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IMPROVED METHODOLOGY FOR ESTIMATING SEISMIC COEFFICIENTS FOR THE PSEUDO-STATIC STABILITY ANALYSIS OF EARTH DAMS

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ABSTRACT

This paper presents an improved methodology for estimating seismic coefficients for the pseudo-static stability analysis of earth dams, which is based on a statistical analysis of input data and results for 112 potential failure surfaces, as estimated from 28 two dimensional seismic response analyses for eight (8) different zoned earth dams and high embankments. The new methodology employs design diagrams and equations and estimates the maximum and the effective seismic coefficients as a function of: (a) the peak ground acceleration at the free-field surface of the foundation soil, (b) the predominant period of the seismic excitation, (c) the eigenperiod of the earth dam, (d) the dam foundation conditions, and (e) the dimensionless ratio z/H of the maximum depth z of the failure surface over the height H of the earth dam. The proposed methodology offers accuracy and consistency with a standard deviation of the relative error in the estimation of the seismic coefficients in the order of ±24%

INTRODUCTION

The seismic response of earth dams is a complicated problem of Earthquake Geotechnical Engineering. This is due to the fact that a proper analysis should take into account seismic ground motion amplification phenomena, the development of inertial forces in the dam, the changes in the stiffness and the shear strength of its construction materials, etc. According to ANCOLD (1998) guidelines, the seismic stability of the slopes of earth dams may be estimated using the following methods:

(a) Pseudo-static limit equilibrium analysis
(b) Simplified estimation of slope displacements
(c) Total or effective stress numerical analysis

The emphasis in this paper is put on the first of the foregoing methods, which was practically the sole method of analysis until the beginning of the 1970's, and leads to the estimation of factor of safety FS_d against seismic "failure" of the slope. The problem at hand is presented schematically in Figure 1, where various problem parameters are defined, such as the peak ground acceleration at the crest, $a_{max,crest}$, and at the base of the dam, $a_{max,base}$, as well as the values of the peak ground acceleration at the free-field of the dam foundation soil, PGA, and its respective value at the outcropping bedrock PGA_b (where PGA_b = PGA, in cases that the dam is founded on stiff rock).

Based on this simplified method of analysis, values of FS_d larger than 1.0 imply seismically safe conditions, while values of the same factor smaller than 1.0 imply seismic "failure" of the slope. The amount of reduction of the factor of safety from its value FS under static conditions to its value FS_d for the pseudo-static conditions depends mostly on the value of the horizontal inertial force F_h that is applied at the center of weight of the sliding mass of the slope. As depicted in Figure 1, this force F_h is equal to the weight W of the sliding mass multiplied by a dimensionless seismic coefficient k_h. Based on all the above, the selection of an appropriate value of k_h for use in the estimation of FS_d is of utmost importance for the rational and safe seismic design of an earth dam.

In concept, the k_h coefficient should reflect the vibration of the sliding mass for the design earthquake. If the sliding mass was rigid, then the maximum value of the seismic coefficient k_h could be correlated to the peak value of the mass acceleration a_{max}, according to $k_h = a_{max}/g$. Nevertheless, given that the sliding mass is not rigid, its various points do not vibrate in phase, and therefore a representative value for k_h should be smaller than a_{max}/g, as for example a value corresponding to the average value of the peak acceleration of all points within the sliding mass. Nevertheless, even such a choice could be problematic since these peak values are not observed concurrently. In reality, this out-of-phase vibration of various points within the sliding mass is a significant complication, especially in tall earth dams, where the predominant wave length of the seismic waves is comparable to their height.

