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COMPARISON OF SEISMIC SLOPE STABILITY ASSESSMENT METHODS FOR REINFORCED GEOSTRUCTURES

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Abstract - ***The construction of geosynthetically reinforced earth structures is constantly developing over the last decades. Seismic design of unreinforced and reinforced embankments and slopes is of extreme importance, due to the environmental and economical consequences related to a potential failure. The main factors that can cause instability are those which tend to increase the shear stresses that are developed in the soil and/or to decrease its shear resistance. These phenomena become more pronounced when dynamic loads are applied to earth structures, such as those due to earthquakes. For this purpose, this work investigates the dynamic distress of reinforced soil structures due to seismic wave propagation. Current seismic design procedures of such technical works involve an amended version of static design methods. Hence, the seismic design of these structures does not account for several important factors, like the compound failure and the global instability which were observed in damaged structures after post-earthquake investigation. The aim of the current study is to assess the dynamic response of reinforced soil structures via different methodologies and to compare the most commonly used approaches for their seismic slope stability assessment.***

Key Words: *Seismic slope stability, geosynthetics reinforcement, Pseudostatic method, Sliding-block method, Permanent deformations, Finite-element analyses.*

1. INTRODUCTION

Geosynthetics are advanced materials, made from various types of polymers, and are used in many environmental, transportation, hydraulic, geotechnical and other engineering applications. Geosynthetics have been widely used over the last decades in various fields of engineering practice, mainly due to the numerous functions that they can be efficiently applied which are reinforcement, drainage, filtration, containment, and separation. The development of these applications is still ongoing, not only on the evaluation of their performance, but also on the application field of these very efficient engineering materials. Geosynthetically reinforcement of soil is a very popular technique used to stabilize slopes [1-10]. The main advantages are the increased stability of earth slopes and embankments against static and dynamic loads, such as those due to seismic motions. Nevertheless, recent earthquakes, such as the 1999 Kocaeli and the 1999 Chi-Chi earthquakes (Figure 1), have demonstrated the seismic vulnerability of mechanically stabilized earth walls and reinforced slopes.

Fig -1: Side view of reinforced slope failure due to Chi-Chi (Taiwan, 1999) earthquake

The most common approach for the analysis of the seismic stability of reinforced earth structures with geosynthetic based on the pseudostatic approach, where the seismic forces are equal by multiplying the seismic coefficient and the weight of the sliding mass. The (both static and seismic) design of geosynthetic reinforced slopes is usually based on modified versions of classical limit equilibrium slope stability methods. Kinematically, the potential failure surface in a reinforced homogenous slope is assumed typically to be defined by the same idealized geometry (but not location) as in the unreinforced case (for example circular, log spiral, bilinear wedge). Statically, the inclination and distribution of the reinforcement tensile force along the failure surface must be postulated. The capacity of reinforcement is taken as either the allowable pull-out resistance behind the potential failure surface, or as its allowable design strength, whichever is less. The target factor of safety for a reinforced slope is the same as for an unreinforced slope. However, more sophisticated methods have been developed to alleviate the deficiencies of the pseudostatic approach.

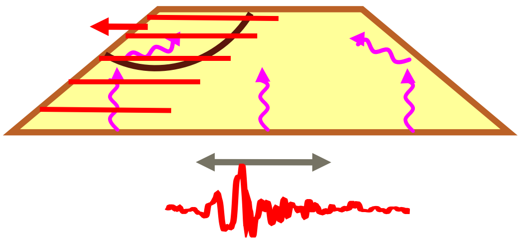


Fig -2: Schematic representation of a reinforced slope under seismic excitation

The main seismic slope stability assessment methodologies for (unreinforced and reinforced earth structures) that will be discussed and compared herein are the following:

- pseudostatic methods (based on limit equilibrium or limit analysis);
- permanent deformation methods (based on Newmark's sliding-block method);
- stress-deformation methods (numerical methods, such as the finite element method (FEM), the finite difference method (FDM), etc).

A brief overview of the aforementioned methods is as follows:

- pseudostatic analyses cannot simulate the extensive (non-discrete) failure surfaces that develop in (reinforced and unreinforced) soil slopes and cannot estimate displacements;
- a critical failure surface predicted by pseudostatic analyses approximates roughly the region of significant deformations;
- permanent deformation analyses may provide a realistic estimate of the developed displacements, but require the characteristics of the failure mass and do not provide a distribution along the height;
- numerical methods can overcome the limitations of the other two approaches, but provide satisfactory results only when proper interface simulation and advanced constitutive material modeling are used, which are not readily available.

