Non-Linearity in the Seismic Design of Reinforced Earth

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SUMMARY:
The use of geosynthetic reinforced structures in seismic areas is becoming very widespread, being these structures able to store large deformations without collapsing. This research refers the results of some numerical modelling of steep geosynthetic reinforced embankments subjected to different earthquakes. Depending on the degree of internal and external fixity steep reinforced embankments may have two types of failures: the internal failure and the external failure. The aptitude of the reinforced earth to slide was verified with the displacement method outlined by Newmark and founded on the dynamic equilibrium of a sliding block provided of a Coulomb friction. Only the direct sliding mechanism was investigated by considering the critical acceleration that was evaluated with limit equilibrium and limit analysis. By comparing the FEM seismic results with the pseudo-static previsions, a reduction factor of the peak ground acceleration of the design earthquake was outlined, with the purpose of introducing the ductile behaviour of the structure within the pseudo-static approach.

Keywords: FEM seismic design, steep geosynthetic reinforced earth, Newmark sliding block.

1. INTRODUCTION

The seismic design of structures, based on their expected performance, is becoming very wide-spread in civil engineering, as it is reasonable to accept different levels of damage in relation to the importance of the structure and to the return period of the design earthquake. Damage may be ascribed to an *ultimate limit state*, in general related to a loss of equilibrium of the structure, or to a *serviceability limit state*, that concerns the loss of functionality caused by excessive displacements. In the general landscape of technical rules, this approach is still little codified for composite and mixed works, such as reinforced earth with geosynthetics. The problem is not easy because in earth work several failure mechanisms may activate in relation to the geometry and the strength parameters. As a consequence, serviceability and ultimate limit analyses are carried out with very different procedures, without regards for the priority and the significance of a given limit state.
The aim of this research is that of highlighting the analysis reliability for ultimate and serviceability limit states in steep geosynthetic reinforced earth structures.
Generally speaking, the seismic design of earth structures can be carried out according to three different approaches, gradually increasing in complexity, and with purposes quite different to each other.
The most common approach is the pseudo-static (Okabe 1924; Mononobe & Matsuo 1929), in which dynamic effects are introduced in terms of additional static forces acting on some parts of the structure supposed rigid and, at the same time, in a plastic equilibrium. Although this method is highly practical, it does not consider some important aspects in the dynamic response of continuum, such as: soil deformability and damping, the time history of the seismic event, soil non-linearity before failure and the soil displacements during plastic equilibrium. For these reasons, the pseudo-static approach is generally employed in the evaluation of the ultimate limit equilibrium once the failure mechanism is already identified.
Following the displacement method (Newmark 1965, Richards & Elms 1979, Whitman & Liao 1984,

Maugeri & Rigano 2011), the structure is studied as a generalized Newmark sliding block. When the earthquake begins, the mechanism will only activate for acceleration greater than the threshold acceleration (also called critical acceleration). Thereafter, displacements can occur only for every overcoming of the critical acceleration. The displacement method implies that a discontinuous mechanism is already developed inside the structure and therefore only rigid displacements may be evaluated. This approach is often preferred for the post-seismic serviceability limit state verification.
The complete dynamic approach can be carried out by numerical modelling (Carrubba & Brusarosco 2007, 2008), to properly describe the state of stress and strain at every point of the structure. With this approach, both the time-history of the seismic event and soil non-linearity can be taken into account. The complete behaviour of the structure, at each instant of the seismic history, is known, thus allowing any form of interaction to be analyzed. However, in this case, it is necessary to select with care a large number of mechanical parameters, both static and dynamic, and to have some experience in the management of both the mesh-depending effects and the boundary effects during dynamic loading. When large displacements are expected, numerical strategies are necessary, such as the introduction of interfaces along the most probable sliding surfaces.
Despite their different capabilities, all the aforesaid methods are routinely used without a proper distinction about the levels of accuracy in forecasting.
In order to study the seismic performance of steep geosynthetic reinforced embankments without facing, all the previous three approaches were used. Four historical design earthquakes were used in the Newmark analysis with the aim of analyzing the induced post-seismic displacements. For this purpose, only the direct sliding mechanism, upon the lower reinforcement, was investigated by introducing the concept of critical acceleration; the latter was evaluated by means of both the limit equilibrium and the limit analysis.
Thereafter, FEM analyses were carried out with the aim of quantifying the reduction factors of the peak ground acceleration of the design earthquake (AGI, 2005; NTC, 2008; EUROCODE 8, 2005). These reduction factors (Carrubba & Brusarosco, 2011, 2012), coming from the comparison between FEM and pseudo-static analyses, took into account the global ductile performance of the structure and should be used in the pseudo-static approach for the verification of the ultimate limit state.