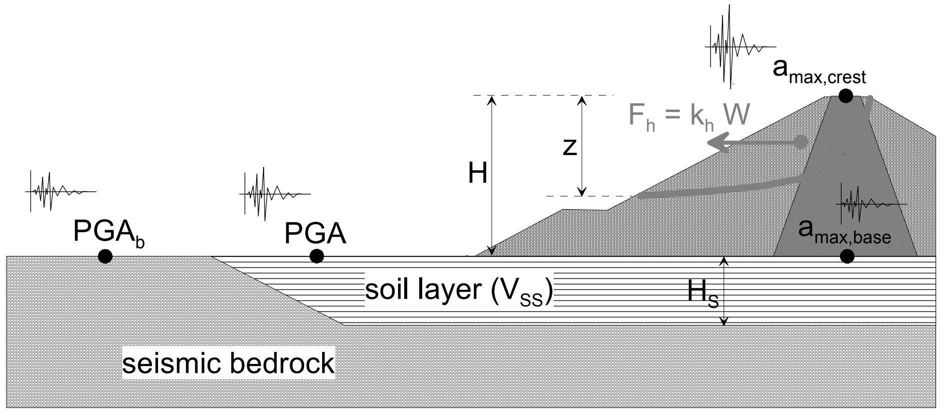

Fig 1. Definition of important parameters of the problem of seismic response and stability analysis of earth dams and tall embankments

Moreover, if one considers the dam as a multi-degree of freedom vibrator undergoing base excitation, then its vibration, as well as that of any sliding mass within, is affected by the vibration of the foundation soil in a complex soil-structure-interaction system. Based on all the above, the peak value of the acceleration within the sliding mass a_{max} may be correlated with the peak values of the acceleration at various points of the dam (e.g. $a_{max,crest}$, $a_{max,base}$) and/or the free ground or rock surface (e.g. PGA, PGA_b), but does not coincide with any of the foregoing peak values.

Note that the aforementioned values of acceleration a_{max}, $a_{max,crest}$, $a_{max,base}$, PGA, PGA_b, as well as the seismic coefficient k_h with whom they may be correlated, are maximum values and therefore they are only observed momentarily during seismic shaking. Hence, use of pseudo-static analysis with a value of the seismic coefficient equal to k_h and a concurrent requirement of $FS_d \geq 1.0$ is greatly conservative. For this reason, it has become common practice to use "effective" values of acceleration and of the seismic coefficient, along with a concurrent requirement of $FS_d \geq 1.0$ at the cost of "small" downslope seismic displacements. The magnitude of these "small" seismic displacements may be estimated in a simplified manner with various literature methods, which are generally based on the sliding block method (e.g. Newmark 1965, Richards & Elms 1979), an issue that is beyond the scope of this paper.

In the paragraphs that follow, the effective values of the accelerations and of the seismic coefficient are denoted by a, a_{crest}, a_{base}, EGA, EGA_b and k_{hE} respectively, and are considered to be a percentile of their respective peak values presented in Figure 1. More specifically, the literature values of the ratio of the effective over the peak value of acceleration range from 0.50 (e.g. Hynes-Griffin and Franklin 1984 for pseudo-static slope stability analyses) to 0.80 (e.g. in the definition of elastic response spectra according to the Greek Seismic Code EAK 2002 and the EC-8). The most commonly used value of this ratio is equal to 0.65 – 0.67 [e.g. in the British Standards for pseudo-static analyses of the slopes of earth dams or tall embankments (Charles et al 1991), in the liquefaction resistance analysis according to Youd & Idriss 2001].

Based on all the above, the effective value of the seismic coefficient k_{hE} that is to be used in pseudo-static slope stability analyses of earth dams along with a requirement for $FS_d \geq 1.0$ is much smaller than the average value of the peak acceleration of all points within the sliding mass. In this manner, one takes into account the out-of-phase vibration of various points within the sliding mass, as well as the fact that real seismic motions have variable intensity and their peak acceleration is only observed momentarily.

To our knowledge, there is no universally acceptable methodology for estimating seismic coefficients for use in the pseudo-static analysis of earth dams. Therefore, each designer uses a different methodology, based on his experience. Hence, in the paragraphs that follow, a critical review of existing methodologies is presented and an evaluation of their accuracy in comparison with pertinent numerical results. The latter originate from two dimensional seismic response numerical analyses of earth dams and are then used for depicting the critical problem parameters and for presenting an improved methodology for the approximate estimation of seismic coefficients.

