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SEISMIC PERFORMANCE OF SLOPES IN PSEUDO-STATIC DESIGNS WITH DIFFERENT SAFETY FACTORS*

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Abstract– Seismic coefficient values coupled with minimum pseudo-static safety factors are still used for analysis where selection of seismic coefficients relies on expertise and judgment. However, safety factor approach does not give any idea about the deformations and displacements that are expected to occur during earthquake loading. Displacements are mostly evaluated by equations based on yield acceleration of the slope and maximum acceleration of sliding mass. The method based on rigid block gives co-seismic permanent slope deformation when its factor of safety equals 1.0, hence, there is a need to link slope displacements, seismic coefficients and pseudo-static safety factors. This will enable the designers to predict slope displacements based on selected seismic coefficients. In the present paper, slope displacements obtained for different peak ground accelerations and safety factors are used to propose charts linking co-seismic slope displacements (D), seismic coefficients (k_h) and pseudo-static safety factors (FS), which are important parameters in pseudo-static approach. This enables the k_h values to be chosen based on allowable displacements instead of using judgment and expertise. Results show that k_h values for any allowable displacement should be based on anticipated PGA and FS values. Subsequently, slope displacements are utilized in developing a novel displacement-based methodology to select the seismic coefficient which will be used to calculate the pseudo-static safety factor.

Keywords– Slope stability, slope, displacement, performance, seismic coefficients

1. INTRODUCTION

Slope failures are often observed following large earthquakes. Because of their effects on infrastructure facilities such as buildings, bridges, roads and lifelines, they have a significant impact on casualties and economic losses. As a result, evaluation of the stability of slopes has become an important part of geotechnical earthquake engineering. Several approaches for evaluation of seismic slope stability, ranging from simple to complex, are available and can be divided into: 1) pseudo-static methods, 2) sliding block methods, and 3) stress-deformation methods [1].

The performance of earth structures subjected to seismic action can be evaluated through force-based pseudo-static methods, displacement-based sliding block methods, non-linear soil behavior and fully coupled effective stress numerical analyses. In principle, numerical methods allow the most comprehensive analyses of the response of earth structures to seismic loading. However, reliable numerical analyses require accurate evaluation of soil profile, initial stress state, stress history, pore water pressure conditions, strength and deformation characteristics of the selected soil layers. Moreover, cyclic soil behavior can be properly described using advanced constitutive models developed within the

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framework of bounding surface plasticity or kinematic hardening plasticity, which requires input parameters that are not usually measured in field or laboratory testing.

For slope stability analysis, limit equilibrium method (LEM) is widely used by engineers and researchers which is a traditional and well-established method. Although the LEM does not consider the stress–strain relations of the soil, it can provide an estimate of the safety factor without requiring the initial conditions. For this reason, the method is favored by engineers. LEM is known to be a statically indeterminate method which requires assumptions on the distributions of internal forces for evaluation of the safety factor.

The displacement-based approach provides a compromise between the rather inadequate pseudo-static approach and the more refined numerical analyses; indeed, it has the advantage of giving a quantitative assessment of the earthquake-induced displacements using a rather simple analytical procedure. Prediction of earthquake-triggered landslide displacements is important for the design of engineered slopes seismic hazard analysis as well as for co-seismic landslide analysis. The earthquake acceleration needed to reduce the factor of safety to 1.0 is called the yield or critical acceleration. This procedure is simple and requires no more information than what is needed for a static factor-of-safety analysis.

A common approach to using pseudo static analysis is to iteratively conduct a limit-equilibrium analysis using different values of k until FS=1. The resulting pseudo-static coefficient is called the yield coefficient, k_y. As mentioned above, the conventional methods used for evaluating the performance of slopes under seismic loading includes application of a seismic coefficient to calculate the pseudo-static safety factor and calculation of permanent displacements. These approaches employ pseudo-static limit equilibrium analysis. Until the 1960s, engineers employed a seismic coefficient to assess the safety factor of slopes and embankments. In the current state of the art, seismic coefficient values coupled with minimum pseudo-static safety factors are used in the analyses, where the selected seismic coefficients rely on expertise and judgment. However, safety factor approach does not provide any information about deformations that are expected to occur during earthquake loading. Deformation is a better indicator of slope performance and therefore, seismic slope stability is evaluated more and more frequently based on the permanent deformations rather than the safety factor criterion. In this context, Newmark's [2] sliding block model is a widely used tool for calculating permanent slope displacements.

Displacements are mostly calculated by equations based on yield (k_y) for rigid slopes. The materials that comprise slopes are compliant and respond dynamically as deformable bodies to ground motions. As the motion propagates, different parts of the slope move by different amounts and different phases, thereby creating a distribution of accelerations throughout the slope that vary in space and time. These accelerations induce inertial forces, that when superimposed on the self-weight of the soil mass, destabilize the slope. Therefore, slope displacements are calculated by equations based on maximum equivalent accelerations (k_{max}). Where, k_{max} represents the peak value of the *HEA* time history and represents a spatial average of the accelerations acting on the slide mass. The dynamic response analysis is then performed to quantify the accelerations experienced by the slide mass and is expressed as Horizontal Equivalent Acceleration or *HEA* time history.

Based on the above arguments, to predict slope displacements based on selected seismic coefficients, there is a need to link slope displacements, seismic coefficients and pseudo static safety factors. This will enable the designers to predict slope displacements based on selected seismic coefficients. Following a review of the literature on different methods used in seismic slope stability problems, a methodology is proposed to link the permanent slope displacements, seismic coefficients, and pseudo-static safety factors.

In order to facilitate more relations between LEM and sliding block methods, the following activities were conducted in the present paper:

1- Determination of horizontal acceleration effects on slope (based on various limit equilibrium procedures including Bishop's method).
2- Determination of earth slope's factor of safety under effects of imposed seismic horizontal acceleration.
3- Determination of slope yield acceleration coefficient (variation in slope yield acceleration coefficient, where factor of safety is required to become equal 1).
4- Determination of co-seismic permanent slope deformation (based on various rigid block methods).

2. BACKGROUND

The earliest methods of seismic slope stability analyses used a limit-equilibrium pseudo-static approach and considered stability in terms of a simple factor-of-safety. These very basic procedures evolved into more sophisticated deformation-based approaches during the 1960s and 1980s. Today it is common practice in geotechnical earthquake engineering to estimate seismically-induced displacements in slopes and earth structures, using one of the available deformation-based analysis procedures. Such an approach is appropriate as displacements ultimately govern the serviceability of a slope after an earthquake.

Over the past 50 years, roughly 30 different deformation-based methods have been developed to compute seismic slope displacements. These procedures generally fall into one of three categories: (1) *rigid block-type procedures*, which ignore the dynamic response of slopes [2-4],(2) *decoupled procedures*, which account for dynamic response, but “decouple” this response from the sliding response of slopes[5-6], and (3) *coupled procedures*, which “couple” the dynamic and sliding response of slopes[7].

In design applications, these methods are used in a predictive capacity to estimate the amount of earthquake-induced displacement for a design earthquake event. More detailed information is presented in individual sections devoted to each method.

a) *Pseudo-static coefficient*

Landslides account for a significant portion of total earthquake damage; therefore, seismic stability of slopes is of primary concern. As emphasized by Kramer [1], analysis of seismic stability of slopes is complicated by the need to consider the effects of seismic stresses, their effects on strength and stress-strain properties of the slope materials.