2. THE DISPLACEMENT APPROACH

During earthquake, the ground acceleration may excess the value corresponding to the limit equilibrium of the reinforced earth with respect to a given failure mode. Even if this excess of acceleration happens only for very short time, it may repeat many times and would be capable of inducing cumulated displacements of some parts of the earth work. This unfavourable circumstance may be reviewed as design criteria: instead of strengthening the structure to resist to the maximum design acceleration, permanent displacements may be planned in order to built a more ductile structure. Therefore, the method may be used for the verification of the serviceability limit state in a post-seismic condition. The displacement approach is founded on the sliding block concept outlined by Newmark (1965). When applying this method, should not be forgotten that the structure is supposed to be a set of rigid blocks separated by rigid-plastic sliding surfaces.
Calling a_{crit} the acceleration approaching to a limiting equilibrium of the block resting on the horizontal ground, and $a_g(t)$ the design ground acceleration, any excess of the ground acceleration, above the critical value, produces a relative displacements x_{rel} of the block respect to the ground, according to the following equilibrium equation:

$$\ddot{x}_{rel}(t) = a_g(t) - a_{crit} \tag{2.1}$$

and by double integration:

$$x_{rel} = \iint \left[K_g(t) - K_{crit} \right] g \; dt \tag{2.2}$$

having expressed the accelerations in terms of seismic coefficients:

$$a_{crit} = K_{crit}\ g$$
$$a_g(t) = K_g(t)\ g \qquad (2.3)$$

As shown by equation 2.2 the relative displacement is only function of the critical acceleration; the latter may be considered quite a constant related to the geometry and the shear strength of the earth work. The critical acceleration may be evaluated under different hypotheses, by limit equilibrium (Wong 1982; Bathurst & Cai 1995; Bathurst & Alfaro 1997; Cai & Bathurst, 1996) or by limit analysis (Michalowski & You 2000; Ausilio et al. 2000).

Following the limit equilibrium, the horizontal component of the active seismic earth pressure, behind the reinforced nucleus, is evaluated by the Mononobe & Okabe approach. Neglecting the vertical seismic coefficient (K_V), the function describing the equilibrium of horizontal forces may be written in terms of horizontal seismic coefficient (K_H); this latter becomes minimum ($K_{H,crit}$) when the sliding of the reinforced nucleus in incipient: under these conditions, the critical seismic coefficient is only function of the critical inclination of the failure plane (α_{crit}):

$$K_{H,crit} = \frac{W_{nucleus} \tan\varphi_{base} - W_{soil} \dfrac{\cos(\varphi-\psi)\tan(\alpha_{crit}-\varphi)}{\tan(\psi-\varphi)\cdot\tan(\alpha_{crit}-\varphi)}}{W_{nucleus} + W_{soil} \dfrac{\cos(\varphi-\psi)}{\tan(\psi-\varphi)\cdot\tan(\alpha_{crit}-\varphi)}} \qquad (2.4)$$

in which φ= soil shear strength angle, φ_{base}=shear strength angle at the base of the reinforced nucleus, ψ=face slope angle respect to the vertical, $W_{nucleus}$=weight of the reinforced nucleus, W_{soil}=weight of the driving soil.