It should be mentioned that the numerical analyses used for the proposal of the improved methodology refer to earth dams, as well as tall embankments. Therefore, the proposed methodology may also be used for the design of tall embankments having a trapezoidal cross section, and not only earth dams as mentioned in the title of the paper for reasons of brevity.

LITERATURE SURVEY

Correlation of the seismic coefficient with local seismicity

The first reference for selecting seismic coefficients may be attributed to Terzaghi (1950), who depicted values of k_{hE} =

0.1, 0.2 and 0.5 for «severe», «violent, destructive» and «catastrophic» earthquakes, respectively. In practice, and until the mid of the 1970's, the depiction of k_{hE} was based on (local) experience and led to values from 0.10 to 0.15 usually, with the assumed value increasing as a function of the design earthquake magnitude M or the importance of the civil engineering work, without exceeding a value of 0.20 (e.g. old design guidelines of US Corps of Engineers).

Correlation of the seismic coefficient with the PGA

Based on technical report from USCOLD (1985), the usual practice until the mid 80's in USA was to use k_{hE} ranging from 0.25(PGA/g) to PGA/g, with the largest values taking into account the elastic amplification of the motion within the body of the dam. Similarly, the respective British Standards (Charles et al 1991) propose the use of k_{hE} = 0.67(PGA/g), which implies that they consider the PGA being equal to the peak acceleration of any sliding mass within the dam.

It is mentioned here that a similar correlation of the seismic coefficient with the PGA is also proposed by seismic codes (e.g. EC-8 and the Greek code EAK 2002) for the pseudo-static analyses of slopes. For example, EC-8 proposes the use of k_{hE} = 0.5(EGA/g), with EGA (= 0.8 PGA) taking into account site amplification according to the ground category. Moreover, EC-8 also takes roughly into account the maximum depth z (measured from the crest) of the failure surface as compared to the height H of the slope and this due to topographic amplification of the seismic motion in the vicinity of earth slopes. In particular, it is proposed that the foregoing estimate of k_{hE} increases linearly from its minimum foregoing value when z = H, to its maximum value being 20% to 40% higher for really shallow failure surfaces ($z \rightarrow 0$). The figure shows that these code provisions under-estimate the numerical results. Nevertheless, it should be mentioned here that these code provisions should be evaluated with caution, since they are not intended for use in earth dams or tall embankments.

Correlation of the seismic coefficient to $a_{max,crest}$

The work of Makdisi & Seed (1978) must be considered as a very important contribution in this field. The reason for this is that they first correlated the value of the maximum seismic coefficient k_h not only to the value of ($a_{max,crest}$/g), but also to a decreasing function of the ratio of the maximum depth z of the failure surface (measured from the crest) over the height H of the dam (see also Figure 1). The benefit of the foregoing correlations is that in this way the k_h takes into account the vibration of the dam (which is not depicted by PGA), but also the geometric characteristics of the sliding mass. Obviously, since k_{hE} = (0.5 to 0.8)k_h, the foregoing methodology may also be used for the estimation of the effective seismic coefficient k_{hE}. In addition, according to Marcusson (1981), the slope stability of earth dams should be performed with k_{hE} values ranging from 0.33 to 0.50 of ($a_{max,crest}$/g), which roughly correspond to values of k_h = (0.50 to 0.75)($a_{max,crest}$/g).

Use of the aforementioned correlations creates the practical problem of estimating the $a_{max,crest}$, a value that is not equal to PGA or the PGA_b that may be known from the seismic hazard study for the site. In general, the accurate estimation of $a_{max,crest}$ requires the execution of non-linear numerical analyses, like the ones used in this paper. Alternatively, an approximate estimate may be obtained by using anelastic response spectra for the free-field surface of the dam foundation soil, which should be available from the aforementioned seismic hazard study. In doing so, the value of $a_{max,crest}$ may be estimated by taking into account the first 2-3 modes of vibration of the dam.

