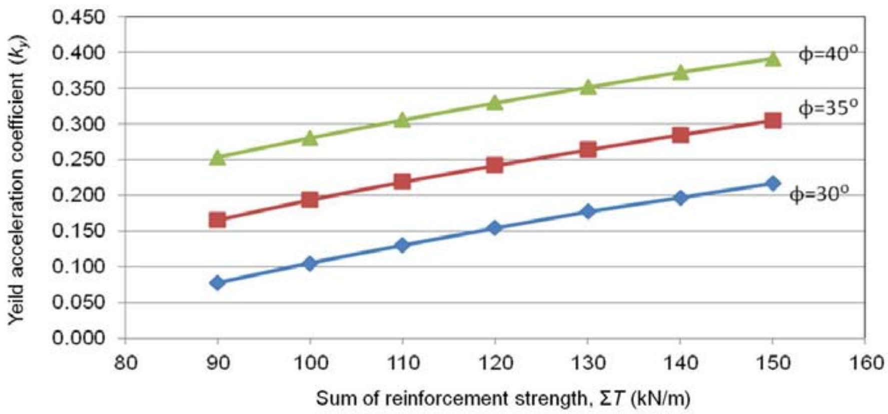

Fig. 6 Variations of the coefficient of yield acceleration against the sum of reinforcing forces (H = 8 m, θ = 30°)

the coefficient of yield acceleration is of a linear relation with the sum of resisting forces of the reinforcements (ΣT). Figs. 7 and 8 show the influence of the angle (θ) and height of the slope (H) on the value of k_y. Also Fig. 9 shows the variations of coefficient of yield acceleration versus the term $\Sigma T / \gamma H^2$ for different slopes.

After calculating the coefficient of yield acceleration (k_y), using the Newmark method and double integration from the difference between critical acceleration and the acceleration value of accelerogram, permanent displacements of the slope have been calculated. In this regard, four records of accelerations from previous earthquakes have been used for the purpose of analysis. Fig. 10 shows the variations of horizontal acceleration along the time. The properties of mentioned earthquakes are noted in Table 2.

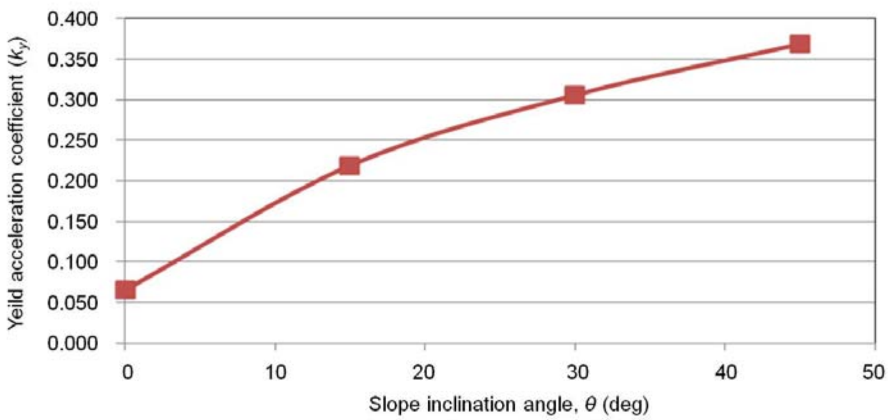

Fig. 7 Variations of the coefficient of yield acceleration against the slope inclination angle (H = 5 m, ϕ = 28°, ΣT = 90 kN/m)

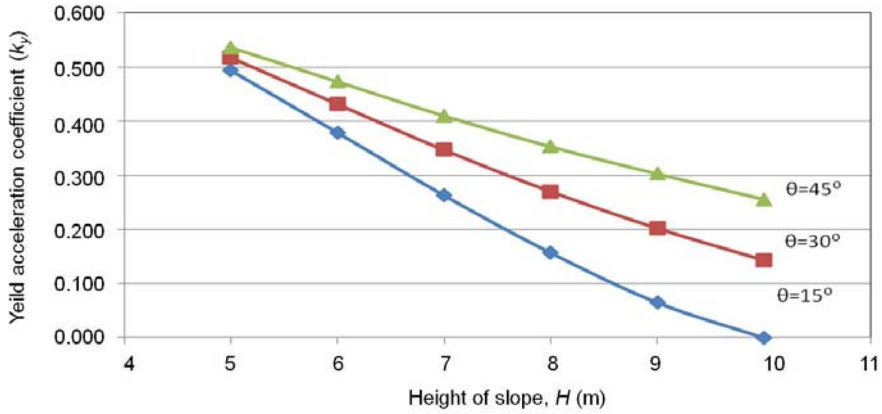

Fig. 8 Variations of the coefficient of yield acceleration against the height of the slope (ΣT = 180 kN/m, ϕ = 30°)

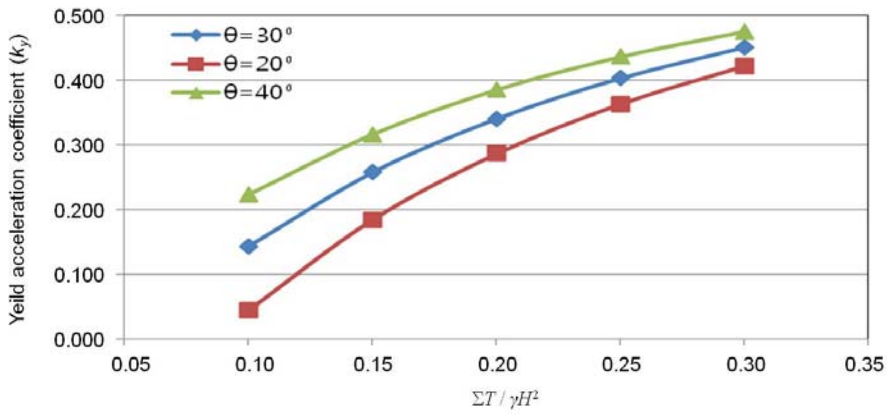

Fig. 9 Variations of the coefficient of yield acceleration against $\Sigma T / \gamma H^2$ (ϕ = 30°)

Table 2 Suggested formulas for calculating the coefficient of the critical seismic acceleration in reinforced slopes