Following the limit analysis, both the reinforced nucleus and the driving soil wedge must satisfy the kinematic compatibility, which implies that the displacement at the interfaces must occur according to an associated flow rule. Also in this case, by neglecting the vertical seismic coefficient, the function describing the equilibrium of the works may be written in terms of horizontal seismic coefficient, which becomes critical when the sliding of the reinforced nucleus in incipient: under these conditions, the critical seismic coefficient is only function of the critical inclination of the failure plane (α_{crit}):

$$K_{H,crit} = \frac{W_{nucleus} \sin\varphi_{base} - W_{soil} \sin(\alpha_{crit}-\varphi)\ R}{W_{soil}\ R \cos(\alpha_{crit}-\varphi) + W_{nucleus} \cos\varphi_{base}} \qquad (2.5)$$

in which:

$$R = \frac{\mathrm{sen}(90-\psi+\varphi+\varphi_{base})}{\mathrm{sen}\ (90-\psi+2\varphi-\alpha_{crit})} \qquad (2.6)$$

In this paper an evaluation of the permanent displacements was carried out for the model of reinforced earth showed in Figure 2.1. The following length of reinforcements: $L=3m$, $L=4m$ and $L=5m$, were considered in order to highlight the effect of the reinforcement length under the direct sliding mode of failure. The height of the model was $H=10m$, the unit weight $\gamma=20kN/m^3$, the slope angle of the facing $\psi=6°$, the soil shear strength angle $\varphi°=35°$ and $\varphi°=45°$ and the shear strength angle at the base of the reinforced nucleus $\tan\varphi_{base}=0.8\tan\varphi$.

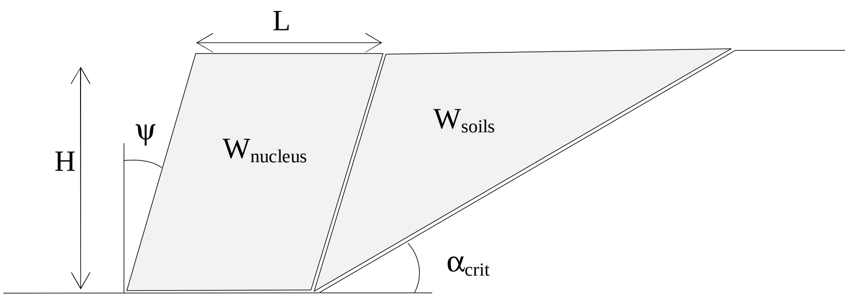

Figure 2.1. Steep geosynthetic reinforced embankments for the sliding block analysis

The critical seismic coefficient was evaluated under both the limit equilibrium and the limit analysis hypotheses, that is, without considering dilatancy of the sliding interfaces or considering a dilatancy equal to the angle of shear strength. Only the effect of horizontal acceleration, in terms of horizontal seismic coefficient K_H, was considered. The results of these analyses are resumed in Table 2.1.

Table 2.1 Evaluation of the critical seismic coefficient of the reinforced earth model for direct sliding failure mode under only horizontal acceleration.

	$K_{H,crit}$			
	Limit equilibrium		Limit analysis	
	φ=35°	φ=45°	φ=35°	φ=45°
L=3m	0,09	0,27	0,22	0,50
L=4m	0,15	0,33	0,26	0,55
L=5m	0,19	0,37	0,29	0,58

As expected, the limit analysis gave a critical seismic coefficient almost double respect to the limit equilibrium approach; in fact, the presence of dilatancy along the sliding interfaces was able in introducing more restraints to the horizontal displacements.
The block model of Figure 2.1 was subjected to the Newmark analysis by considering the historical earthquakes of Friuli (1976), Imperial Valley (1940), Loma Prieta (1989) and Kobe (1995). The characteristics of these seismic events were deduced by the Pacific Earthquake Engineering Research Center database (http://peer.berkeley.edu/smcat/index.html) and reported in Table 2.2.
A gradually increasing energy content was highlighted by the Arias intensity (Arias 1970) moving from the Tolmezzo record toward the Kjma one.