Stability analyses of earth slopes during earthquake were initiated in the early 20thcentury using what has come to be known as the pseudo-static method. The first known documentation of this method in the technical literature was proposed by Terzhagi [8]. Pseudo-static analysis, models the seismic shaking as a permanent body force that is added to the force-body diagram of a conventional static limit-equilibrium analysis. Normally, only the horizontal component of earthquake shaking is modeled because the effects of vertical forces tend to average out to near zero.

Figure 1 shows the forces acting on a sliding mass of the soil above a failure surface in a pseudo static analysis .The magnitude of the inertial forces acting on the sliding mass, $F_h$andF_v, are calculated as

$$k_{\mathrm{h}}={}^{a_h}/_{g} \tag{1}$$

$$k_{\mathrm{v}}={}^{a_v}/_{g} \tag{2}$$

Where a_h and a_v are horizontal and vertical pseudo-static accelerations; k_h and k_v are dimensionless horizontal and vertical pseudo-static coefficients (seismic coefficients) and W is the weight per unit length of slope, β is the slope angle and g is the acceleration of gravity. The horizontal pseudo-static force affects the pseudo-static safety factor considerably, whereas the vertical pseudo-static force has been shown to be completely insignificant and therefore it can be neglected. In this context, the pseudo-static factor of safety (FS) is calculated as:

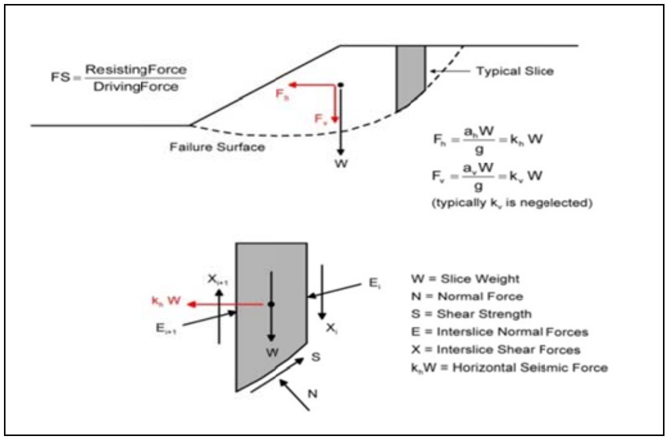

Fig. 1. Force diagram of a landslide in dry, cohesive and frictional soil. W is the weight per unit length of the landslide, k is the pseudo-static coefficient, s is the shear resistance along the slip surface, and β is the angle of inclination of the slope surface

$$FS = \frac{\text{Resisting forces}}{\text{Driving forces}} \tag{3}$$

The choice of the seismic coefficient value gives rise to some uncertainties and further uncertainties are related to the values of the safety factor [9-13]. Even though the peak acceleration causes the safety factor value to drop below 1.0, this may be for a very short duration and the resulting displacement may be negligible. Therefore, use of peak ground acceleration as the seismic coefficient in conjunction with a safety factor of 1.0 has been shown to give excessively conservative assessments of slope performance during earthquakes [14]. In many building codes, empirical values based on judgment are used. Simonelli [13] presented charts to find out whether the seismic coefficients used in Italy were appropriate. Kramer [10] stated that recommendations given would be appropriate for most slopes, indicating that it should be based on the actual anticipated level of acceleration in the failure mass. Gazetas showed that the value of k_h for earth dams depended on the size of the failure mass [15].

Selection of the pseudo-static coefficient is thus the most important aspect of pseudo static analysis; however, it is most difficult to calculate. Table 1 lists several recommendations for selecting a pseudo static coefficient. Significant differences in approaches and resulting values clearly exist among the studies cited. One key issue is calibration. Some of these studies have been calibrated for earth-dam design, in which as much as 1m of displacement is acceptable. These same values are commonly used in the stability assessment of natural slopes, in which the acceptable displacement might be as little as 5–30cm. The most commonly used values in California are k=0.15 and FS>1.1. But then again, these criteria were formulated for earth dams that could accommodate about 1m of displacement.

After summarizing a number of published approaches for determining an appropriate seismic coefficient, Kramer concluded that “there are no hard and fast rules for selection of a pseudo static coefficient for design.”

Stewart et al. [19] developed a site screening procedure based on the statistical relationship of Bray and Rathje [6], wherein a pseudo-static coefficient is calculated as a function of maximum horizontal ground acceleration, earthquake magnitude, source distance, and two possible levels of allowable displacement (5 and 15 cm) [19]. Bray and Travasarou [7] presented a straight forward approach that calculates the pseudo static coefficient as a function of allowable displacement, earthquake magnitude, and spectral acceleration. The common basis of these rationalized approaches is calibration based on allowable displacement [11].

Table 1. Pseudo-static coefficients from various studies

Investigator	Recommended Pseudo static Coefficient (k)	Recommended factor of safety (FS)	Calibration conditions
Terzhagi [8]	0.1(R-F=IX)	>1.0	Unspecified
	0.2(R-F=X)		
	0.5(R-F>X)		
Seed (1979) [16]	0.10(M=6.50)	> 1.15	<1 m displacement in earth dams
	0.15 (M=8.25)		
Marcuson (1981) [17]	0.33-0.50 PGA/g	>1.0	Unspecified
Hynes- Griffin and Franklin (1984) [18]	0.5 PGA/g	>1.0	<1 m displacement in earth dams
California Division Of Mines and Geology (1997)	0.15	>1.1	Unspecified: probably based on <1 m displacement in dams
JCOLD Japan	0.12-0.25		
Corps of Engineering	0.1 (Major Earthquake)	>1	Unspecified
	0.15 (Great Earthquake)	>1	
IRI Road and Railway Bridges Seismic Resistant Design Code NO: 463	0.5 A	>1	Unspecified
Indian standard for Seismic design of earth	0.33 Z I S	>1	Unspecified

R-F is Rossi –Forel earthquake intensity scale.
M is earthquake magnitude.
PGA is Peak Ground Acceleration.
G is acceleration of gravity.
A is ratio of design acceleration to acceleration of gravity (0.2 to 0.35)
Z is zone factor (0.1 to 0.36)
I importance factor (1.0 to 2.0)
S site amplification factor (1.0 to 2.0)

b) Deformation analysis

The following section includes a discussion of appropriate applications of these types of analysis.

1. Stress-strain analysis: The advantage of stress-strain modeling such as Finite Element Analysis is that it gives the most accurate picture of what actually happens in the slope during an earthquake. Clearly, models that account for the complexity of spatial variability of properties and the stress-strain behavior of slope materials yield more reliable results. But stress-deformation modeling has also its drawbacks. The complex modeling is warranted only if the quantity and quality of the data merit it.

2. Permanent deformation analysis: Newmark [2] introduced a method to assess the performance of slopes during earthquakes that bridges the gap between overly simplistic pseudo static analysis and overly complex stress-deformation analysis. Newmark's method models a landslide as a rigid block that slides on an inclined plane. The block has a known yield or critical acceleration, the acceleration being required to overcome basal resistance and initiating sliding (Fig. 2).