EVALUATION OF LITERATURE METHODOLOGIES

In this section, an evaluation of the foregoing literature methodologies is performed on the basis of pertinent numerical data. For this purpose the authors compiled input parameters and results from a number of 2D numerical analyses of the seismic response of real earth dams and tall embankments that were performed as part of consulting efforts over the last 10 years. These analyses took into account the non-linear soil behavior, and employed either the finite element (e.g. by using QUAD4M, Hudson et al 1994), or the finite difference method (e.g. by using FLAC, Itasca Inc 1998). In the sequel, the aforementioned literature methodologies were bluntly applied and their predictions for the seismic coefficients were compared to the respective numerical data.

More specifically, the compiled 2D analyses refer to twelve (12) cross sections from eight (8) earth dams and tall embankments, with height H ranging from 20 to 120m, each of which was analyzed by applying to its base up to 4 different seismic excitations. On the whole, input parameters and results from 28 numerical analyses were compiled in a database, which pertain to seismic excitations with intensity PGA = 0.16 to 0.37g and predominant period T_e = 0.13 to 0.49sec. In all compiled analyses the mesh discretization continued to large depths and widths away from the dam itself and the seismic excitation was applied uniformly to the base nodes of the mesh. Appropriate boundary conditions were applied at the bottom and lateral boundaries of the mesh [e.g. free field boundaries in the lateral boundaries of analyses employing FLAC (Itasca Inc 1998)]. The non-linear soil response was taken into account either via the equivalent-linear method (e.g. when employing QUAD4M, Hudson et al 1994) or via a truly non-linear constitutive law (e.g. using the User-Defined-Model capability in FLAC, Itasca Inc 1998).

From each numerical analysis, the maximum value of the seismic coefficient k_h was estimated for 2 up to 5 failure surfaces, thus creating a database for 112 failure surfaces in total. It is noted that in each case the maximum value of the seismic coefficient k_h was estimated on the basis of the maximum value of the resultant horizontal acceleration time history of the mass included by the failure surface in question. In the evaluation figures that follow, wherever necessary, the effective value of the seismic coefficient k_{hE} is estimated by $k_{hE} = 0.67\ k_h$. Furthermore, it is noted, that for the cases of dams 7 & 8 (see upcoming figures), the estimation of the seismic coefficients k_h and k_{hE} is partly based on the methodology of Makdisi & Seed (1978) and therefore their values are included in the database merely indicatively.

Figure 2 evaluates the empirical estimates for the seismic coefficient k_{hE} that stem from a rough estimate of local seismicity. From the comparison of empirical estimates to numerical results it is concluded that the usual empirical values of the seismic coefficients (k_{hE} = 0.10 to 0.20) are considered safe options for values of PGA ≤ 0.30g, but may prove intensely non conservative for earth dams or tall embankments that are designed against earthquakes of larger intensities. In addition, this figure shows that there is an increasing effect of PGA on the value of k_{hE}. Yet, the large scatter of the numerical results shows that other problem parameters must exist, apart from PGA, which should be taken into account when estimating a value for the seismic coefficient.

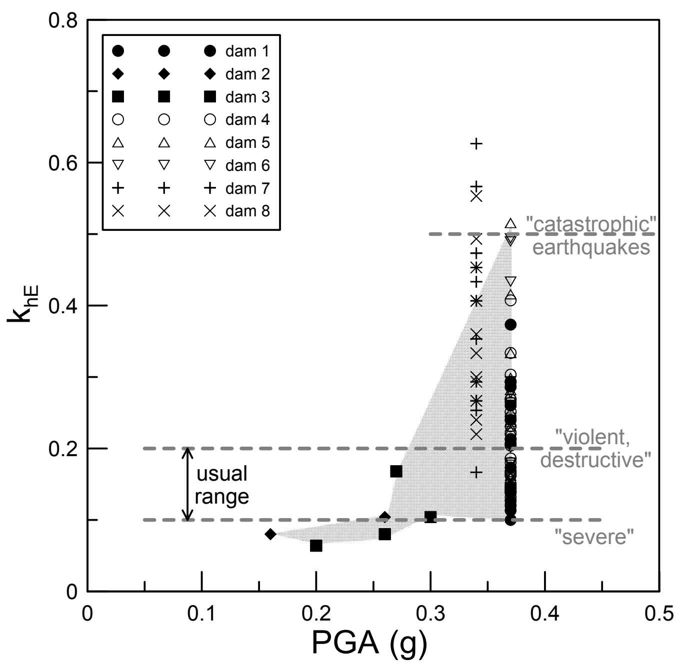

Fig. 2. Evaluation of empirical estimates for k_{hE}, on the basis of numerical data