Ongoing research is focused on further improvement of numerical simulations and analytical methods to overcome their deficiencies.

The aim of the current study is to illustrate the potentially beneficial role of the geosynthetics against the earthquake hazard that can prevent or minimimize the development of slope instability of reinforced soil slopes (Figure 2). Furthermore, this works aims to examine the available design methods, to evaluate the effect of the most important parameters involved in each method and to compare the obtained results with available experimental data from the literature. For this purpose, initially the pseudostatic method is employed and specifically the limit equilibrium method. A parametric study is presented, in which the required geosynthetic force is estimated for different slope heights, a wide range of slope inclination, various angles of internal friction and several cases of applied acceleration. In the sequence, a modified model of the well-known Newmark's sliding-block method is used to calculate permanent displacements of reinforced slopes due to a seismic excitation. Similarly, a detailed parametric study has been performed considering a wide range of the parameters involved in the calculations. Finally, the results obtained via pseudostatic and sliding-block methods are compared with finite element analysis results in order to illustrate the pros and cons of each approach. These analyses contribute not only to the examination of the dynamic response of soil reinforced structures, but also to the identification of the developed failure modes. The comparison is conducted for a reinforced embankment utilizing experimental data and results available in the literature [11]. The results of the present investigation provide a valuable insight into the seismic slope stability assessment of geosynthetically reinforced slopes.

2. PSEUDOSTATIC METHOD

2.1 Description of the methodology

The investigation of the use of geosynthetics in reinforced soil slopes of geostructures aiming to prevent the development of a potential instability is initially performed by employing the pseudostatic method. The pseudostatic method is a direct extension of the static slope stability analysis, considering also the seismic loading as horizontal and vertical inertia forces, which are equal to the seismic coefficient multiplied by the weight of the sliding mass. The basic assumptions of this methodology are the following:

- equal tensile force of each geosynthetic layer,
- length of reinforcement is sufficient to avoid pull-out,
- direction of the geosynthetic forces is tangent to the examined circle of failure.

In the current study the limit equilibrium method and more specifically the simplified Bishop method [12] was modified and applied in a fortran code developed by the authors, in order to calculate the required geosynthetic force. In the employed approach, the total required geosynthetic force for target safety factory equal to unity is calculated. More specifically, after the estimation of the unreinforced factor of safety of the slope, the total reinforcement tension per unit width of slope (T_S) required to obtain the required factor of safety (FS_R) was calculated according to the following equation [13]:

$$T_S = (FS_R - FS_U)\frac{M_D}{R} \quad (1)$$

where FS_R is the factor of safety of the reinforced slope, FS_U is the factor of safety of the unreinforced slope, M_D is the destabilizing moment and R is the radius of the circle.

The procedure described was conducted in each examined test case for several potential failure circles (Figure 3) and the maximum calculated value of the total reinforcement tension was determined. As aforementioned, according to the contemporary seismic norms [14], the required factor of safety of a soil slope is considered to be equal to unity. Therefore, the above equation, which is based on the assumption that the reinforcing moment is added on the resisting moments, yields the same results as if the reinforcing moment was subtracted from the driving moments. Additionally, the moment arm of the reinforcement forces was assumed to be equal to the radius of the circle, i.e., the forces were considered to act tangentially to the circle, as proposed for continuous extensible reinforcement, such as geotextiles or geogrids [13].

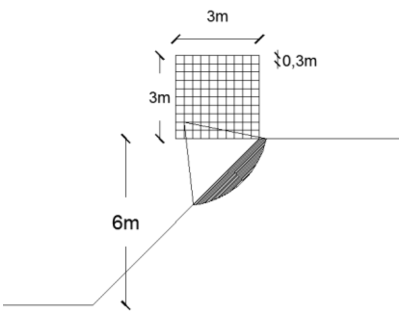


Fig -3: Determining the critical circle for a 6 m high slope having inclination of 45°

The application of the pseudostatic method was performed utilizing an in-house software developed in Fortran. Note that as described in the norms, the vertical component of the excitation is taken either as a percentage of the horizontal (i.e., $a_v = 0.5a_h$), or it is neglected (i.e., $a_v = 0$). Initially, a 6m high slope shown in Figure 3 was analysed, considering slope inclination that varied between 45° and 70°, while several values for angle of friction of the soil within the range of 30° and 45° were selected.