Unknowns	Number	Equations	Number
H_i (Inter-slice shear force)	n-1	$\Sigma F_x = 0$ (For each slice)	n
N_i (Normal forces at base of each slice)	n	$\Sigma F_y = 0$ (For each slice)	n
S_i (Shear forces at base of each slice)	n	$\Sigma M_O = 0$ (For each slice)	n
V_i (Inter-slice vertical force)	n	$Si = N_i(\tan \phi)$ (For each slice)	n
k_y (Yield acceleration coefficient)	1		
Summation	4n		4n

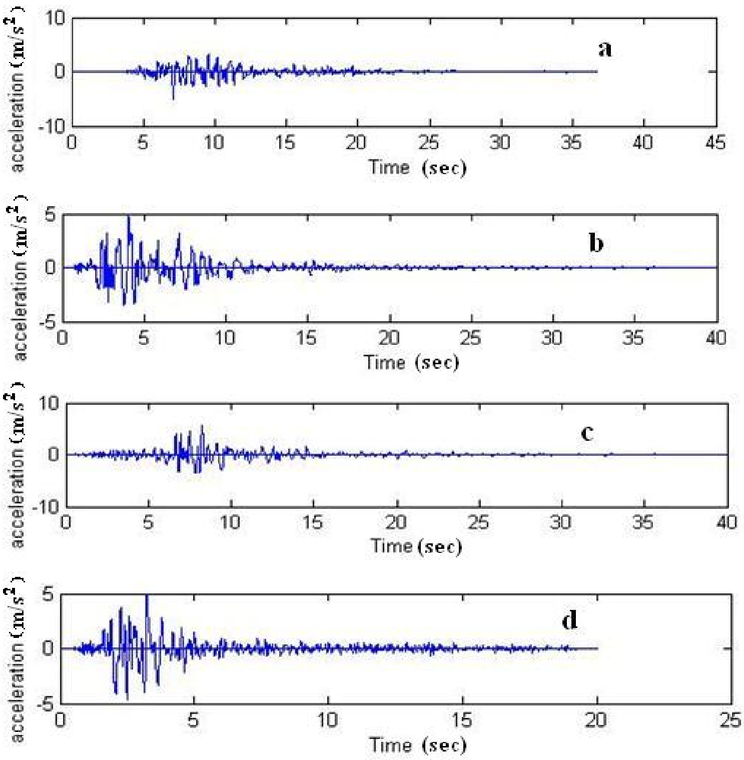

Fig. 10 Records of the used earthquakes (a) Kobe; (b) Loma Priata; (c) Northridge; (d) Palm Spring

Tables 3 and 4 show the permanent displacements calculated by the suggested method for a slope with 8 m height under different earthquakes. As can be observed in this figure, by increase in the soil's internal friction angle, the displacements have decreased non-linearly in all the cases. However, for the Northridge earthquake with the maximum acceleration of 5.68 m/s^2 the permanent displacements have been smaller than that of the Loma Prieta earthquake with

Table 3 Calculated displacements for the considered reinforced slope (θ = 20°, ΣT = 180 kN/m)

ϕ (deg.)	Permanent displacement (mm)			
	Northridge (Castaic – Old Ridge Route1994)	Kobe (Nishi-Akashi 1995)	Loma Prieta (Corralitos 1989)	Palm Springs (Whitewater Trout Farm 1986)
22	876	640	882	384
24	435	247	465	223
26	258	118	261	145
28	168	58	152	91
30	95	24	81	51
32	60	3	30	29

Table 4 Calculated displacements for the considered reinforced slope($\theta=30^0$‹ΣT=180 kN/m)

ϕ (deg.)	Permanent displacement (mm)			
	Northridge (Castaic – Old Ridge Route1994)	Kobe (Nishi-Akashi 1995)	Loma Prieta (Corralitos 1989)	Palm Springs (Whitewater Trout Farm 1986)
22	256	116	257	143
24	168	58	152	91
26	121	25	82	52
28	61	4	30	30
30	44	1	8	11
32	23	0	3	6

Table 5 Comparison of the k_y values calculated by the proposed method with those of other researchers for the slope associated with the curves shown in Fig. 11

ϕ (deg.)	k_y			
	Proposed Method	Michalowski and You (2000)	Ausilio *et al*. (2000)	
			Rotational mode	Translational mode
20	0.318	0.317	0.475	0.318
22	0.353	0.354	0.506	0.353
24	0.387	0.39	0.537	0.389
26	0.422	0.427	0.568	0.425
28	0.458	0.463	0.599	0.461
30	0.494	0.5	0.63	0.497
32	0.529	0.541	0.661	0.534
34	0.565	0.581	0.692	0.571
36	0.601	0.622	0.723	0.609
38	0.638	0.662	0.754	0.647
40	0.674	0.703	0.786	0.686

maximum acceleration of 4.79 m/s^2.This difference can be explained by the fact that in calculation of permanent displacements by the suggested method, the frequency content of the record, velocity content and duration of excitation are also important along with the maximum acceleration.

5. Comparison of the k_y values calculated by the suggested method and those of other researchers

In this part, the coefficients of yield acceleration calculated by the suggested formulation have been compared with the values calculated by Ausilio *et al*. (2000) and Michalowski and You (2000) methods. Assuming linear variation for the force of reinforcements, Michalowski and You (2000) proposed some curves for calculating the coefficient of yield acceleration for different internal friction angles of soil. Considering various modes of failure, i.e., spiral-logarithmic and planar, and by applying the limit analysis, Ausilio *et al*. (2000) proposed a formula for calculating