Table 2.2 Kinematic characteristics of the selected seismic events from the PEER database

Earthquake	Friuli Italy (06.05.76)	Imperial Valley California (19.06.40)	Loma Prieta California (18.10.89)	Kobe Japan (16.01.95)
Recording station	Tolmezzo	El Centro	Corralitos	Kjma
PEER Rec. ID	P0126	P0006	0745	P1043
Comp.	A-TMZ000	I-ELC180	CLS000	KJM000
Mom. Mag.	6.5	7.0	6.9	6.9
PGA (g)	0.351	0.313	0.644	0.820
PGV (cm/s)	22.04	29.70	55.18	81.30
PGD (cm)	4.11	13.04	10.75	17.69
Sampling time (s)	0.005	0.010	0.005	0.020
Arias Int. (m/s)	0.78	1.70	3.24	8.39

The maximum post-seismic displacements of the reinforced earth are reported in Table 2.3; as expected the limit equilibrium approach gave always more large displacements respect to the limit analysis; these displacements, however, were strongly dependent on the soil friction angle. Referring to the limit equilibrium results, the less firm structure (L=3m and φ=35°) subjected to the Kobe earthquake gave a maximum displacement of about 108 cm; under the same conditions, by only changing the soil friction angle (L=3m and φ=45°) the displacement reduced to about 13 cm.

Analogous considerations may be advanced for the limit analysis approach, but in this case the displacement of the less firm structure (L=3m and φ=35°) would have been about 23 cm and by changing the soil friction angle (L=3m and φ=45°), it would have reduced to nearly zero. Therefore, a considerable practical aspect in the seismic displacement prevision is the correct evaluation of soil strength, included the aptitude to dilate.Equation (2.2) was integrated by many authors (Richards and Elms, 1979; Whitman and Liao, 1984; Yegian et al. 1991) with reference to records coming from many seismic regions. It was found that the post-seismic displacement ($x_{rel,max}$) may be forecasted only on the basis of a few kinematic parameters of the seismic event such as the peak ground displacement *(PGA)* and the peak ground velocity *(PGV)*. For example, Whitman and Liao (1984) predicted the permanent displacement over inclined plane by means of the following expression:

$$x_{rel,max} = A\left(\frac{PGV^2}{PGA}\right)e^{\left(B\frac{a_{crit}}{PGA}\right)} \tag{2.7}$$

in which A = 37 and B = –9.4 are two constants evaluated by the authors via regression analysis, for earthquakes of magnitude varying between 6.3 and 6.7.

Table 2.3 Maximum post-seismic displacement of the reinforced earth model using the Newmark approach

		$x_{rel,max}$ (cm)			
		φ=35°		φ=45°	
Earthquake	L(m)	Limit Equilibrium	Limit analysis	Limit Equilibrium	Limit analysis
Friuli (Tolmezzo)	3	5,0	0,3	0,1	0,0
	4	1,6	0,1	0,0	0,0
	5	0,7	0,0	0,0	0,0
Imperial Valley (El Centro)	3	13,7	0,1	0,0	0,0
	4	1,3	0,0	0,0	0,0
	5	0,3	0,0	0,0	0,0
Loma Prieta (Corralitos)	3	36,3	5,1	3,5	0,4
	4	12,5	3,8	2,3	0,2
	5	6,9	3,0	1,7	0,1
Kobe (Kjma)	3	108,5	22,7	13,4	0,3
	4	52,2	15,0	6,7	0,0
	5	32,5	10,3	3,7	0,0

Table 2.4 Comparisons between the maximum post-seismic displacements of the reinforced earth model using the Newmark approach and the Whitman and Liao (1984) formula by evaluating the critical seismic coefficient by means of the limit equilibrium.

		$x_{rel,max}$ (cm) (*by limit equilibrium*)			
		φ′=35°		φ′=45°	
Earthquake	L(m)	Whitman e Liao (1984)	Newmark (1965)	Whitman e Liao (1984)	Newmark (1965)
Friuli (Tolmezzo)	3	4,7	5,0	0,0	0,1
	4	0,9	1,6	0,0	0,0
	5	0,3	0,7	0,0	0,0
Imperial Valley (El Centro)	3	7,1	13,7	0,0	0,0
	4	1,2	1,3	0,0	0,0
	5	0,4	0,3	0,0	0,0
Loma Prieta (Corralitos)	3	47,9	36,3	3,5	3,5
	4	20,0	12,5	1,4	2,3
	5	11,1	6,9	0,8	1,7
Kobe (Kjma)	3	108,4	108,5	13,8	13,4
	4	54,5	52,2	6,9	6,7
	5	34,4	32,5	4,4	3,7

A comparison between the prevision of equation (2.4) and the results of the Newmark analysis is reported in Table 2.4 for the case of critical seismic coefficient evaluated with the limit equilibrium and in Table 2.5 for the case of critical seismic coefficient evaluated with the limit analysis.