On conceptual level, all deformation-based methods are *models* which are simplified approximations of the real physical mechanism of seismic-induced deformation in slopes. As mentioned above there are three fundamental models, all of them relied on deformation-based methods. These model categories range from simple to complex and differ with respect to the assumptions and idealizations used to represent the mechanism of earthquake-induced displacement.

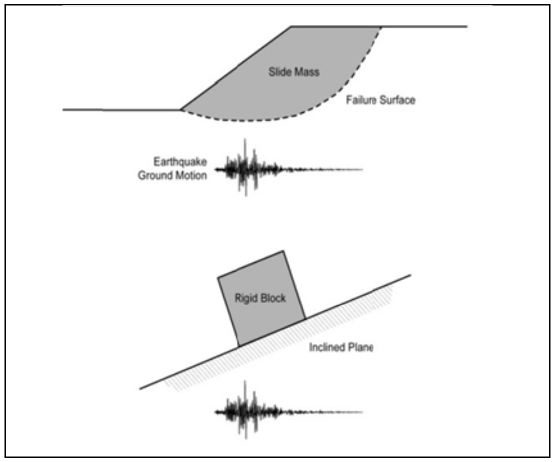

Fig. 2. Newmark's sliding-block analogy

1. Rigid block

The rigid-block model was originally proposed by Newmark [2] and is based on the sliding-block analogy illustrated in Fig. 2. It considers an acceleration time history represented by a simple sine motion as shown in Fig. 3 and assumes that permanent deformation initiates when the earthquake-induced accelerations acting on a slide mass exceed the yield resistance on the slip surface. This resistance is quantified by the seismic yield coefficient (k_y) [refer to (1) in Fig. 3a]. At this point the slide mass breaks away from the rest of the underlying slope and sliding occurs at a constant rate of acceleration equal to k_y[refer to (2) in Fig 3a]. During this time, the velocity of the ground is greater than the velocity of the slide mass (Fig. 3b). Sliding continues until the following conditions are met: (1) accelerations fall below $k_y$and (2) velocity of the slide mass and the underlying ground coincide [refer to (3) in Fig. 3b]. The permanent displacement that occurred during each interval of slip, is determined by *double-integrating* the shaded regions of the acceleration time history. The total permanent deformation accumulated during each interval of slip is indicated by the displacement plot shown in Fig. 3c.

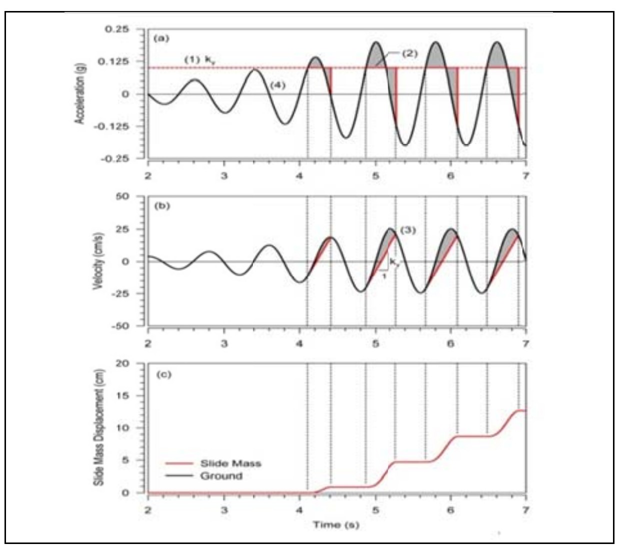

Fig. 3. Newmark's rigid -block procedure for calculating Earthquake-induced permanent deformation (simple sine motion)

Newmark [2], Ambraseys and Srbulov [20], Jibson [9], Saygili and Rathje [4] used this method with various assumptions for calculation or co-seismic slope permanent deformation. The recommended single (scalar) ground motion parameter model is the "peak ground acceleration PGA, earthquake magnitude M" model from Saygili and Rathje [21], and the recommended two vector ground motion parameter model is the PGA and PGV(peak ground velocity), models from Saygili and Rathje [4]. For simplicity, these models will be called the SR08/RS09 models.

2. Decoupled analysis

Soon after Newmark [2] published his rigid-block method, more sophisticated analyses were developed to account for the fact that landslide masses are not rigid bodies, but deform internally when subjected to seismic shaking [22-23]. The most commonly used of such analyses was developed by Makdisi and Seed [5]. They produced design charts for estimating co-seismic displacements as a function of slope geometry, earthquake magnitude, and the ratio of yield acceleration to peak acceleration.

In decoupled analysis, the first step is dynamic-response analysis of the slope with 1D programs such as EERA and SHAKE, or with 2D programs such as PLAXIS and FLAC; through estimating average acceleration-time history for the several points within the slope mass above the potential failure surface. The average acceleration has been referred to as k or HEA, the horizontal equivalent acceleration [6]. Peak values are generally referred to as k_{max} or MHEA, the maximum horizontal equivalent acceleration. In the second step, the permanent earthquake-induced deformation is calculated through double-integration of the HEA time history. This approach is referred to as a decoupled analysis. Computation of dynamic response and the plastic displacement are performed independently.

3. Coupled analysis

In a fully coupled analysis, the dynamic response of the sliding mass and the permanent displacement are modeled together so that the effect of plastic sliding displacement on the ground motions is taken into account. Wartman et al. [24] compared the sliding response of deformable clay masses and a rigid block on an included plane subjected to cyclic motion. They found that the Newmark-type rigid block analysis was overly conservative for cases where the tuning ratio (T_{ratio}), the ratio of the predominant frequency of the input motion (T_p) to the predominant natural frequency of the slope (T_s) was greater than about 1.3 and un-conservative when T_{ratio} was in the range of 0.2–1.3 [24].

More recently, Rathje and Bray (1999, 2000) compared results from rigid block analysis with linear and non-linear coupled and decoupled analyses. Many empirical models predict permanent deformation "D" as a function of k_y and one or more ground motion parameters (e.g., PGA, Arias Intensity I_a, PGV, and mean period, T_m). There are many available predictive models, but the most recently developed models are those proposed by Rathje and Antonakos [22] and Bray and Travasarou [7].

3. METHODOLOGY

In methods based on the permanent deformation method, safety factor is assumed to be equal to 1. In pseudo-static methods based on limit states, safety factor is usually considered to be greater than 1 under specified pseudo-static acceleration. Developing a method to calculate the seismic deformation of an earth slope with a specified safety factor, would be a bridge between these two methods of analysis. In other words, if an earth slope is designed based on a specified horizontal pseudo static accelerations and safety factor, how much would the estimated earth slope deformations be?