Details for the values of the examined parameters range that were used in the parametric study are as follows:

- height (6m – 15m),
- seismic acceleration level (0.16g, 0.24g, 0.36g),
- vertical acceleration (with: $a_v = 0.5a_h$ or without: $a_v = 0$),
- slope inclination (45° – 70°),
- shear strength of soil material (friction angle 30° – 45°).

Note that the selected values of the seismic coefficient (0.16, 0.24, 0.36) correspond to the peak ground acceleration levels (0.16g, 0.24g, 0.36g) for the standard 10% in 50 years seismic hazard level of the three seismic zones in Greece [15].

In Figure 4 the resulting total reinforcing tension is shown for two cases: (a) no applied acceleration, and (b) horizontal seismic coefficient equal to 0.16. The first case is only presented for completeness, since the static factor of safety of reinforced slopes is at least equal to 1.3 [13]. It is evident that the increase of the slope inclination, and the decrease of the angle of friction result to increase of the required reinforcement tension. In addition, the application of the seismic coefficient has increased substantially (almost 100%) the required reinforcement tension.

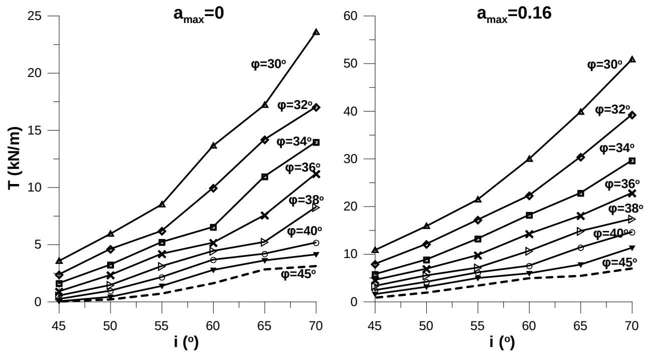


Fig -4: Variation of the total reinforcement tension for several slope inclinations and angles of internal friction. The results refer to a slope with 6m height subjected to a_{max}=0g and to a_{max}=0.16g

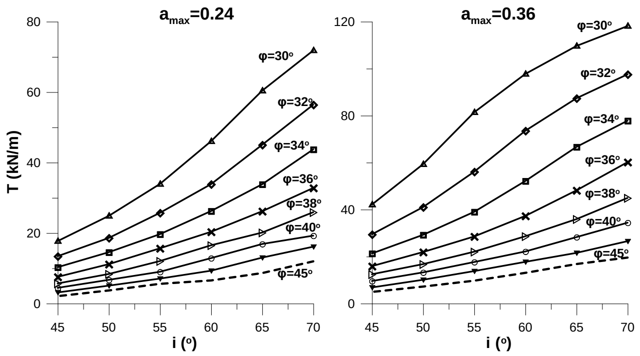


Fig -5: Variation of the total reinforcement tension for several slope inclinations and angles of internal friction. The results refer to a slope with 6m height subjected to a_{max}=0.24g and to a_{max}=0.36g

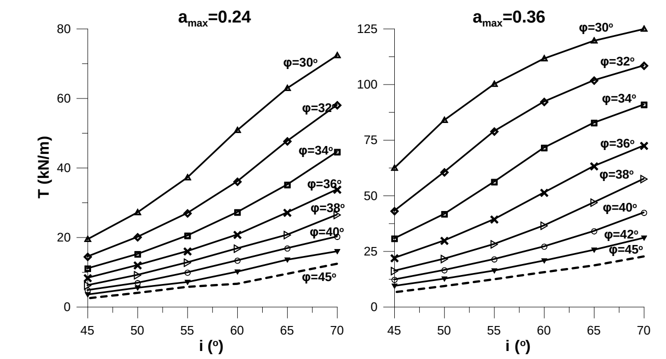


Fig -6: Variation of the total reinforcement tension for several slope inclinations and angles of internal friction. The results refer to a slope with 6m height subjected to a_{max} =0.24g and to a_{max} =0.36g and vertical acceleration equal to 50% of the a_{max}