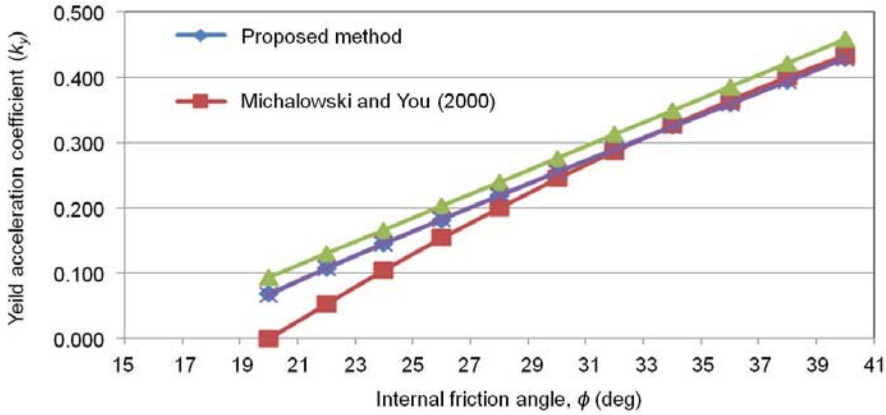

Fig. 11 Comparison of the critical accelerations for the case where H = 5 m, θ = 15°, ΣT = 90 kN/m, γ= 18 kN/m^3

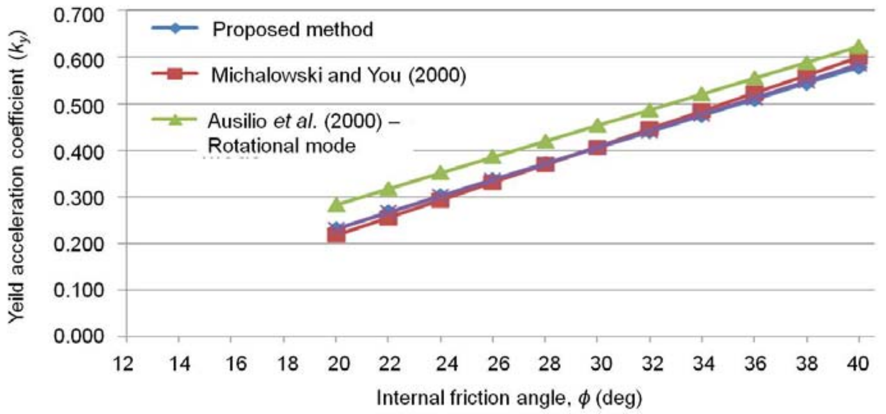

Fig. 12 Comparison of the critical accelerations for the case where H = 5 m, θ = 15°, ΣT = 135 kN/m, γ= 18 kN/m^3

the coefficient of yield acceleration (k_y) in reinforced slopes with uniformly distributed reinforcements. It has to be mentioned that although in both methods the coefficients of yield acceleration are obtained by the limit analysis method, however the solution presented by Michalowski and You (2000) has lower values than suggested technique by Ausilio *et al*. (2000).

Figs.11-13 illustrate a comparison between the coefficients of yield acceleration obtained from the suggested method for a slope with the height of 5 m and various values of ΣT with the values reported by other researchers. Generally speaking, the trend of increase in the coefficient of yield acceleration against increase in the internal friction angle of soil has been the same in all the three methods; however in translational mode the results derived from the suggested method are more consistent with those obtained from Ausilio *et al*. (2000) technique.

The difference between the results of these two modes is smaller than one percent. In order to provide a better way of comparison, the values associated with the curves in Fig. 11 are also indicated in Table 5. The obtained results show that by increase in the internal friction angle all the solutions converge to the same number. On the other hand, for high values of ΣT the results of present method are completely consistent with those from Michalowski and You (2000) method. However, in the rotational mode the values obtained from the method of Ausilio *et al*. (2000) are greater than those from the suggested method for all the cases.

Figs. 14-16 demonstrate the variations of k_y values obtained from different methods against the slope of reinforced slope (θ), for a slope with 5 m height and considering different internal friction angles. In all the mentioned graphs, it has been always assumed that kN/m ΣT = 90. Comparison of the results reveals that the values calculated from the suggested method generally fall between the results obtained from the other two methods however in low friction angles the present method gets closer to the method of Ausilio *et al*. (2000) and in high internal friction angles to the technique suggested by Michalowski and You (2000).

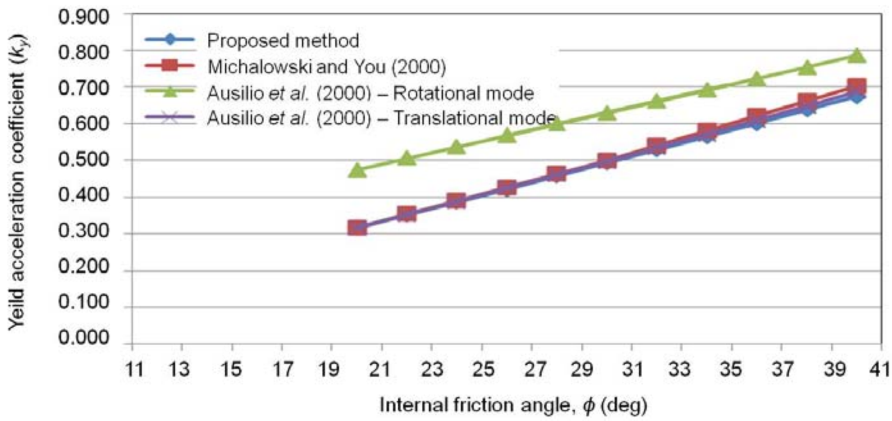

Fig. 13 Comparison of the critical accelerations for the case where H = 5 m, θ = 15°, ΣT = 180 kN/m, γ = 18 kN/m^3

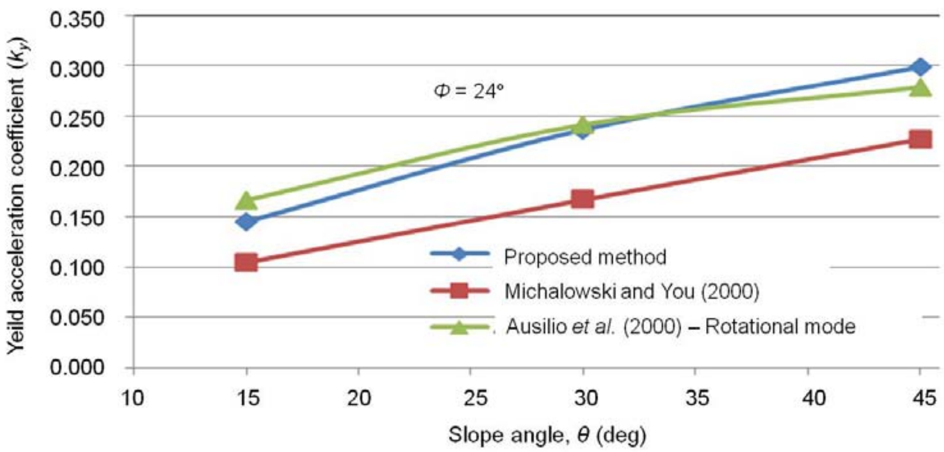

Fig. 14 Comparison of the k_y values obtained from different methods for various angles of slope at ϕ = 24°