Similarly, Figure 3 evaluates the estimates for the seismic coefficient k_{hE} that are based on correlations to the peak acceleration at the free-field of the foundation soil PGA. From the comparison of the foregoing estimates to numerical results it is deduced that the correlation of the effective seismic coefficient k_{hE} with the PGA is rational, and therefore reduces the scatter as compared to that in Figure 2. In addition, this figure studies whether there is an additional effect of the normalized depth z/H (see Figure 1 for definition). This figure shows clearly that an increase in the normalized depth z/H reduces consistently the value of the seismic coefficient for the same PGA level. From a quantitative point of view, it should be mentioned that the proposal of the British Standards (Charles et al 1991) is sufficiently conservative for failure surfaces of medium to large depth (z/H ≥ 0.4), but underestimates the seismic coefficient for shallower failure surfaces. In addition, it is observed that the range of empirical estimates compiled by USCOLD (1985) includes the majority of the numerical results, with the exception of failure surfaces of very small depth. Nevertheless, this agreement is of little practical importance, since the denoted range of variation is quite large.

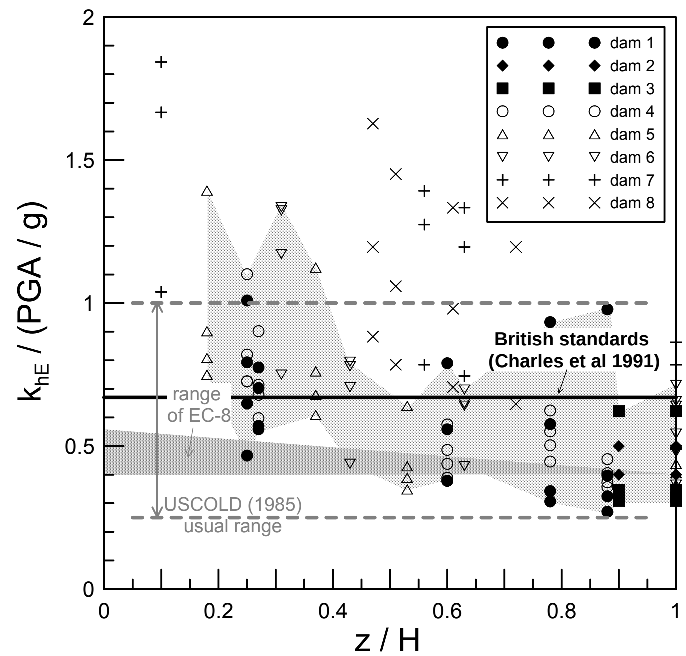

Fig. 3. Evaluation of estimates for k_{hE} related to the PGA, on the basis of numerical data

Finally, Figure 4 evaluates the estimates of the maximum seismic coefficient k_h that are based on correlations to the maximum acceleration at the dam crest $a_{max,crest}$ and the normalized depth z/H. Observe that the correlation of the maximum seismic coefficient k_h to $a_{max,crest}$ reduces even further the scatter of numerical results, as compared to the pertinent correlation to PGA presented in Figure 3. In addition, this figure shows that the reducing effect of the normalized depth z/H to the value of k_h, also depicted in Figure 3, remains consistent. With respect to the pertinent proposals from the literature it is observed that the guidelines of Marcusson (1981) are over-simplistic and lead to conservative estimates of k_h for failure surfaces of medium to large depth (with z/H > 0.30). On the contrary, the proposal of Makdisi & Seed (1978) is qualitatively accurate, and leads to accurate estimates of k_h for deep failure surfaces (z/H ≥ 0.70).