In the current state of the art, although many engineers still employ seismic coefficient values coupled with minimum, seismic performance of slopes based on the permanent deformation may be evaluated much better than that of the safety factor criterion. Regarding the above arguments, the link among slope displacements, seismic coefficients and pseudo-static safety factors may be very useful in engineering practice. The method that is proposed in this paper discusses this link. Characterizing the relationships between the safety factor and the displacements of an earth slope, is the aim of this study, which consists of three stages as follows:

1. Investigation of the relationships between $\boldsymbol{k_h}$, FS and $\boldsymbol{k_y}$ in a slope.
2. Evaluation of the relationships between $k_y$and D.
3. Determination of relationships between k_y, FS, and D.

a) Relationships BETWEEN k_h, FS and k_y in a slope

Figure 4 shows the schematic slope considered in this study. The analyses were carried out for a range of slope geometries and shear strength parameters as follows:

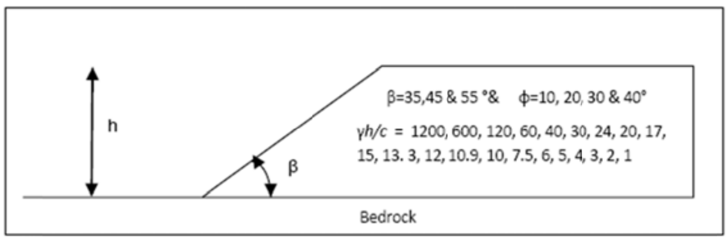

Fig. 4. Schematic natural slope considered in this study

In order to determine the relationships between k_h, FS and k_y first a range of mechanical and geometric characteristics were considered and variation of FS versus k_h were calculated by Bishop Method.

Subsequently, the slopes under the effect of different pseudo-static acceleration coefficients of 0.05, 0.1, 0.15, 0.2, 0.25, 0.3, 0.35, 0.4, 0.5, 0.6, 0.7, 1.0, and 1.2 were analyzed and FS was determined.

The relationship between $k_h$and γh/c (γ, Unit Weight, h, height of slope and c, cohesion of materials that comprise the slope), for constant values of φ (internal friction angle of soil), β (slope angle) and values of FS equal to 0.9, 1.0, 1.1, 1.2 were calculated.This relationship is calculated and depicted for a sample slope with an adopted angle of 35 degrees with internal friction angle of 10 degrees for various safety factors. Drawing the diagram assuming a safety factor of 1, the relationship between k_y and γh/c can be calculated (Fig. 5).

Using this chart, by calculation of the safety factor of a slope with given values of γh/c and k_h, the value of $k_y$can be determined. For example, according to Fig. 5, for a slope angle (β) of 35°, internal friction angle (φ) of 10°; and $\gamma h/c$ of 7.5 under the effect of pseudo-static acceleration 0.16 , FS is determined as 1.2 ,and increasing $k_h$to 0.29 causes the safety factor, FS, to decrease from 1.2 to 1.0.

In Fig. 6, the relation between k_y, $k_h$and FS is shown in a different form. Usually, k_y is used to determine the seismic permanent displacements of slopes. Therefore, it may be possible to express the k_y in relationships of permanent deformation of earth slopes as a function of k_h and FS. In this case, the displacement would be expressed as a function of k_h, FS, ground motion parameters and earth slope characteristics.

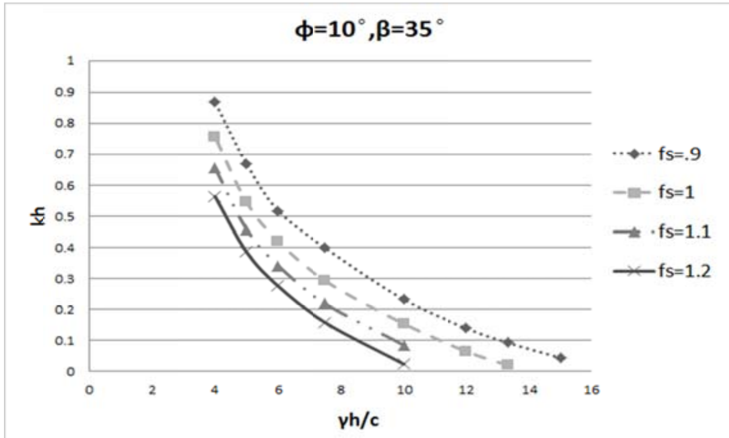

Fig. 5. Relationship between γ*h/c* and k_h for φ=10°, β=35° (Bishop Method)

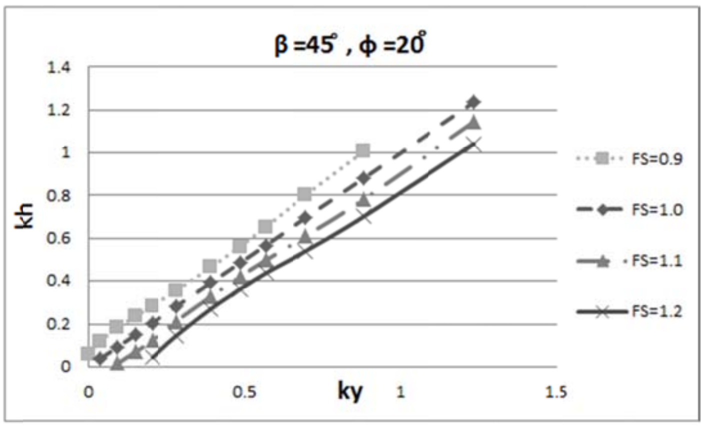

Fig. 6. Relationship between k_h, k_y for various values of β, φ and FS

b) Evaluation of the relationship between k_y and D

As mentioned in *background* section, several relationships have been proposed between k_y and the permanent deformation of a slope, D, based on Newmark Method.

Miraboutalebi et al [25] used records from seven earthquakes recorded in Iran with magnitudes greater than 7.5 in order to calculate the seismic deformation of earth slopes with PLAXIS software (decoupled analysis). Table 2 shows the details of these earthquakes. The earth slopes with 35 and 45 degree angles on horizontal bedrock with $\gamma h/c$ ratios of 30 and 45 and h/v_s ratios of 0.04, 0.1 and 0.3 seconds were analyzed [25].

Where, v_s is the shear wave velocity of soil.

Table 2. Details of earthquake records of different locations used in this study

Station	Long. Tm	Lat.	Date D/M/Y	P.G.A Cm/s/s	Predominant Frequency(Hz) Longitudinal	Transversal	Magnitude Mw	Ms	Mb
Deyhook	57.5	33.29	16/09/1978	410	5	2.5	-	7.4	6.4
Tabas	56.92	33.58	16/08/1978	897	5	4.2	-	7.4	6.4
Ab bar	48.97	36.92	20/06/1990	635	6.3	8.3	-	7.7	6.4
Meymand	52.75	28.87	20/06/1994	503	5.6	4.5	-	5.7	5.9
Zanjiran	52.62	29.07	20/06/1994	1006	10	10	-	6.4	6.2
Avaj	49.22	35.58	22/06/2002	498	5	4.2	6.5	6.4	6.2
Bam	58.33	29	26/12/2003	989	5	4.5	-	6.7	-

Miraboutalebi et al‘s [25] analyses were decoupled. First, the average response acceleration time history, based on the records which are presented in Table 2, was evaluated in the sliding mass using the results of dynamic analysis. Finally, the slope's permanent displacement was calculated based on Newmark's method.

In order to select an appropriate seismic earth slope deformation predictive model based on Iran's earthquake records the results of the analyses of PLAXIS software (decoupled analysis) were compared with the results of scalar and vector predictive deformation models of Rathje et al. [22], Jibson [9], Bray and Travasarou [7] and Miraboutalebi et al [25]. The results are shown in Fig. 7.