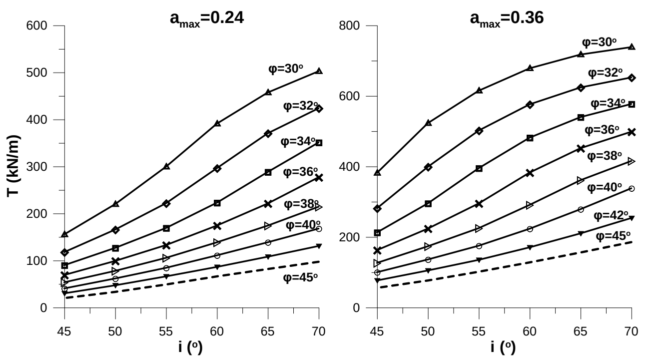


Fig -7: Variation of the total reinforcement tension for several slope inclinations and angles of internal friction. The results refer to a slope with 15m height subjected to a_{max} =0.24g and to a_{max} =0.36g and vertical acceleration equal to 50% of the a_{max}

2.2 Parametric study results

The impact of several parameters was investigated by an extended parametric study. Two values of seismic coefficient were selected to highlight the impact of the level of the applied acceleration and the results are presented in Figure 5. The maximum reinforcement tension increased by almost 50% as the seismic coefficient increased from 0.24 to 0.36.

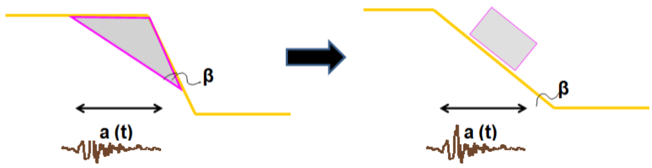
Moreover, the impact of the vertical component of the seismic coefficient was estimated. The vertical seismic coefficient was considered equal to 50% of the horizontal component as suggested by seismic norms [14]. By observing Figure 6 it is obvious that the total reinforcing tension is increased compared to the corresponding one shown in Figure 5, which does not account for the effect of the vertical seismic coefficient on slope stability. Finally, an additional parameter was examined, i.e., the height of the reinforced slope. The results shown in Figure 7 refer to a slope height equal to 15m. It can be observed that the reinforcement tension is substantially increased, by receiving values almost five times the corresponding of the slope with height equal to 6m.

In summary, the main findings of the parametric study which was performed for a geosynthetically reinforced slope utilizing pseudostatic analysis method are the following:

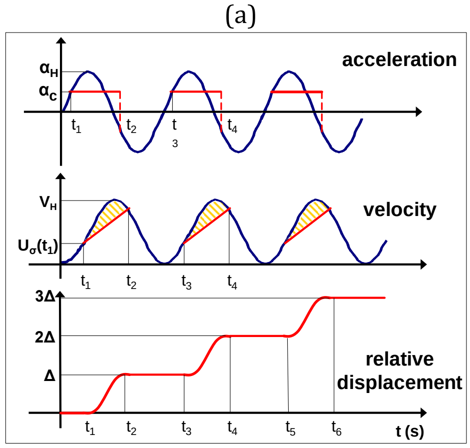
- for steeper slope the required tension increases,
- with the increase of the seismic coefficient the required tension increases,
- for higher angle of internal friction the required tension decreases,
- the height of the slope is the most crucial factor.

3. PERMANENT DEFORMATION ANALYSIS

The seismic response of engineering structures is often related to the development of permanent displacements. Permanent deformation analysis of earth structures is based on the well-known Newmark's sliding block model [16]. It was originally formulated for slope stability assessment in order to overcome the weaknesses of the pseudostatic method and it is based on a simple model of rigid block which slides on an inclined plane. As shown in Figure 8, at the basis of the model a seismic movement is horizontally imposed and permanent seismic deformation along the planar failure surface is gradually developing whenever the inertia forces of the block resting on this plane exceed the shear resistance of the interface.



(a)



(b)

Fig -8: (a) Sketch of the initial Newmark model, (b) calculation of permanent deformations

The computation of the permanent displacement is achieved by double integrating the relative acceleration, i.e., the difference between the applied and the critical acceleration. The critical acceleration is the value of the horizontal acceleration required to provide an incipient sliding state (see Figure 8b). The major assumptions associated with this method are: (a) the sliding block is infinitely rigid, (b) the stress-strain behaviour of the shear strength of the interface is rigid-plastic, (c) the uphill resistance is infinitely large, (d) the input motion is horizontal, and (e) the sliding surface is planar.