Table 2.5 Comparisons between the maximum post-seismic displacements of the reinforced earth model using the Newmark approach and the Whitman and Liao (1984) formula by evaluating the critical seismic coefficient by means of the limit analysis.

		$x_{rel,max}$ (cm) (*by limit analysis*)			
		φ′=35°		φ′=45°	
Earthquake	L(m)	Whitman e Liao (1984)	Newmark (1965)	Whitman e Liao (1984)	Newmark (1965)
Friuli (Tolmezzo)	3	0,1	0,3	0,0	0,0
	4	0,0	0,1	0,0	0,0
	5	0,0	0,0	0,0	0,0
Imperial Valley (El Centro)	3	0,1	0,1	0,0	0,0
	4	0,0	0,0	0,0	0,0
	5	0,0	0,0	0,0	0,0
Loma Prieta (Corralitos)	3	7,2	5,1	0,1	0,4
	4	4,0	3,8	0,1	0,2
	5	2,6	3,0	0,0	0,1
Kobe (Kjma)	3	24,4	22,7	1,0	0,3
	4	15,4	15,0	0,6	0,0
	5	10,9	0,3	0,4	0,0

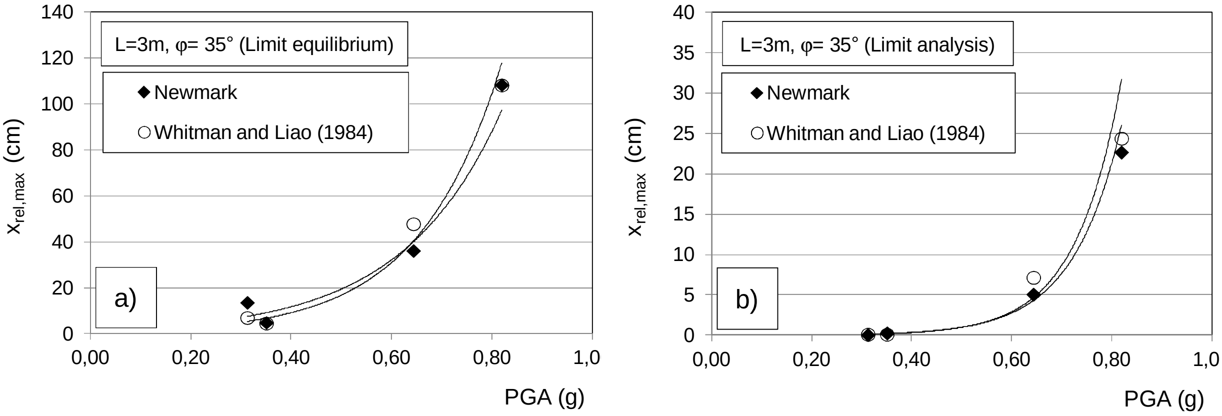

Figure 2.2 Comparisons between the obtained final displacements using Whitman and Liao (1984) and the sliding Newmark block approaches for the less firm structure: a) critical acceleration evaluated by limit equilibrium, b) critical acceleration evaluated by limit analysis.

3. THE FEM APPROACH

Numerical modelling was used in this study to quantify the reduction factor of the peak ground acceleration (PGA) of the design earthquake (NTC 2008). If the global ductile behaviour of the structure is not taken into account, the selected pseudo-static acceleration is quite arbitrary and could lead to inappropriate results. The study was limited to the cases which could be analyzed with the Mononobe & Okabe method.