Yet, for shallower surfaces, their proposal seems to overestimate significantly the numerical estimates of the maximum seismic coefficient (up to 100%).

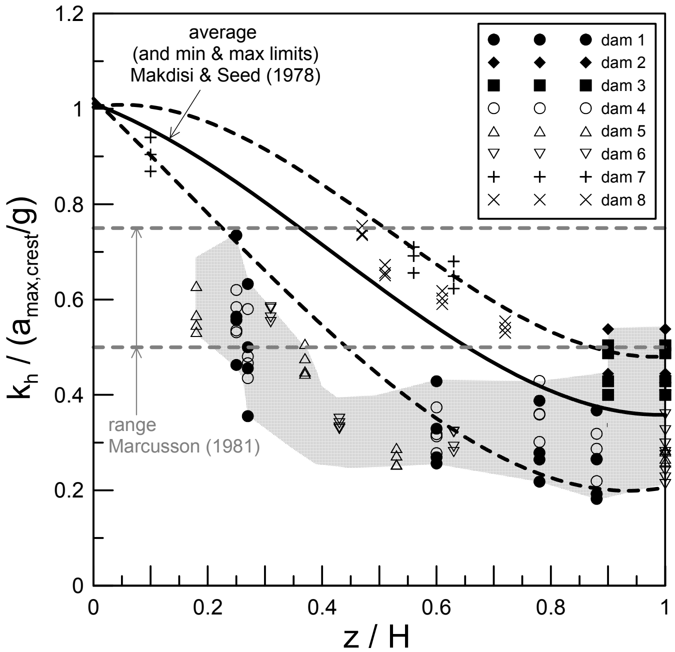

Fig. 4. Evaluation of estimates for k_{hE} related to the $a_{max,crest}$, on the basis of numerical data

PARAMETRIC EVALUATION OF NUMERICAL RESULTS

Based on the findings of the previous section, none of the literature methodologies that were studied agrees with the numerical results for the whole range of problem parameters. Hence, in order to propose an improved methodology that agrees well with the numerical results, this section studies which variables affect the significant problem parameters, namely:

- the ratio $k_h/(PGA/g)$, which allows for estimating the seismic coefficient on the basis of seismological data and the dam foundation conditions, and
- the ratio $k_h/(a_{max,crest}/g)$, which leads to estimates of the seismic coefficient by taking into account the vibration of the dam itself.

It is noted here that the emphasis is put on the maximum value of the seismic coefficient k_h that is calculated directly from the numerical analyses. An effective value for the seismic coefficient k_{hE} may always be estimated on the basis of the approximate relation k_{hE} = (0.5 to 0.8) k_h, according to the literature.

Estimation of seismic coefficient on the basis of PGA

As deduced by Figure 3, the correlation of the seismic coefficient to the value of PGA and the normalized depth z/H is satisfactory and relatively simple. Moreover, it takes into account roughly the effect of dam foundation soil conditions via PGA that is different, in general, from PGA_b. Hence, Figure 5 presents the design diagram for the maximum seismic coefficient k_h as a function of PGA and z/H, with the dashed line being considered as an indicative best fit relation given analytically by the following equation (1):

$$\frac{k_h}{(PGA/g)} = \begin{cases} 1.3 & , \quad 0.0 \le z/H \le 0.3 \\ 1.3 - 2.17(z/H - 0.3) & , \quad 0.3 \le z/H \le 0.6 \\ 0.65 & , \quad 0.6 \le z/H \le 1.0 \end{cases} \qquad (1)$$

Overall, the foregoing approximation is considered simplistic, since it does not take into account the dam vibration. Furthermore, it is considered approximate, especially for shallow failure surfaces (z/H ≤ 0.4).