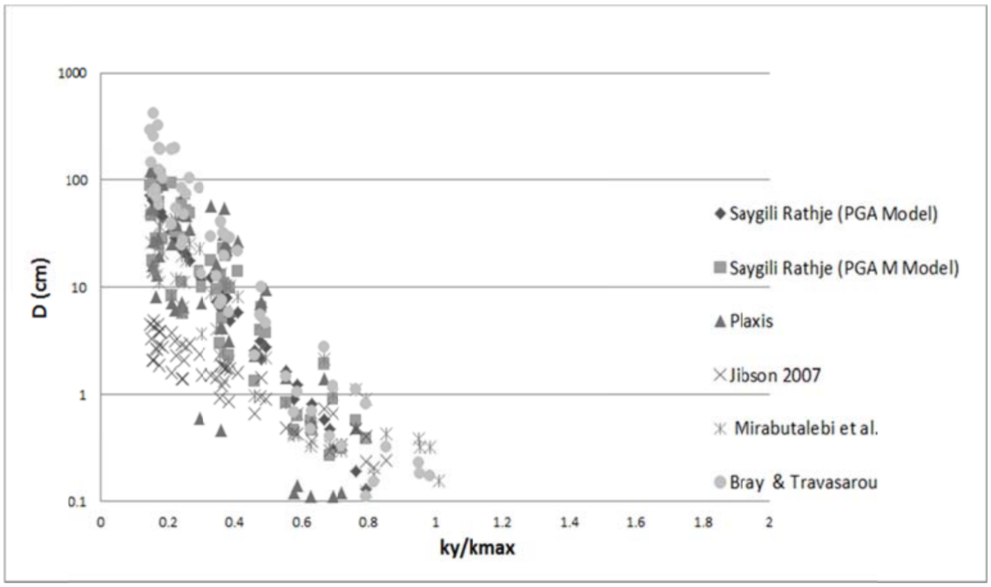

Fig. 7. Comparison of slope deformation predictive models

To compare the results, the mean residual (the difference of natural log of the calculated slope displacements from PLAXIS and natural log of the predicted displacements obtained from other methods) and the standard deviation of the mean residuals was calculated. The obtained standard deviations for RS09 (PGA, M model), SR08 (PGA Model) of Rathje et al., Jibson [9], Miraboutalebi et al. [25] and Bray & Travasarou models were 1.004, 1.197, 1.871, 0.836 and 1.186, respectively. It can be concluded that results of PGA&M unified model based on RS09 is more consistent with the results of Miraboutalebi et al. (2011), which is based on earthquake records from Iran.

The RS09 (PGA &M) modified model [21-22] is as follows:

$$\mathrm{Ln}\,(D) = a_1 + a_2\left(\frac{k_y}{PGA}\right) + a_3\left(\frac{k_y}{PGA}\right)^2 + a_4\left(\frac{k_y}{PGA}\right)^3 + a_5\left(\frac{k_y}{PGA}\right)^4 + a_6 \ln(PGA) + a_7(\mathrm{M}-6) + \varepsilon\, \sigma_{LnD} + 3.69T_s - 1.22{T_s}^2 \quad (4)$$

Where: $\sigma_{LnD} = 0.694 + 0.322\ k_y / k_{max}$

The coefficients of the model are given below:

a_1=4.89, a_2=−4.85, a_3=−19.64, a_4=42.49, a_5=−29.06, a_6=0.72 and a_7=0.89

The most important feature of the decoupled model is the fact that the slide mass is modeled as a compliant, non-rigid block. This means that the slide mass is a deformable body that can respond dynamically to earthquake shaking. The main implication of this expression is that accelerations within the slide mass and slope will be different than those in the foundation material below the slope. This is due to accelerations originating from the foundation level and propagating up (possibly amplifying or de-amplifying along the way); causing the slope to move by different amounts and in different phases thereby creating a spatial distribution of accelerations. The dynamic response analysis models calculate this behavior and its effect on the slide mass which is quantified through the k_{max}. In comparison the rock outcrop motion used with the rigid-block model, k_{max} is a much more realistic representation of the seismic loading experienced by the slide mass. In this way, the decoupled model marks a conceptual improvement over the rigid-block model for modeling the mechanism of earthquake-induced deformation.

For a dynamic response of compliant, non-rigid slope to earthquake shaking, $k_{max}$is used instead of PGA, which can be obtained from Fig. 8 as a function of T_s/T_m and PGA.

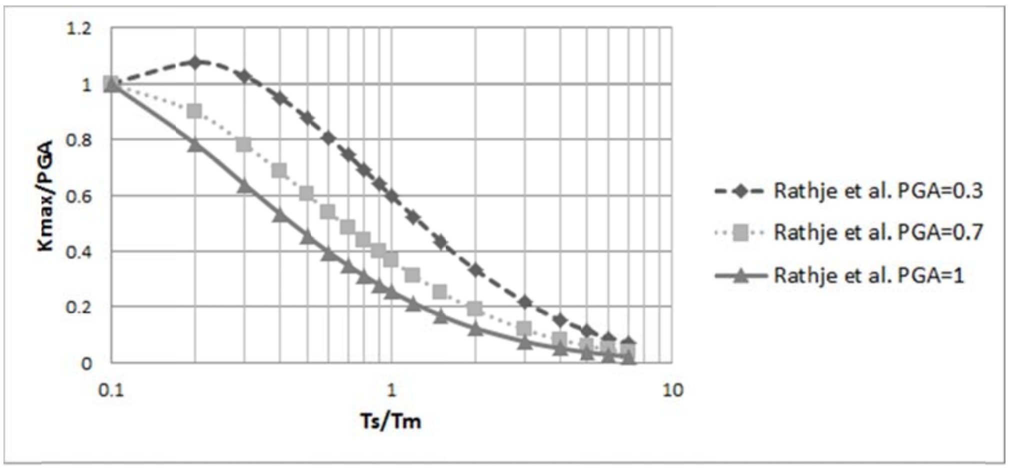

Fig. 8. Computed k_{max} values plotted versus

As noted, topography can locally modify earthquake ground shaking in slopes. Additionally, site effects and the one dimensional amplification of earthquake ground motion, can also affect the dynamic response of soil and soft rock slopes during earthquakes. Topographic effects have shown a dramatic increase in the amplitude of ground shaking near the crests of steep slopes [26].The intensity of base shaking is a function of the defined seismic hazard level (e.g. return period) and may be estimated based on a site-specific seismic hazard analysis, or from local seismic codes. The effects of topography are accounted based on a simplified procedure developed by Ashford et al. [27]. The procedure suggests that the ground motion of the free field behind the crest PGA_{ff} is increased by 50%. The PGA on the slope face at the location of the cells $PGA_{sf}$is obtained by linear interpolation between the base and the crest as prescribed by Euro Code 8.

To estimate the seismic deformation for a given slope with the apparent safety factor, calculated by a given pseudo- static coefficient, the steps are as follow:

1. According to Fig. 6, based on geometric characteristics of the slope and physical and mechanical properties of the soil, γh/c is determined for the defined k_h and calculated FS.
2. k_y is determined for the obtained γh/c in Fig. 6 (obtaining k_h with obtained γh/c on curve FS=1).
3. According to Fig. 8 for compliant, non-rigid slope k_{max} could be predicted directly from T_s/T_m and PGA.
4. Based on k_y and other characteristics of the predictive model which is used, permanent slope deformation can be determined which is also the seismic permanent slope deformation for the given k_h and FS.

Continuing such computations, seismic deformation of the given earth slope with defined FS of 0.9, 1.0, 1.1 and 1.2 can be obtained for different values of k_h as presented in Fig. 9. This figure can be described as a bridge from limit equilibrium results to Newmark's based methods of estimating the seismic deformation of the slopes.