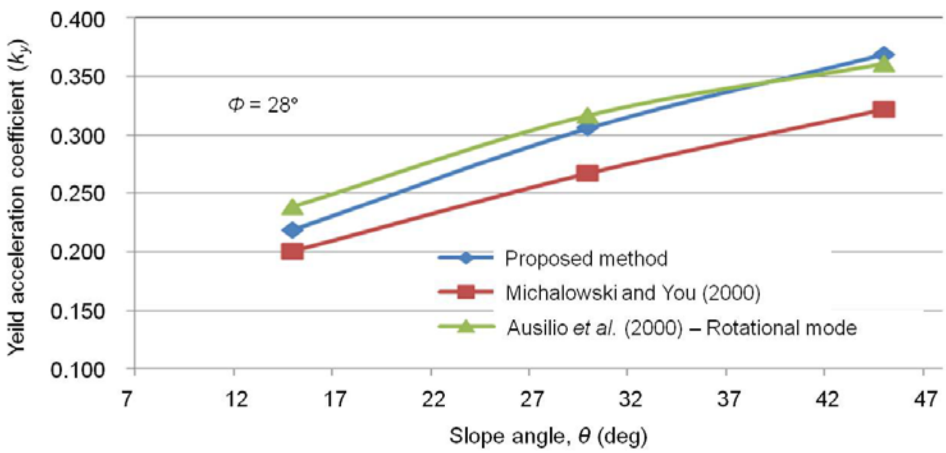

Fig. 15 Comparison of the k_y values obtained from different methods for various angles of slope at $\phi = 28°$

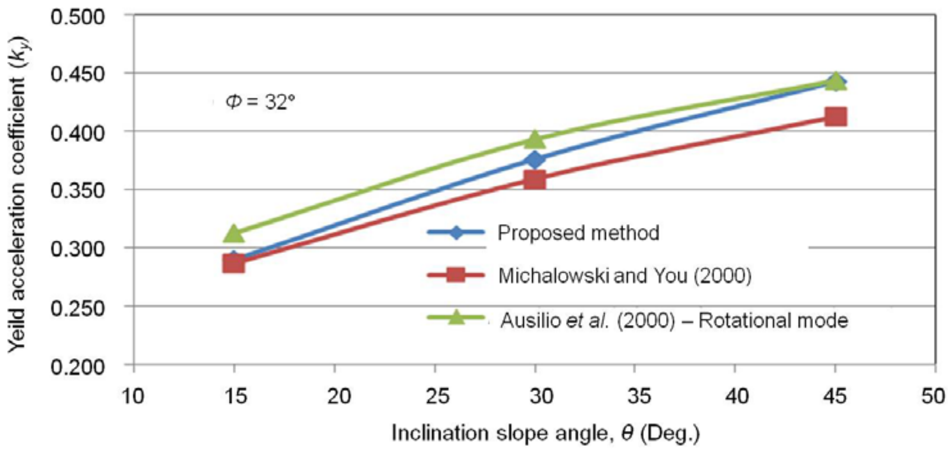

Fig. 16 Comparison of the k_y values obtained from different methods for various angles of slope at $\phi = 32°$

6. Comparison of the deformation values calculated by the suggested method and those of other researchers

In the suggested method after calculating the coefficient of yield acceleration (k_y), by double integration from the difference between yield acceleration and the values of accelerogram, permanent displacements have been determined. In this part, permanent displacements calculated by the suggested method are compared with those of other studies. Figs.17-19 show a comparison between displacements resulted from the suggested method and those reported by Ausilio *et al*. (2000) and Michalowski and You (2000) for different slopes under the Kobe earthquake (Nishi-Akashi 1995). As can be observed, obtained displacements by the current method are smaller than those proposed by Michalowski and You (2000) and are greater than those presented by Ausilio *et al*. (2000). However the present results are closer to the Ausilio *et al*. (2000) method.

On the other hand, by increase in the internal friction angle of soil, the difference between the results obtained by the suggested method and those of Michalowski and You (2000) reduces. Generally, the source of this difference can generate to the difference between the shape of failure wedge and also the method of analysis.

The displacements obtained from various methods are noted in Table 6 for a reinforced slope assuming H = 8 m, θ = 30°, γ = 18 kN/m^3 and for the three cases of ΣT = 135 kN/m, ΣT = 150 kN/m and ΣT = 180 kN/m. These displacements have been calculated for different internal friction angles under the Northridge (Castaic-Old-Ridge Route 1994) earthquake. In this case also the obtained displacements from the suggested method are smaller than those of Michalowski and You (2000) and are greater than those of Ausilio *et al*. (2000). By increase in the internal friction angle of soil, the difference between the results obtained from the suggested method with those of other techniques reduces. However the results are about two times different as an average.