The reference structure had a height of H = 10 m and a slope of 84° in respect to the horizontal. Various numerical models were developed in order to investigate the effects of spacing and length of the reinforcements (Carrubba & Brusarosco, 2011, 2012; Carrubba et al 2012). Two spacings, P = 0.5 m and P = 1 m, together with two lengths, L=5 m and L=10 m, were considered. In the case of P = 0.5 m, the extensional stiffness of the reinforcement was assumed to be E_rA_r = 1000 kN/m, while in the case of P = 1 m it was assumed to be double, thus giving a constant rigidity to the reinforced mass.

The two lengths of the reinforcements allowed possible changes in the failure mechanisms to be highlighted, depending on the extent of the reinforcements themselves.
The soil, having two possible effective shear strength angles (φ'=35° and φ'=45°), had only one density (ρ=2000 kg_m/m^3), as well as only one elastic modulus (E_s=50000 kPa), one Poisson ratio (ν=0.25). These assumptions allowed the influence of the shear strength to be highlighted with respect to the global ductility of the models.
All the models were provided with interface elements, to localize any possible slippage and pullout along the soil-reinforcement discontinuity. The interface fiction angle δ was assumed to be in a fixed proportion in respect to the soil friction angle ($\tan\delta/\tan\varphi$=0.80).
The continuous removal of energy during earthquake was guaranteed by soil damping, both hysteretic and viscous. Hysteretic damping was provided by soil non-linearity, while viscous damping was introduced in terms of Rayleigh α and β constants.
The results of the totality of numerical analyses, carried out with the selected earthquakes, allowed us to conclude that when the energy of the earthquake was moderate, the seismic tensile loads provided by the FEM analyses were a little less than those of Mononobe & Okabe, while behind the reinforced nucleus the FEM soil pressures were generally greater than those of Mononobe & Okabe, due to the fact that in this case the soil pressure never completely reached an active state.
As seismic energy increased, the Mononobe & Okabe approach gave higher values of both the tensile load in the reinforcements and of the soil pressure behind the reinforced nucleus, with respect to the FEM results.
To describe such a situation, the nominal seismic coefficient of the expected earthquake:

$$K_h = \frac{PGA}{g} \tag{3.1}$$

was modified to take into account the ductile behaviour of the reinforced earth when using the pseudo-static approach of Mononobe & Okabe (NTC, 2008). Therefore, a reduction factor (β) of the expected PGA was introduced to obtain a fictitious seismic coefficient *(K_h*)*:

$$K_h^* = \beta \frac{PGA}{g} \tag{3.2}$$

By considering the post-seismic FEM thrust as generated by the Mononobe & Okabe formula, the fictitious seismic coefficient was deduced by comparison:

$$P_{dyn,FEM} = P_{dyn,MO}\,(K_h^*) \tag{3.3}$$

where: $P_{dyn,FEM}$ = the total seismic thrust in the reinforced nucleus provided by the FEM analysis, taking non-linearity into account; $P_{dyn,MO}$ (K_h^*) = the pseudo-static total thrust in the reinforced nucleus as provided by the Mononobe & Okabe formula by introducing the fictitious seismic coefficient (K_h^*).
Therefore the reduced factor of the PGA was evaluated by the ratio:

$$\beta = \frac{K_h^*}{K_h} \tag{3.4}$$

In order to verify the external stability of the reinforced mass, an approach similar to the previous one was proposed (Carrubba & Brusarosco, 2011, 2012) to appraise the reduction factor β_1 of the pseudo-static total thrust behind it.
The obtained reduction factors, β and β_1, are shown in Figure 3.1 with respect to the *PGA* of the reference design earthquake. Only two extreme conditions are represented in this picture: the stiffest reinforced embankment (L=10 m, P=0.5 m and φ'=45°) and the most ductile one (L=5 m, P=1 m and φ'=35°).

As expected, the strongest Loma earthquake (PGA≅0,644g) affecting the most ductile structure, involved reduction factors, β and β_1, of about 0.5 and 0.6 respectively. Therefore, both internal and external stability evaluations could be performed by means of the Mononobe and Okabe formula, but considering a reduced PGA of the reference earthquake as suggested with Equation (3.2).
Otherwise, in stiff models, independently of the earthquake intensity, the β factor reached values greater than 0.8; this means that the internal verification of the reinforcements, by using the Mononobe and Okabe formula, requires nearly the full value of the PGA of the design earthquake, with the aim of fitting the FEM results. In these stiff models the β_1 factor generally reached values greater than unit: this is due to the confining effect induced by a thick reinforced nucleus, which prevents the soil lying behind the reinforced embankment from reaching an active state of stress.
Finally, even if ductility was able in reducing the design PGA - to be considered in the Mononobe & Okabe formula - on the other hand remarkable deformations occurred under the strongest earthquake here considered.