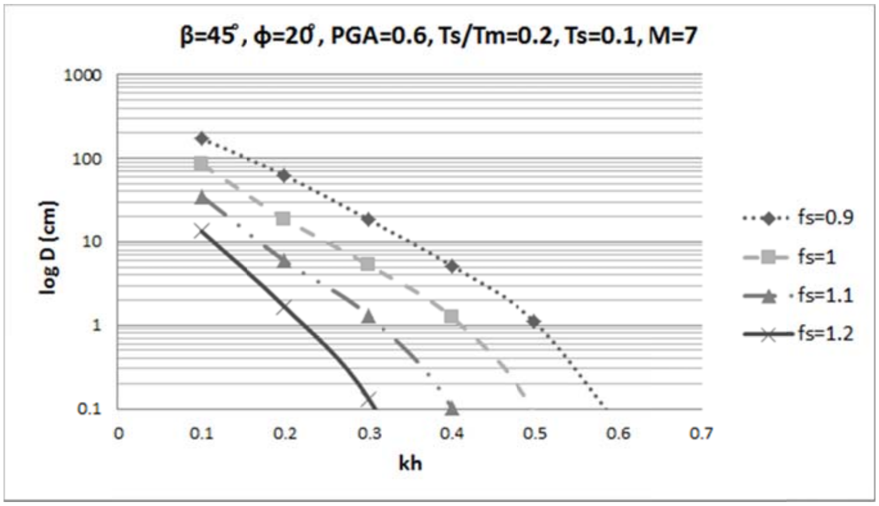

Fig. 9. Variation of seismic earth slope permanent deformation with different values of k_h & FS

4. RESULTS AND EVALUATION

For different β, φ, $\gamma h/c$, $k_h$and various methods used for calculating FS from limit equilibrium method, pseudo-static analysis was performed and the relationship between k_h,k_y, $\gamma h/c$,FS for different values of β and φ was determined. The slope angle (β) for 55°, 45°,and35°; internal friction angle (φ) for 10°, 20°, 30°, and40°; and $\gamma h/c$ for 1200, 600, 120, 60, 40, 30, 24, 20, 17, 15, 13. 3, 12, 10.9, 10, 7.5, 6, 5, 4, 3, 2, 1 were considered. Next, the slopes were analyzed under the effect of different pseudo-static accelerations as 0.05, 0.1, 0.15, 0.2, 0.25, 0.3, 0.35, 0.4, 0.5, 0.6, 0.7, 1.0, 1.2and safety factors were determined.

According to Fig. 9, it is can be seen that in addition to FS andk_h, seismic displacement of slope is a function of φ, β, PGA, T_s/T_m,T_s and M. The following sections describe the effect of each variable on the seismic permanent slope deformation.

a) The effect of PGA

Peak ground acceleration is a function of earthquake magnitude and the region's distance from the place of the earthquake occurrence. Moreover, the effects of the site are also considered based on T_s/T_m and T_s. For constant characteristics of slope and earthquake, the increase in PGA causes deformation of the slope increase for the same k_h value. For example, according to Fig. 10, if PGA=0.6, the seismic permanent deformation of an earth slope with a safety factor of 1.1,slope angle of 45 °, internal friction angle of 20° under the effects ofk_h= 0.1will be 35cm. In case of decreasing PGA from 0.6 to 0.3, deformation will decrease to 5 cm, and vice versa. In case of increasing in
PGA from 0.6to 0.9, deformation will increase to 75cm.

As can be seen, if FS is equal to or greater than 1 and the k_h is considered equal to PGA, the deformation of the slope will be negligible. Choosing this state for designing can lead to very conservative designs. Moreover, this has also been expressed by Kramer as: ''Use of peak ground acceleration as the seismic coefficient in conjunction with a safety factor of 1.0 has been shown to give excessively conservative assessments of slope performance in earthquakes'' [10].

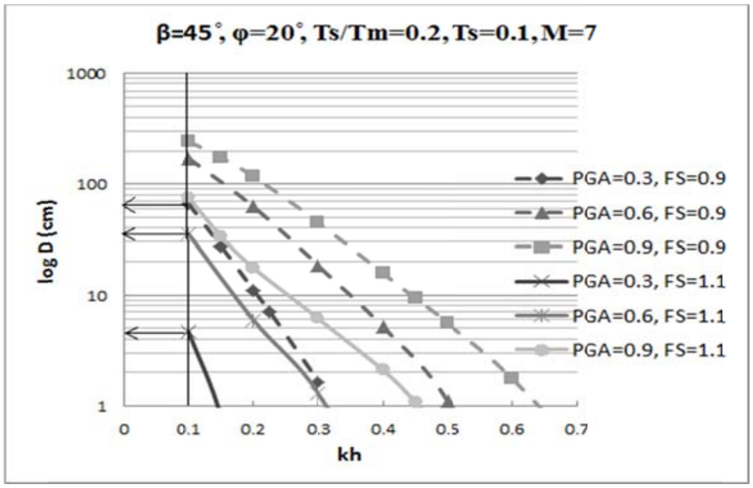

Fig. 10. Effect of PGA on seismic slope displacement

b) The effect of T_s/T_m

Figure 8 presents the model predictions of k_{max}/PGA as a function of input PGA and T_s/T_m. Generally, k_{max}/PGA is greater than 1.0 at smaller values of T_s/T_m, and then falls below 1.0 at larger period ratios. The range of T_s/T_m values that predict k_{max} greater than PGA (i.e., k_{max}/PGA>1.0) decrease with increasing PGA, and at large input intensities k_{max} is less than PGA at all period ratios. All curves predict k_{max}/PGA=1.0 for $T_s/T_m \leq 0.1$, i.e., rigid sliding conditions.

At large input ground motion of PGA=0.6, k_{max} is less than PGA at all period ratios. Assuming all the information in Fig.9 to be constant and gaining FS equal to 0.9,for k_h=0.2 the decrease inT_s/T_m from 0.2 to 0.1 decreases slope deformation from 75cm to 60cm, and increasing $T_s/T_m$from 0.2 to 0.3 also reduces slope deformation from 60 cm to 45 cm.(Fig. 11).

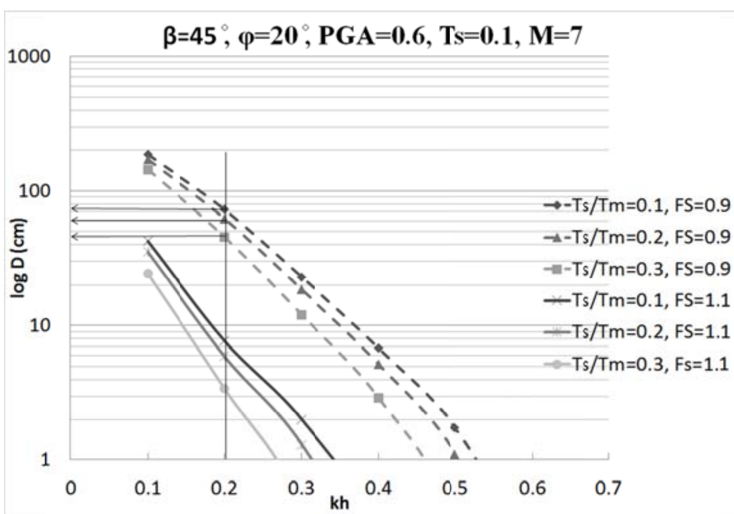

Fig. 11. Effect of T_s/T_m on seismic slope displacement

c) The effect of T_s

Slope seismic displacement increases with an increase in T_s up to 1.5 seconds, but increase of T_s more than 1.5 second will not have a significant effect on displacement. [22]

In the example shown in Fig. 9, considering FS=1.1 and k_h=0.1 , decreasing T_s from 0.1 to 0.05 causes a decrease in slope displacement from 35 cm to 30 cm, and increasing T_s from 0.1 to 0.2 increases displacement from 35cm to 50cm (Fig. 12). The increase in T_s is due to increase in earth slope height or a decrease in soil shear wave velocity.