Table 6 Comparison between the permanent displacements (in millimeters) obtained from different methods for a reinforced slope

H = 8 m, θ = 30°, γ = 18 kN/m^3									
	ΣT = 135 kN/m			ΣT = 150 kN/m			ΣT = 180 kN/m		
ϕ (deg.)	Michalowski and You (2000)	Proposed Method	Ausilio *et al.* (2000)	Michalowski and You (2000)	Proposed Method	Ausilio *et al.* (2000)	Michalowski and You (2000)	Proposed Method	Ausilio *et al.* (2000)
28	568	197	123	320	139	74	146	61	55
30	335	133	61	214	71	52	71	43	28
32	219	67	43	136	51	27	48	23	19
34	136	48	22	65	27	17	24	15	12
36	63	26	14	42	19	8	14	7	5
38	39	17	6	19	12	5	6	5	3

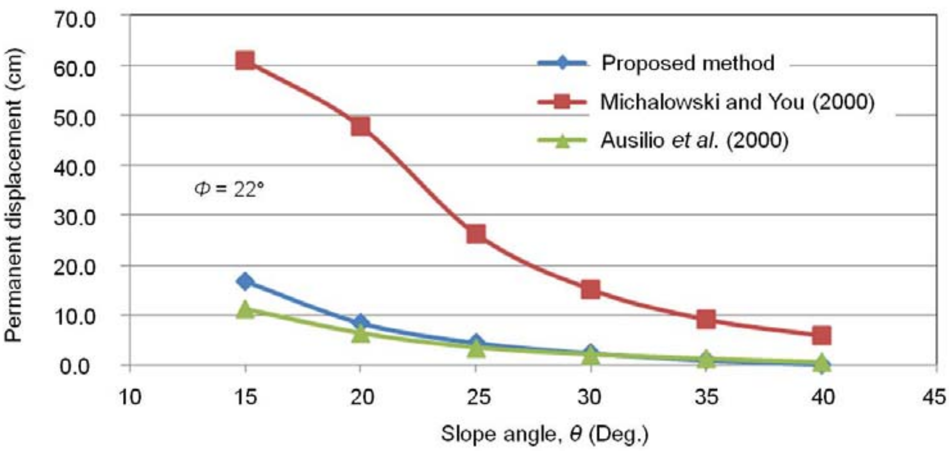

Fig. 17 Comparison of the displacements for the case where H = 5 m, ΣT = 90 kN/m, ϕ=22°

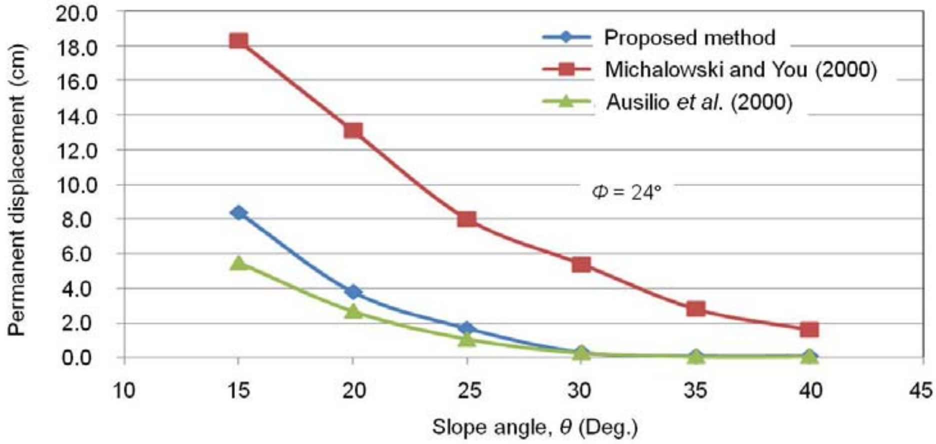

Fig. 18 Comparison of the displacements for the case where $H = 5$ m, $\Sigma T = 90$kN/m, ϕ=24°

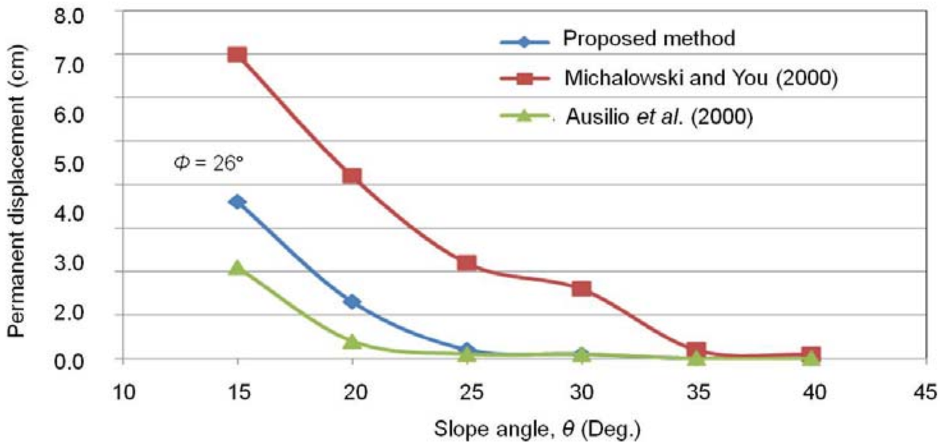

Fig. 19 Comparison of the displacements for the case where $H = 5$ m, $\Sigma T = 90$ kN/m, $\phi = 26°$

7. Conclusions

Calculation of seismic displacements is a very important part in design of reinforced slopes and it has to be done with a reasonable precision particularly in the earthquake prone areas. However the current methods of analysis lead to different displacement values. Therefore, it is necessary that more studies be conducted on this topic.

Proposed method based on limit equilibrium technique is capable of calculating the permanent displacements of reinforced slopes in a simple procedure. In the suggested method, a complete earthquake record needs to be employed for calculation of permanent displacements. Hence among all the available techniques the suggested method has this privilege to include the effect of frequency content of input triggers into the calculations. Comparison between the obtained results from the analysis of a certain reinforced slope under four different earthquakes with close maximum accelerations shows that by varying the accelerogram the permanent displacements also change up to several times.

Comparison between the coefficients of yield acceleration shows that the results obtained by the suggested method for transitional mode are of at most four percent of difference with similar values resulted from Michalowski and You (2000) and Ausilio *et al*. (2000) techniques. However since the failure wedge in the suggested method is considered as planar, there is a considerable difference between the coefficient of yield acceleration obtained from the suggested method and results of Ausilio *et al*. (2000) in rotational mode.