In the sequence, several researchers have investigated this simplified -yet efficient- method aiming to examine the impact of the aforementioned assumptions in the computed displacements, or to propose analytical solutions and predictive relationships, or to modify the simple model in order to represent more realistically the seismic response of other types of structures. For instance, one of the major assumptions of this approach, i.e., the contribution of the flexibility of the sliding mass, has attracted quite intense research interest. The calculation of the seismic displacements taking simultaneously into account the inertia response (coupled) of simple single-degree-of-freedom (SDOF) models has shown that the two-step procedure (decoupled) may be more conservative [17, 18].

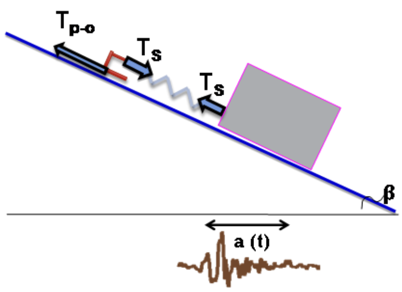


Fig -9: (a) Sketch of the modified Newmark model

Compared to the basic Newmark model the main modifications (as shown in Figure 9) of the so-called reinforced modified Newmark model (RMNM) that were implemented to take into account the presence of reinforcement are as follows [19]:

1. elastic spring, T_S, i.e., corresponding to tension of the reinforcement;
2. pull-out condition, $T_{P\text{-}O}$, corresponding to pull-out resistance of the reinforcement.

These modifications are implemented by the addition of a spring and a Coulomb-type sliding element as illustrated in Figure 9. The elasto-plastic spring simulates the nonlinear force-elongation response of the confined reinforcement. On the other hand, the Coulomb element represents the pull-out resistance of the reinforcement, which is assumed to decrease linearly with displacement to model reduction of the anchor length during permanent displacements accumulation. RMNM model assumes uniform height distribution of the seismic displacements and consequently uniform distribution of the resultant tensile force to all layers of reinforcement. Another assumption of the model is that although the spring representing the reinforcement is elastic perfectly plastic, it is modeled so that it cannot sustain plastic deformations. Therefore, when the limit value of the tensile force is exceeded, this results in reinforcement failure.

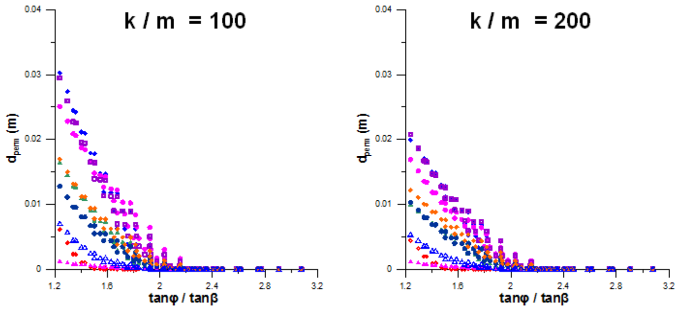


Fig -10: Impact of ratio k/m (100 and 200) for several slope inclinations and angles of internal friction for a slope with 15m height subjected to a_{max}=0.36g

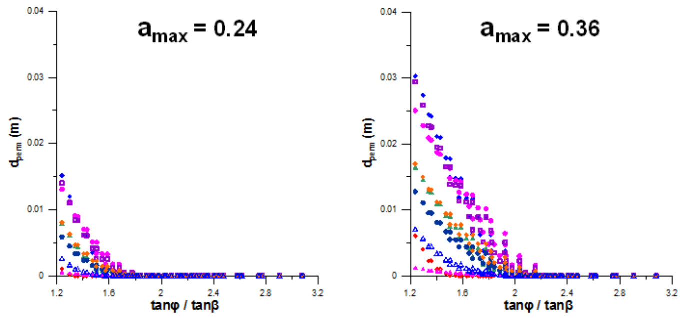


Fig -11: Impact of a_{max} (0.24g and 0.36g) for a ratio k/m = 100 for several slope inclinations and angles of internal friction for a slope with 15m height

A series of permanent deformation analyses utilizing modified Newmark model were performed to investigate the impact of the main parameters involved on the calculation of displacements by RMNM utilizing an in-house software developed in Fortran. The parametric investigation was conducted taking into account the following data:

- ratio of the stiffness of the activated reinforcement to the total mass of the sliding wedge (ratio denoted as k/m) (10, 50, 100, 150, 200),
- shear strength of soil material (i.e., soil friction angle φ = 15° – 22.5°),
- inclination of failure plane (β = 9° – 12.5°),
- seismic excitation (20 records),
- level of maximum acceleration (0.16g, 0.24g, 0.36g).