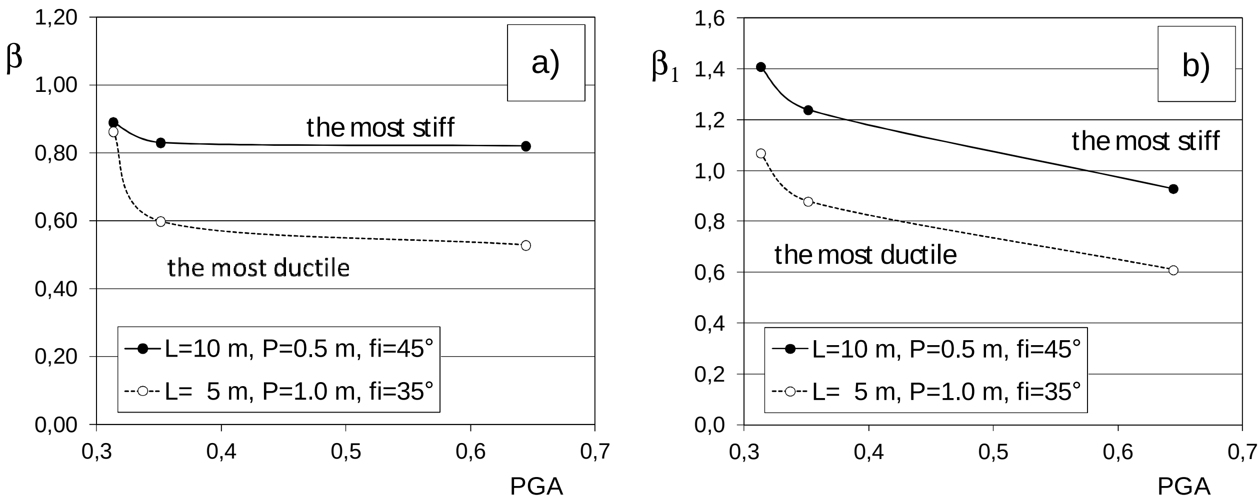

Figure 3.1 The reduction factors of PGA for the stiffest and the most ductile reinforced embankments: a) reduction factor for the thrust localised inside the reinforced nucleus, b) reduction factor for the thrust localised behind the reinforced nucleus.

CONCLUSIONS

The seismic design of earth structures is currently carried out by means of the pseudo-static approach, in which dynamic effects are introduced in terms of static forces. Although this method is very practical and suggested by many codes, it does not take into consideration important aspects of the dynamical behaviour of the soil continuum. Within the range of applicability of the Mononobe & Okabe formula, a reduction factor of the seismic coefficient was proposed (Carrubba & Brusarosco, 2011, 2012) in order to achieve a Mononobe & Okabe prediction of the active thrust comparable with that given by a dynamic complete approach.
In the case of strong earthquakes, the Mononobe & Okabe seismic thrust, computed with the pertinent *PGA* of a design earthquake, could be overestimated in the more ductile structure. To this purpose, reduction factors, β and β_1 for the thrust localised inside and behind the reinforced nucleus, were found to be about 0.5 and 0.6 respectively. Therefore, both external and internal stability verifications can be carried out with the Mononobe & Okabe approach, by considering the seismic coefficients, as suggested with the equation (6).
Otherwise, in stiff embankments, more than *80%* of the *PGA* of the design earthquake is necessary for the safe pseudo-static stability verification, both internal and external, independently of the earthquake intensity.
Referring to the displacement method, it was possible to verify that the expression of Whitman and Liao (1984) is capable of giving a good prevision of the sliding Newmark block for both the procedure

used in the evaluating of the critical acceleration.

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