Also a comparison between the permanent displacements shows that different methods give similar trends regarding the decrease of displacements by increase in the internal friction angle. On the other hand, the values from the suggested method fall between the other two analytical techniques. The relation between permanent displacements and tensile strength of reinforcements shows that 10 percent increase in the sum of reinforcing resistance leads to decrease in permanent displacements of the slope to about 30 to 40 percent. Also, 10 degrees of increase in the angle of

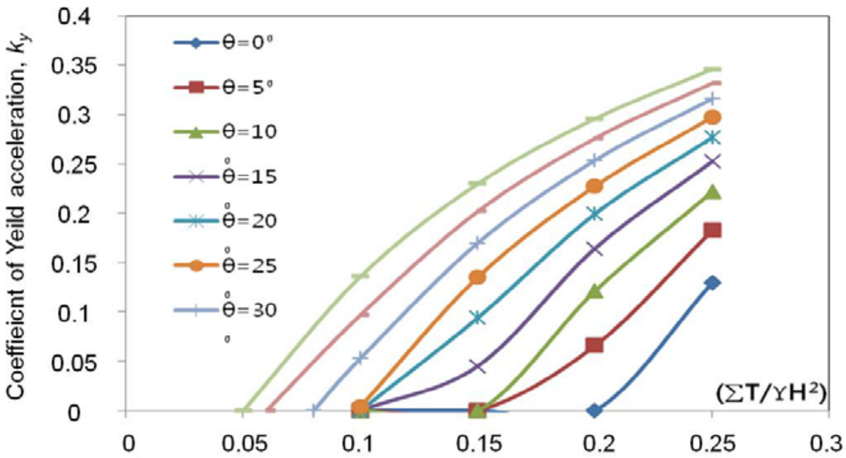

Fig. 20 Design chart for k_y ($\phi = 25°$)

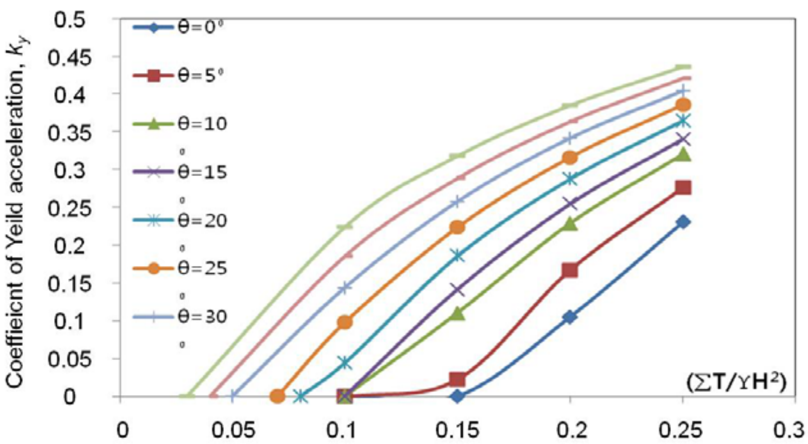

Fig. 21 Design chart for k_y ($\phi = 30°$)

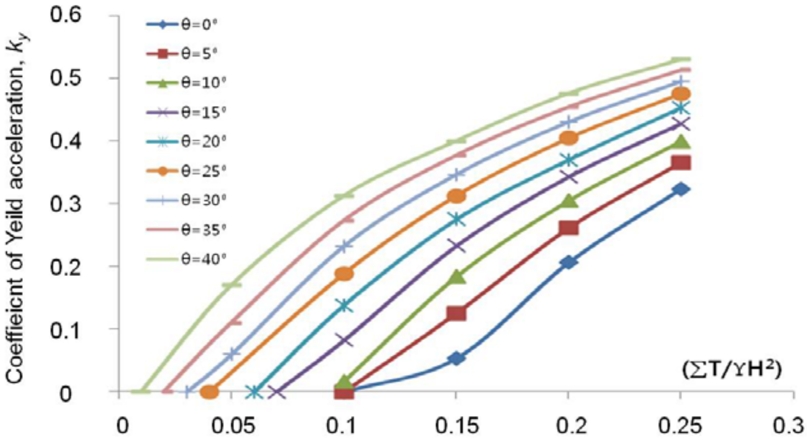

Fig. 22 Design chart for k_y (ϕ = 40°)

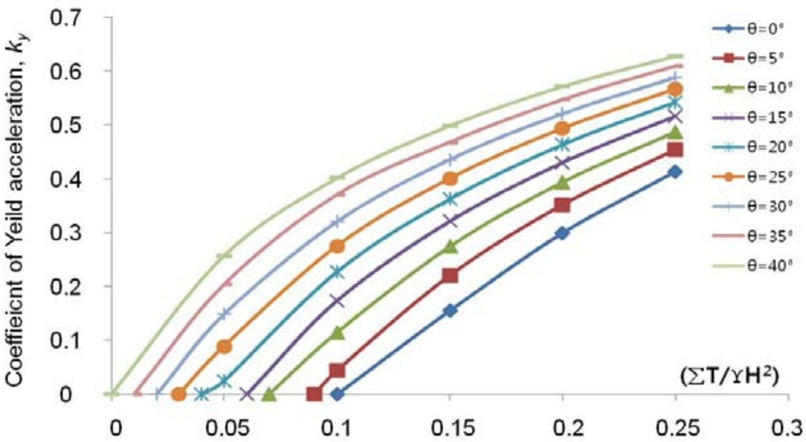

Fig. 23 Design chart for k_y (ϕ = 45°)

reinforced slope (with the vertical axis) causes a reduction in permanent displacements to about 60 to 80 percent.

Finally, this approach of using horizontal slices method is a powerful tool for predicting the permanent displacements of reinforced slopes with a satisfactory precision however it will be necessary that appropriate formulations be developed for various shapes of failure wedges. The design charts based of proposed method are presented in Figs. 20-23.

References

Ahmad, S.M. and Choudhury, D. (2008), “Pseudo-dynamic approach of seismic design for waterfront reinforced soil wall”, *Geotex. Geomemb.*, **26**(4), 291-301.

Ahmadabadi, M. and Ghanbari, A. (2009), “New procedure for active earth pressure calculation in retaining walls with reinforced cohesive-frictional backfill”, *Geotex. Geomemb.*, **27**(6), 456-463.

Al-Hattamleh, O. and Muhunthan, B. (2006), "Numerical procedures for deformation calculations in reinforced soil walls", *Geotex. Geomemb.*, **24**(1), 52-57.

Ausilio, E., Conte, E. and Dente, E. (2000), "Seismic stability analysis of reinforced slopes", *Soil Dyn. Earthq. Eng.*, **19**(3), 159-172.