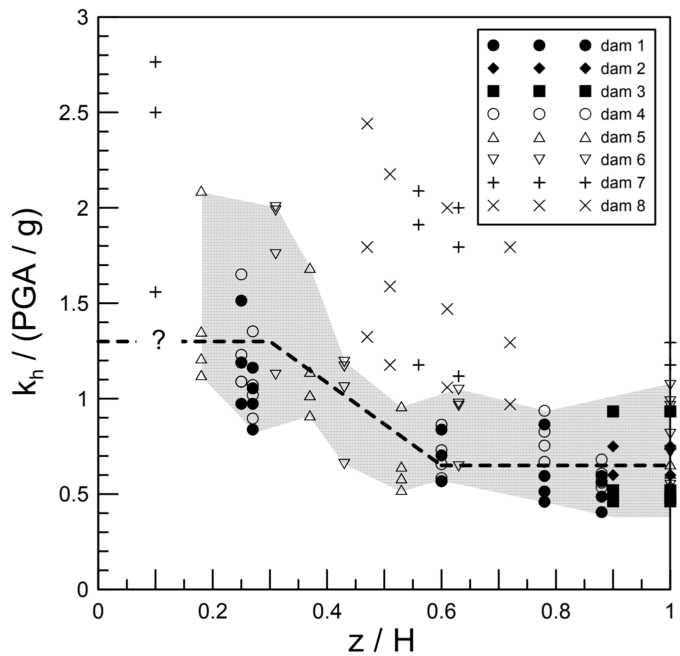

Fig. 5. Design chart for estimating the maximum value of the seismic coefficient k_h as a function of PGA and z/H, on the basis of numerical data

Estimation of seismic coefficient on the basis of $a_{max,crest}$

As deduced by Figure 4, the correlation of the seismic coefficient to the value of $a_{max,crest}$ and the normalized depth z/H is more than satisfactory. Hence, Figure 6 presents the design diagram for the maximum seismic coefficient k_h as a function of the two foregoing parameters, with the solid line depicting the best fit relation given analytically by the following equation (2):

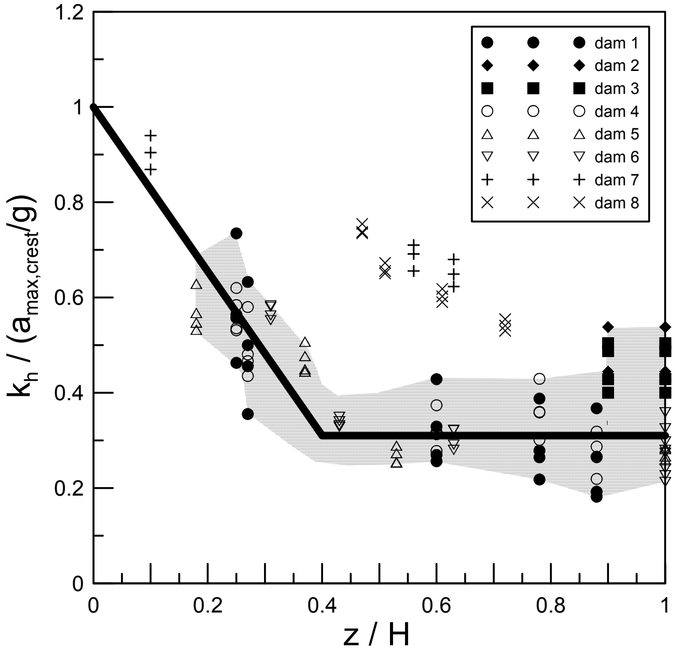

Fig. 6. Design chart for estimating the maximum value of the seismic coefficient k_h as a function of $a_{max,crest}$ and z/H, on the basis of numerical data

$$\frac{k_h}{(a_{max,crest}/g)} = \begin{cases} 1-1.725(z/H) & , \ 0.0 \le (z/H) \le 0.4 \\ 0.31 & , \ 0.4 \le (z/H) \le 1.0 \end{cases} \tag{2}$$

This approximation is more rational than Equation (1), since it takes into account the dam vibration (via $a_{max,crest}$), and leads to reduced scatter for all possible depths of the failure surfaces and especially the most shallow ($z/H \le 0.4$). Nevertheless, the practical problem for using Equation (2), as well as other similar methodologies from the literature (e.g. Marcusson 1981, Makdisi & Seed 1978), is that one needs a rational and relative simple way of estimating $a_{max,crest}$.