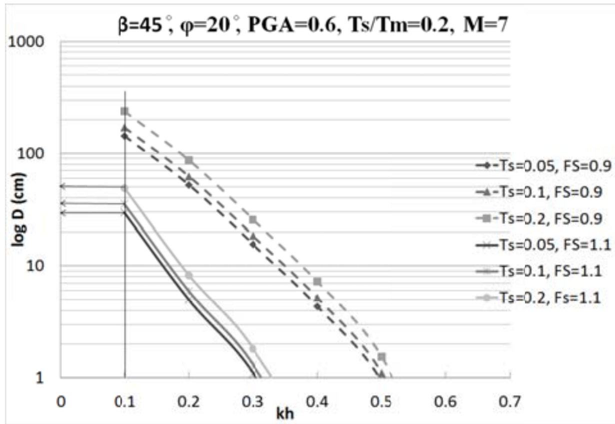

Fig. 12. Effect of T_s on seismic slope displacement

d) The effect of φ

Variations of the seismic displacement with changes in internal friction angle depend on the safety factor of the slope. For example, according to Figs. 9 and 13, if an earth slope with FS equal to 1.1 and characteristics shown in Fig. 9 is influenced by k_h of 0.2, the seismic displacement for soils with internal friction angles of 10˚, 20˚ and 30˚ will be 9, 6, and 7 cm, respectively. On the other hand, if an earth slope with FS is equal to 0.9 and characteristics shown in Fig. 9 are influenced by horizontal acceleration of 0.2, displacement for internal friction angles of 10˚, 20˚, and 30˚will be 50, 60 and 80cm, respectively.

It should be noted that effect of the internal friction on seismic displacement depends on the amount of other parameters affecting the problem. For instance, the influence of change in value of the horizontal acceleration in the above example will be taken into consideration. According to Figs. 9 and 13, if an earth slope with FS=1.1 and characteristics shown in Fig. 9 is analyzed by k_h=0.4, slope displacements for soils with internal friction angles of 10˚, 20˚, and 30˚ will be negligible. If FS reduces to 0.9, the slope displacements would be 4, 5, and 4cm, respectively. In general, variations in soil internal friction angle will have a significant impact on the k_y.

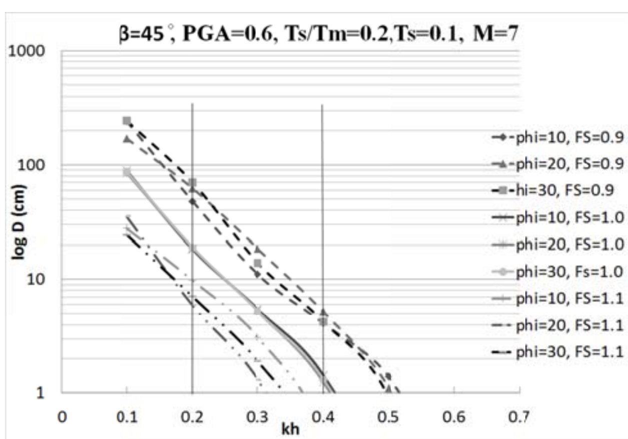

Fig. 13. Effect of φ on seismic slope displacement (PGA=0.6g)

e) The effect of β

According to Euro Code 8, based on a simplified procedure developed by Ashford et al. [27] the seismic loading can be amplified for steep (>60°) slopes (i.e., ∼1.5 *PGA*). [27]

f) Verification and example applications

To illustrate the verification and application of the model for predicting the sliding displacement of slopes with specified safety factor, consider the following example:

Consider two slope Models (M1 and M2) on Fig. 4, with the height of 30m from bedrock, angle of 35° and soil unit weight of 20 kN/m^3, having factor of safety (FS) equal to 1.1 and 1.0 respectively under horizontal acceleration of 0.139 g. Regarding the above mentioned specifications, deformation of slopes using the proposed Model can be determined and dynamic analysis using PLAXIS code[25] under effects of earthquakes can be observed in Table 3. The records are those referred to in Table 2.

Last two columns of Table 3 illustrate the seismic slope deformations obtained from finite element method (PLAXIS) and predicted deformations from the proposed Model. Run time of the PLAXIS code for calculation of slope permanent deformation in each phase is approximately 8 hours. In this case, curves are able to introduce suitable provision on determination of seismic slope deformation with a given factor of safety under effect of a given horizontal acceleration.

Table 3. Earth slopes permanent deformation

Record No.	Model No.	M	k_y	k_h	FS	T_s/T_m	U(cm) PLAXIS	U (cm) Model	Displacement reduction (PLAXIS)%	Displacement reduction (Model)%
Tabas-1084L	M1	7.4	0.139	0.139	1.0	0.65	97	78.55	-	-
Tabas-1084T	M1	7.4	0.139	0.139	1.0	0.63	79	87.59	-	-
Ab bar-1362T	M1	7.7	0.139	0.139	1.0	1.03	22	17.28	-	-
Zanjiran-1502L	M1	6.4	0.139	0.139	1.0	2.03	5.1	0.0	-	-
Zanjiran-1502T	M1	6.4	0.139	0.139	1.0	1.74	17	0.07	-	-
Bam -3168T	M1	6.7	0.139	0.139	1.0	0.81	22	18.11	-	-
Tabas-1084L	M2	7.4	0.165	0.139	1.1	0.88	23	24.47	76	69
Tabas-1084T	M2	7.4	0.165	0.139	1.1	0.86	31	28.28	61	68
Ab bar-1362T	M2	7.7	0.165	0.139	1.1	1.38	1.4	0.31	94	98
Zanjiran-1502L	M2	6.4	0.165	0.139	1.1	2.73	0.0	0.0	100	-
Zanjiran-1502T	M2	6.4	0.165	0.139	1.1	2.35	0.6	0.0	96	100
Bam -3168T	M2	6.7	0.165	0.139	1.1	1.1	9.5	3.02	57	83

As apparent in the last two columns of Table 3 and the Figures, increasing slope's safety factor from 1 to 1.1 brings about >50% reduction in seismic deformation. This reduction percentage for deformations with < 10cm is much higher. Often, the results of such probabilistic assessments lead to better engineering decisions [28].

6. DETERMINING THE HORIZONTAL ACCELERATION COEFFICIENT FOR DESIGNING

The charts in Fig. 14a-d are prepared for PSF values of 0.9, 1.0, 1.1 and 1.2 and for four different PGA values. Other charts can be produced by users with different FS and PGA values.