Due to space limitations only a small part of the results is included herein, as presented in Figures 10 and 11, where each symbol corresponds to the result of one run of the RMNM code for different data (φ and β values, earthquake record, etc). By observing the plots of Figure 10 it is evident that -as expected- increase of reinforcement (and ratio k/m) leads to a decrease of the resulting displacements. Similarly, the plots of Figure 11 demonstrate that as the seismic acceleration levels increase the permanent displacements also increase, while a greater scattering of the results is also noticed. Based on the parametric permanent deformation analyses, the following conclusions were drawn:

- increase of slope inclination leads to increase of displacements,
- increase of imposed acceleration level leads to increase of displacements,
- increase of reinforcement (ratio k/m) leads to a decrease of displacements,
- not so clear trends where observed regarding the impact of the height of the slope on seismic displacements,
- the characteristics of the imposed excitation affect substantially the developed permanent deformations.

4. FINITE ELEMENT ANALYSIS

Subsequently, more elaborate analyses, i.e., dynamic numerical analyses, were performed utilizing the finite element method and results were verified with experimental results. The dynamic finite element analyses of the numerical investigation were conducted utilizing ABAQUS software [20]. The models developed for the numerical investigation in the current study are based on the corresponding ones of an elaborate experimental study by Nova-Roessig and Sitar [11]. In that study a series of dynamic centrifuge tests were performed on geosynthetically-reinforced slopes (as well as vertical walls reinforced with metallic mesh), aiming to investigate the response of reinforced soil structures due to dynamic loading performed a series of centrifuge tests on reinforced slopes of 2V:1H face inclination.

Each centrifuge test included two back-to-back slopes, one reinforced with L/H =0.7 and the other with L/H =0.9 (as showm in Figure 12). The two opposing slopes, shown in Figure 12, are called as “north” (at the left side where the length of reinforcements was 90%H) and “south” (at the right side where the length of reinforcements was 70%H). The prototype model slope had been reinforced with eighteen sheets of Tru-Grid reinforcement (i.e., 18 layers of metallic grid strips) so that to maintain a static factor of safety of 1.5 when using a backfill with relative density of 75%. The results of this study indicated that lateral displacements of a reinforced soil slope increase with: a) increase of input motion amplitude and duration, b) decrease of reinforcement length and stiffness, and c) decrease of backfill density. The aforementioned findings of the experimental study were also verified via recent numerical analyses [21].

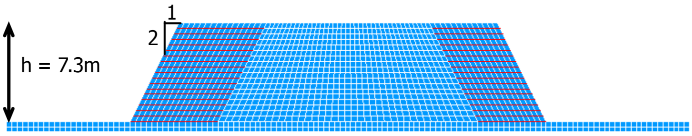


Fig -12: Finite element discretization of the examined model: south (right side with 70%H reinforcement) and north (left side with 90%H reinforcement) slopes

Figure 12 depicts the finite element mesh that was developed based on the prototype experimental configuration. The discretization of the backfill was performed using quadrilateral plane strain elements, the size (maximum length 0.5m) of which was tailored to the wavelengths of interest. The eighteen geosynthetic layers were placed as in the experimental setup. They were discretized with rod elements, since the geosynthetics are considered to attain only axial stiffness. The material properties were chosen as close as possible with those used in the experimental study. Hence, the axial stiffness of the geosynthetics was set equal to 8.3kN/m/m and the yield strength equal to 2.3kN/m^2. The elasticity modulus of the sand was set equal to 124MPa, leading thus to a shear wave velocity V_S equal to 170m/sec, while a Mohr-Coulomb failure criterion was selected to represent the yield and plastic soil behaviour with angle of friction 42.5^o and angle of dilation 2^o. In order to ensure the stability of the symmetric slopes a small cohesion intercept was also applied, equal to 5kPa.

Dynamic analyses of the model were conducted by imposing horizontal input motions at the base of the model. For this purpose, a harmonic excitation with period T equal to approximately 0.3 sec. The duration of the sinusoidal pulse had six cycles and the applied motion was scaled to two maximum acceleration levels: 0.4g and 0.8g. The results of harmonic excitations are easier to understand, provide a clearer insight into the governing mechanisms of the dynamic response of a tructure or a geostructure, and are often used in dynamic analyses, especially in analytical calculations. In addition, two real records: Gazli and Tabas, which were used in the experimental study, were also used in the numerical investigation. For more details, readers are referred to a recent study by the authors [21].