Baker, R. and Klein, Y. (2004a), "An integrated limiting equilibrium approach for design of reinforced soil retaining structures, Part I: formulation", *Geotex. Geomemb.*, **22**(3), 119-150.

Baker, R. and Klein, Y. (2004b), "An integrated limiting equilibrium approach for design of reinforced soil retaining structures, Part III: optimal design", *Geotex. Geomemb.*, **22**(6), 455-479.

Bathurst, R.J. and Alfaro, M.C. (1996), "Review of seismic design, analysis and performance of geosynthetic reinforced walls, slopes and embankments", *Proceedings of the International Symposium*, Fukuoka, Kyushu, Japan, November.

Bathurst, R.J., Allen, T.M. and Walters, D.L. (2005), "Reinforcement loads in geosynthetic walls and the case for a new working stress design method", *Geotex. Geomemb.*, **23**(4), 287-322.

Cai, Z. and Bathurst, R.J. (1996), "Deterministic sliding block methods for estimating seismic displacements of earth structures", *Soil Dyn. Earthq. Eng.*, **15**(4), 255-268.

Chen, H.T., Hung, W.Y., Chang, C.C., Chen, Y.J. and Lee, C.J. (2007), "Centrifuge modeling test of a geotextile-reinforced wall with a very wet clayey backfill", *Geotex. Geomemb.*, **25**(6), 346-359.

Chen, T.C., Chen, R.H. and Lin, S.S. (2000), "A nonlinear homogenized model applicable to reinforced soil analysis", *Geotex. Geomemb.*, **18**(6), 349-366.

Collin, J.G. (2001), "Lessons learned from a segmental retaining wall failure", *Geotex. Geomemb.*, **19**(7), 445-454.

El-Emam, M.M. and Bathurst, R.J. (2007), "Influence of reinforcement parameters on the seismic response of reduced-scale reinforced soil retaining walls", *Geotex. Geomemb.*, **25**(1), 33-49.

Garg, K.G. (1998), "Retaining wall with reinforced backfill: A case study", *Geotex. Geomemb.*, **16**(3), 135-149.

Ghanbari, A. and Ahmadabadi, M. (2010a), "Pseudo-dynamic active earth pressure analysis of inclined retaining walls using horizontal slices method", Scientia Iranica, *Transact. A*: *Civil Eng.*, **17**(2), 118-130.

Ghanbari, A. and Ahmadabadi, M. (2010b), "Active earth pressure on inclined retaining walls in static and pseudo-static conditions", *Int'l J. Civil Eng.*, **8**(2), 159-173.

Ghanbari, A. and Ahmadabadi, M. (2010c), "New analytical procedure for seismic analysis of reinforced retaining wall with cohesive-frictional backfill", *Geosynth. Int'l*, **17**(6), 364-379.

Ghanbari, A. and Taheri, M. (2012), "An analytical method for calculating active Earth pressure in reinforced retaining walls subject to a line surcharge", *Geotex. Geomemb.*, **34**, 1-10.

Gnanapragasam, G. (2000), "Active earth pressure in cohesive soils with an inclined ground surface", *Can. Geotech. J.*, **37**(1), 171-177.

Hatami, K. and Bathurst, R.J. (2000), "Effect of structural design on fundamental frequency of reinforced-soil retaining walls", *Soil Dyn. Earthq. Eng.*, **19**(3), 137-157.

Huang, C.C. and Wu, S.H. (2006), "Simplified approach for assessing seismic displacements of soil retaining walls, Part I: geosynthetic-reinforced modular block walls", Geosynthetics International, **13**(6), 219-233.

Huang, C.C., Chou, L.H. and Tatsouka, F. (2003), "Seismic displacement of geosynthetic reinforced soil modular block walls", *Geosynth. Int'l*, **10**(1), 2-23.

Huang, C.C. and Wang, W.C. (2005), "Seismic displacement charts for the performance based assessment of reinforced soil walls", *Geosynth. Int'l*, **12**(4), 176-190.

Huang, C.C. and Wu, S.H. (2007), "Simplified approach for assessing seismic displacements of soil retaining walls, Part II: geosynthetic-reinforced walls with rigid panel facing", *Geosynth. Int'l*, **14**(5), 264-276.

Huang, Y., Sawada, K., Moriguchi, S., Yashima, A. and Zhang, F. (2006), "Numerical assessment of the effect of reinforcement on the performance of reinforced soil dikes", *Geotex. Geomemb.*, **24**(3), 169-174.

Jahanandish, M. and Keshavarz, A. (2005), "Seismic bearing capacity of foundations on reinforced soil slopes", *Geotex. Geomemb.*, **23**(1), 1-25.