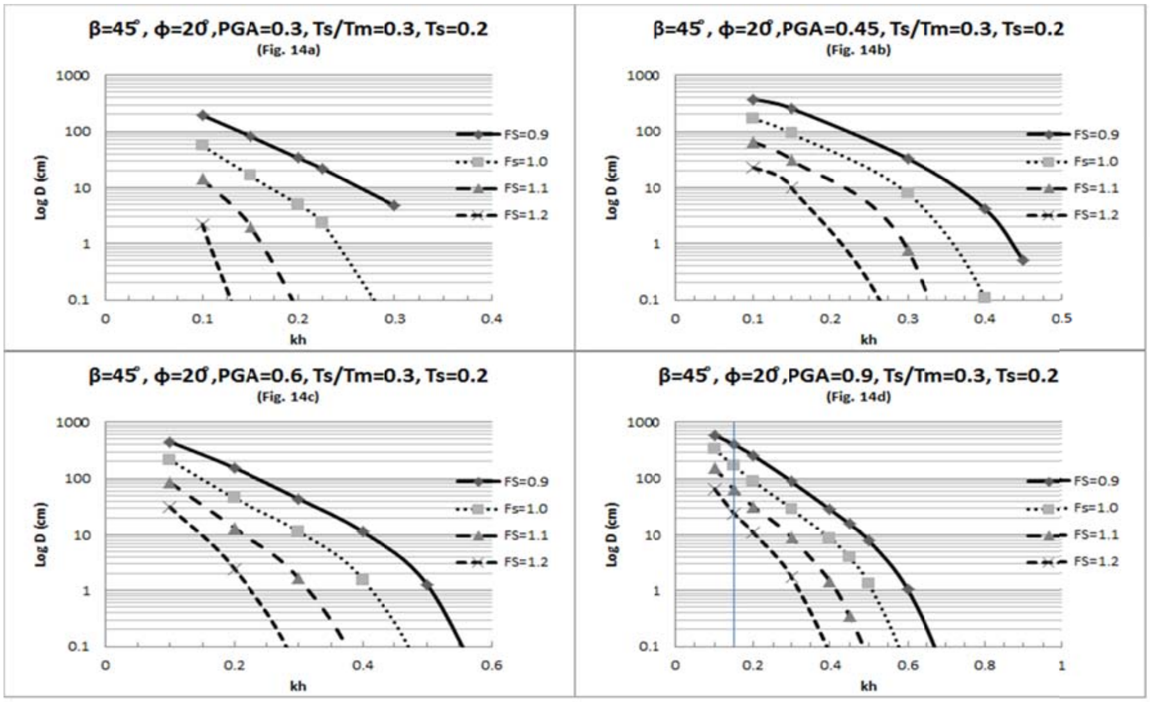

Fig. 14a-d. Seismic coefficients versus anticipated slope displacements for different PGA values. (ε=1)

The evaluations based on Fig. 14a-d are listed as follows:

- Both FS and $k_h$values govern the magnitude of the anticipated displacement. With higher FS values, lower slope displacement values are encountered.
- For the same $k_h$value, magnitude of the slope displacement depends on peak acceleration value.
- If k_h=0.5, then PGA/g is used in conjunction with PSF=1.0, no displacements should be expected. This may be overconservative for many civil engineering works. This is consistent with the current literature that ''use of peak ground acceleration as the seismic coefficient in conjunction with a safety factor of 1.0 has been shown to give excessively conservative assessments of slope performance in Earthquakes (Kramer, 2004)''.
- If k_h=0.5PGA/g along with FS of 1.0, the displacements will be lower than 20 cm for all PGA values studied in this paper. This is consistent with the findings of Hynes and Franklin [18] that showed ''seismic coefficient values as a ratio of peak acceleration such as k_h=0.5PGA/g should be used for 30 cm displacement''.
- In Iran, earthquake code recommends effective peak accelerations of 0.5g for the Seismic Resistant Designs of roads, railways and bridges. According to Iran Road and Railway, Bridges Seismic Resistant Design Code NO: 463 " in Iran, seismic coefficient is taken as a value between 0.05 and 0.125depending on the earthquake zone". k_h values ranging between 0.05 and 0.125 along with a FS of 1.0 will result in low displacement values (<10 cm) for PGA≤0.3g. However, for PGA>0.3g, higher displacements should be expected. Therefore based on the value of the anticipated peak acceleration, higher $k_h$values should be recommended.
- Seed recommended that ''it is necessary to perform a pseudo static analysis for a seismic coefficient of 0.1 for earthquakes of 6.5 magnitude or 0.15 for 8.25 magnitude earthquakes and obtain a safety factor of the order of 1.15 to ensure that displacements will be acceptably small''. Seed acknowledged about 1m of displacement as acceptable, since he was dealing with earth dams. For k_h=0.1, and PGA<0.45g, displacements lower than 30cm are predicted with Seed's recommendations of $k_h$and FS. When the

k_h value of 0.15 and FS of 1.15 is applied with PGA=0.90g, the proposed methodology in this paper predicts maximum displacement of 140cm. This means that even for high PGA values,
Seed's recommendation may be un-conservative.

- Magnitude of the slope displacement depends on the maximum horizontal equivalent acceleration (k_{max}) which depends on PGA and T_s/T_m .
- The Newmark-type rigid block analysis was overly conservative for cases where the T_s/T_m was greater than about 0.5 and un-conservative when T_s/T_m was in the range of 0.2–0.5 for PGA $< 0.5\,g$.When PGA > 0.5 g Newmark-type rigid block analysis was overly conservative for all of T_s/T_m.PGA and T_s/T_m have significant effect on k_{max} and slope displacement.

7. CONCLUSION

The pseudo static approach is a well-known method to calculate the seismic stability of slopes. However, in today's practice, performance based design concept necessitates the anticipated displacements to be known instead of a single pseudo static safety factor (FS). Although there are also several recommendations for selection of k_h value, many of them depend on judgment and expertise.

Based on the provided charts, while designing an earth slope under specific k_h and FS, its performance can be taken into consideration, simultaneously. Design of slopes without consideration of its performance may be conservative or underestimated. This kind of application is a novel approach. Calculated displacements are then investigated in terms of k_y/PGA values and being consistent with the literature. Earthquake induced slope displacements are found to be very sensitive to the value of the yield accelerations. Displacements decrease with increase in the acceleration ratios. Several equations are derived for all data and for different earthquake magnitude values (M) with and without distance constraint and for different peak acceleration (PGA) ranges. The obtained equations are compared with literature and it is shown that categorizations for earthquake magnitude, distance to the epicenter and peak acceleration are important and effective tools.

Based on the results, a methodology is presented in the context of a coupled displacement pseudo-static safety factor–seismic coefficient analysis. The results revealed that seismic coefficient for any allowable displacement should be based on anticipated PGA values. Use of high FS values results in lower displacements. It is also found that use of k_h=PGA/g in conjunction with FS=1.0 results in negligible displacement and it may be over conservative for any civil engineering works. If k_h=0.5PGA/g along with FS of 1.0, displacements will be lower than 30cm for all PGA values studied in this paper, which is consistent with the well-known findings of Hynes and Griffin [18] . Evaluations are also made for Seed's recommendations and it is shown that Seed's criterion limits the displacement values to about 100cm for accelerations as high as 0.7g.

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