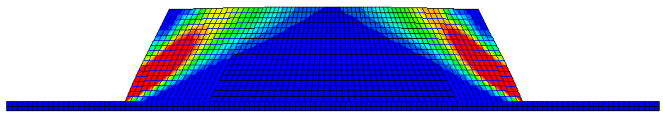
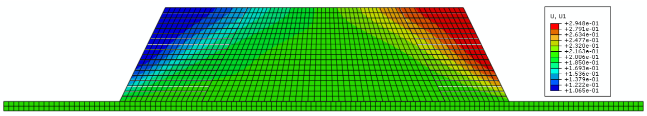


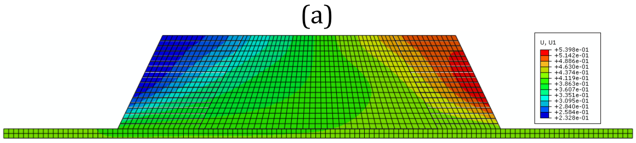
Fig -13: Typical contours of plastic strains.

Figure 13 displays typical contours of plastic strains, where quite wide zones of permanent deformations are formed. Numerical results are very similar to experimental behaviour, indicating also that a distinct failure surface is not formulated. The shape of the failure mass resembles closely a triangular wedge. The permanent horizontal displacements at both north and south slopes appear to obtain a similar pattern when observing the contour plots for the two examined acceleration levels of the harmonic excitation in Figure 14. The inclination of the failure zone does not seem to be drastically affected by the increase of the acceleration. On the other hand, the increase of the acceleration results in increased cumulative plastic deformation per each cycle of applied motion and higher permanent deformation as well [21].

In both numerical and experimental studies it was found that the lower intensity motions are related to smaller horizontal deflections and that the reinforcement layers tend to spread out deformations throughout the reinforced zone and do not allow damage localization along a discrete failure surface. Hence, the assumptions of traditional limit equilibrium-based seismic design methods are not supported by the results of both the experimental and the numerical investigations.



(a)



(b)

Fig -14: Contours of permanent horizontal displacements for maximum applied acceleration equal to: (a) 0.4g, and (b) 0.8g

Moreover, it has been found that depending on the backfill density, amplification occurs even for small to medium peak base accelerations, while de-amplification occurs at greater amplitudes [11, 21]. Detailed representation of the numerical results and a more thorough comparison with experimental results from two centrifuge studies [11, 22] can be found in a recent paper by the authors [21]. A brief overview of the obtained finite element results is as follows:

- increase of input motion amplitude affects system dynamic response in terms of displacements and accelerations;
- the pattern of the plastic zones is not affected substantially by seismic intensity level;
- permanent displacements and plastic strain regions are in agreement with those obtained from the experimental study.

5. NUMERICAL VERSUS ANALYTICAL RESULTS

Figure 15 illustrates the comparison of the numerical results performed with ABAQUS and the results obtained by pseudostatic method for various acceleration levels. As it can be observed, there are certain discrepancies between the two approaches. The results of the numerical study (that as aforementioned are supported by the experimental investigation) do not confirm the assumptions of traditional limit equilibrium-based seismic design methods. Actually, discrete failure surfaces were not formed in any of the models. More specifically, it is evident that pseudostatic failures surfaces only capture a rough approximation of real conditions, since much wider zones of increased plastic strains occur. The slopes deformed in a ductile manner under increased seismic loading, suggesting that a deformation-based seismic design method can be more realistic.

Table 1: Permanent deformations: modified Newmark model vs. experimental/numerical results.