Kazimierowicz-Frankowska, K. (2005), “A case study of a geosynthetic reinforced wall with wrap-around facing”, *Geotex. Geomemb.*, **23**(1), 107-115.
Kramer, S.L. and Paulsen, S.B. (2004), “Seismic performance evaluation of reinforced slopes”, *Geosynth. Int’l*, **11**(6), 429-438.
Latha, G.M. and Krishna, A.M. (2008), “Seismic response of reinforced soil retaining wall models: influence of backfill relative density”, *Geotex. Geomemb.*, **26**(4), 335-349.
Lee, K.Z.Z. and Wu, J.T.H. (2004), “A synthesis of case histories on GRS bridge-supporting structures with flexible facing”, *Geotex. Geomemb.*, **22**(4), 181-204.
Ling, H.I., Leshchinsky, D. and Chou, N.N.S. (2001), “Post-earthquake investigation on several geosynthetic-reinforced soil retaining walls and slopes during 1999 Ji-Ji earthquake of Taiwan”, *Soil Dyn. Earthq. Eng.*, **21**(4), 297-313.
Ling, H.I., Leshchinsky, D. and Perry, E.B. (1997), “Seismic design and performance of geosynthetic - reinforced soil structures”, *Geotech.*, **47**(5), 933-952.
Michalowski, R.L. and You, L. (2000), “Displacement of reinforced slopes subjected to seismic loads”, *J. Geotech. Geoenviron. Eng.*, **126**(8), 685-694.
Mojallal, M. and Ghanbari, A. (2012), “Prediction of seismic displacements in gravity retaining walls based on limit analysis approach”, *Struct. Mech. Int’l J.*, **42**(2), 247-267.
Mojallal, M., Ghanbari, A. and Askari, F. (2012), “A new analytical method for calculating seismic displacements in reinforced retaining walls”, *Geosynth. Int’l*, **19**(3), 212-231.
Newmark, N.M. (1965), “Effects of earthquakes on dams and embankments”, *Geotech.*, **15**(2), 139-159.
Nouri, H., Fakher, A. and Jones, C.J.F.P. (2006), “Development of horizontal slices method for seismic stability analysis of reinforced slopes and walls”, *Geotex. Geomemb.*, **24**, 175-187.
Nouri, H., Fakher, A. and Jones, C.J.F.P. (2008), “Evaluating the effects of the magnitude and amplification of pseudo-static acceleration on reinforced soil slopes and walls using the limit equilibrium horizontal slices method”, *Geotex. Geomemb.*, **26**(3), 263-278.
Porbaha, A., Zhao, A., Kobayashi, M. and Kishida, T. (2000), “Upper bound estimate of scaled reinforced soil retaining walls”, *Geotex. Geomemb.*, **18**(6), 403-413.
Reddy, G.V.N., Madhav, M.R. and Reddy, E.S. (2008), “Pseudo-static seismic analysis of reinforced soil wall: effect of oblique displacement”, *Geotex. Geomemb.*, **26**(5), 393-403.
Richards, R. and Elms, D.G. (1979), “Seismic behavior of gravity retaining walls”, *J. Geotech. Eng., ASCE*, **105**(4), 449-469.
Rowe, R.K. and Skinner, G.D. (2001), “Numerical analysis of geosynthetic reinforced retaining wall constructed on a layered soil foundation”, *Geotex. Geomemb.*, **19**(7), 387-412.
Sabermahani, M., Ghalandarzadeh A. and Fakher A. (2008), “Experimental study on seismic deformation modes of reinforced-soil walls”, *Geotex. Geomemb.*, **27**, 121-136.
Sandri, D. (1997), “A summary of reinforced soil structures performance in the greater Los Angeles area after the Northridge earthquake”, Wu, J.T.H. (Ed.), *Mech. Stabil. Backfill*, Balkema, Rotterdam, 433-442.
Shahgholi, M. (1999), “Design of reinforced soil walls against earthquake”, M.Sc. Thesis, University of Tehran, Tehran, Iran.
Shekarian, S., Ghanbari, A. and Farhadi, A. (2008), “New seismic parameters in the analysis of retaining walls with reinforced backfill”, *Geotex. Geomemb.*, **26**(4), 350-356. 0266-1144.
Skinner, G.D. and Rowe, R.K. (2005), “Design and behavior of a geosynthetic reinforced retaining wall and bridge abutment on a yielding foundation”, *Geotex. Geomemb.*, **23**(3), 234-260.
Tatsuoka, F., Koseki, J. and Tateyama, M. (1997), “Performance of reinforced soil structures during the 1995 Hyogo-ken Nambu earthquake”, Ochiai, H., Yasufuku, N. and Omine, K. (Eds.), *Earth Reinforce. Balkema*, Rotterdam, 979-1008.
Tatsuoka, F., Tateyama, M. and Koseki, J. (1995), “Behavior of geogrid-reinforced soil retaining walls during the great Hanshin-Awaji earthquake”, *Proceeding of the 1st International Symposium of Earthquake Geotechnical Engineering*, **19**(2), 55-60.
White, D. and Holtz, R.D. (1997), “Performance of geosynthetic reinforced slopes and walls during the Northridge California earthquake of January 17, 1994”, Ochiai, H., Yasufuku, N. and Omine, K. (Eds.),

Earth Reinforce. *Balkema*, Rotterdam, 965-972.
Won, M.S. and Kim, Y.S. (2007), "Internal deformation behavior of geosynthetic-reinforced soil walls", *Geotex*. *Geomemb*., **25**(1), 10-22.
Yoo, C. (2004), "Performance of a 6-year-old geosynthetic-reinforced segmental retaining wall", *Geotex*. *Geomemb*., **22**(5), 377-397.
Yoo, C. and Jung, H.S. (2004), "Measured behavior of a geosynthetic-reinforced segmental retaining wall in a tiered configuration", *Geotex*. *Geomemb*., **22**(5), 359-376.
Zarnani, S., El-Emam, M.M. and Bathurst, R.J. (2011), "Comparison of numerical and analytical solutions for reinforced soil wall shaking table tests", *Geomech*. *Eng*., **3**(4), 291-321.
Zornberg, J.G., Sitar, N. and Mitchell, J.K. (1998), "Limit equilibrium as basis for design of geosynthetic reinforced slopes", *J. Geotech*. *Geoenviron*. *Eng*., *ASCE*, **124**(8), 684-698.

PL

Nomenclature

H_i	Horizontal force at top of ith slice (kN/m)
H_{i+1}	Horizontal force at bottom of ith slice (kN/m)
h_i	Height of ith slice (m)
N_i	Normal force on failure surface for ith slice (kN/m)
n	Numer of horizontal slices (dimensionless)
S_i	Shear force on failure surface for ith slice (kN/m)
k_y	The yeild coefficient of seismic acceleration(dimensionless)
T_i	Tensile force in reinforcement for ith slice (kN/m)
ΣT_i	Sum of tensile force in the reinforcements (kN/m)
V_i	Normal force at top of ith slice (kN/m)
V_{i+1}	Normal force at bottom of ith slice (kN/m)
W_i	Weight of ith slice (kN/m)
X_{Vi}	Horizontal distance of V_i from toe of slope (m)
X_{Vi+1}	Horizontal distance of V_{i+1} from toe of slope (m)
$\bar{X}_i$	Horizontal distance of the center of gravity from toe of slope (m)
$\bar{Y}_i$	Vertical distance of the center of gravity from toe of slope (m)
ϕ	Angle of internal friction of soil (degrees)
β	Angle between failure surface and horizontal plan (degrees)
θ	Angle between slope and vertical plan (degrees)
γ	Total unit weight (kN/m^3)