Gazli record:		
	Newmark	experimental/numerical
L = 90%H	22.2 cm	(17.6 cm – 37.3 cm)
L = 70%H	17.0 cm	(16.5 cm – 31.0 cm)
Tabas record:		
	Newmark	experimental/numerical
L = 90%H	10.3 cm	(17.0 cm – 30.0 cm)
L = 70%H	11.1 cm	(12.5 cm – 24.0 cm)

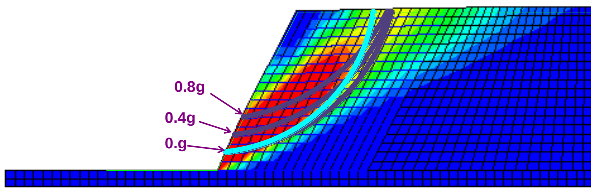


Fig -15: Failure surfaces: Numerical vs. pseudostatic method results for north slope

Since pseudostatic methods do not provide any information regarding potential slope deformations, displacement-based methods (Newmark-type, FEM, etc) are capable to better represent the damage state (functioning and serviceability) of a reinforced slope following a severe seismic event. Under this perspective, Table 1 lists the permanent displacement values fo the two real records, obtained via modified Newmark model compared to experimental/numerical results for both slopes with different geosynthetics length (L=70%H and L=90%H). Obviously, Newmark method is inferior to the finite element method, since it provides only a single displacement value per dynamic analysis and cannot estimate neither the distribution of displacements along the slope height (values in parentheses) nor the plastic deformations zones. Furthermore, the reinforced slopes did not deform rigidly in block-like, outward motions as it is assumed by Newmark approach. Hence, Newmark-type approaches can provide only an estimate of the permanent deformations of the geostructure. However, compared to the pseudostatic method they are superior since the latter cannot provide any information related to geostructure's displacements.

6. CONCLUSIONS

In the current study, the application of the geosynthetics as reinforcement to prevent the development of seismically induced slope instability was investigated with the conventional pseudostatic approach and advanced numerical modelling. Initially, the pseudostatic method was employed following also relevant design guidelines for reinforced slopes. The investigation of the most important parameters has shown that: (i) the increase of the slope inclination results to an increase of the required reinforcement tension for stability, (ii) the increase of the angle of internal friction decreases the total reinforcing tension, (iii) the increase of the seismic coefficient from 0.16 and 0.24 to 0.36 resulted to an approximately 100% and 50% increase of the required reinforcement tension, (iv) the vertical acceleration does not affect considerably the magnitude of the required reinforcement, and (v) the increase of the height of the slope is the most crucial factor, since it is related to the greatest possible efficiency of the application of the reinforcement.

Another series of permanent deformation analyses was also performed in order to investigate the influence of the main parameters involved in the calculation of the seismic displacements of the reinforced earth structures based on the well-known Newmark's sliding block. The findings of this parametric investigation verified that increase of slope inclination and acceleration levels leads to higher displacements, while increase of reinforcement leads to lower displacements. On the other hand, the impact of the increase of slope height isn't so clear, as permanent deformations were found not to be influenced in a straightforward manner. Finally, the frequency content of the imposed acceleration time-histories strongly affects the dynamic response of the model.

Subsequently, dynamic analyses were performed utilizing the finite element method and results were verified with experimental results. In general, conventional pseudostatic design methods use simplifications and are not able to capture the deformation patterns of the problem at hand. Thus, numerical analyses contribute not only to the more accurate evaluation of the dynamic response of reinforced geostructures, but also to the identification of the developed failure modes. Therefore, a dispacement-based approach, in the viewpoint of contemporary performance-based earthquake design, is more reliable and realistic than conventional approaches for the evaluation of seismically-induced deformations of reinforced soil slopes and walls. Nevertheless, regardless the analysis method, the current investigation has shown that the geosynthetics can be efficiently applied as mitigation measures to efficiently prevent the development of seismic slope instability of soil slopes.

Conclusively, the current investigation has shown that the geosynthetics may be efficiently applied as mitigation measures to reduce the anticipated permanent deformation arising from seismic wave propagation and to prevent the development of seismic slope instability in large-scale embankments and slopes, irrespectively of the adopted seismic design method. Each design approach has its advantages and disadvantages, while the general conclusions that can be derived from the presented comparison of various methodologies are as follows:

- pseudostatic analyses are incapable of simulating the extensive (non – discrete) failure surfaces that develop in reinforced soil slopes;
- critical failure surface (predicted by pseudostatic analyses) approximates the region of significant displacements but not the inclination of failure mass;
- permanent deformation analyses may provide a realistic estimate of the developed displacements, but require the characteristics of the failure mass and do not provide a distribution along the height;
- stress deformation approaches, i.e., numerical simulations, provide results within the range of the experiments, however, there are difficulties to fully capture all experimental details;
- finite element analysis has a higher sophistication level, hence it is most frequently used for research purposes; nonetheless, it needs further elaboration on dynamic non-linear material constitutive and interface models.

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