As a first approximation, the value of $a_{max,crest}$ could be correlated to $a_{max,base}$, i.e. the peak acceleration at the base of the dam (see Figure 1). This correlation practically assumes that the dam is a vibrator undergoing base excitation and the estimation of $a_{max,crest}$ may be performed on the basis of a spectral analysis, as for example is performed for low-rise embankments (H < 15m) according to Greek seismic code EAK (2002). Hence, Figure 7a presents the effect of the first eigenperiod T_o of the dam on the value of the dimensionless acceleration ratio $a_{max,crest}/a_{max,base}$, while the same is performed in Figure 7b but as a function of the normalized eigenperiod T_o/T_e, where T_e is the predominant period of the seismic excitation. Based on these figures, the values of ratio $a_{max,crest}/a_{max,base}$ show a large range of variation (from 1.5 to 5.0!) and a clear differentiation in the values depending on the prevailing dam foundation soil conditions. Moreover, the scatter is significant irrespective of whether the correlation is on the basis of the eigenperiod T_o (and indirectly of the height H of the dam) or the normalized eigenperiod T_o/T_e that takes into account resonance phenomena during the vibration of the dam. Based on all the above, using the dimensionless ratio $a_{max,crest}/a_{max,base}$ for design is not considered appropriate.

Alternatively, the value of $a_{max,crest}$ may be correlated directly to PGA, an assumption that may lack the theoretical background of the foregoing correlation to $a_{max,base}$, but is easy to use in practice. Hence, Figure 8a presents the effect of the first eigenperiod T_o of the dam on the value of the dimensionless acceleration ratio $a_{max,crest}$/PGA and reveals significant scatter of the numerical results that is comparable to that of Figure 7a. On the contrary, the respective correlation of ratio $a_{max,crest}$/PGA to the normalized eigenperiod T_o/T_e in Figure 8b shows significantly reduced scatter and a clear differentiation of the amplification response due to the dam (as this is expressed via $a_{max,crest}$/PGA) depending on the dam foundation soil conditions. In particular, soft soil foundation conditions introduce higher radiation damping and therefore less amplification (i.e. smaller values of $a_{max,crest}$/PGA) according to these results. Based on the above, the use of the dimensionless acceleration ratio $a_{max,crest}$/PGA for design is considered appropriate and this is performed on the basis of the results shown in Figure 8b. Specifically, two design curves are defined, one for dams founded on rock or stiff soils (equation 3 below, solid line in Figure 8b) and another for dams or tall embankments founded on soft soils (equation 4, dashed line in Figure 8b):

$$\frac{a_{max,crest}}{PGA} = \begin{cases} 1+4.4\left(\frac{T_o}{T_e}\right) & , \ 0.0 \le \frac{T_o}{T_e} \le 0.5 \\ 3.2 & , \ 0.5 \le \frac{T_o}{T_e} \le 2.0 \\ 3.2\left(\frac{2T_o}{T_e}\right)^{2/3} & , \ 2.0 \le \frac{T_o}{T_e} \end{cases} \tag{3}$$

$$\frac{a_{max,crest}}{PGA} = \begin{cases} 1+0.8\left(\frac{T_o}{T_e}\right) & , \ 0.0 \le \frac{T_o}{T_e} \le 0.5 \\ 1.4 & , \ 0.5 \le \frac{T_o}{T_e} \le 2.0 \\ 1.4\left(\frac{2T_o}{T_e}\right)^{2/3} & , \ 2.0 \le \frac{T_o}{T_e} \end{cases} \tag{4}$$

It becomes obvious that Equations (3) and (4) borrow their form from code-related design spectra (e.g. EC-8). Nevertheless, they differ from usually employed code-related design spectra, since they explicitly take into account the predominant period of the excitation T_e. In particular, Equations (3) and (4) take their maximum values for normalized eigenperiods T_o/T_e around 1.0, thus underlining the importance of resonance phenomena in the seismic response of earth dams.

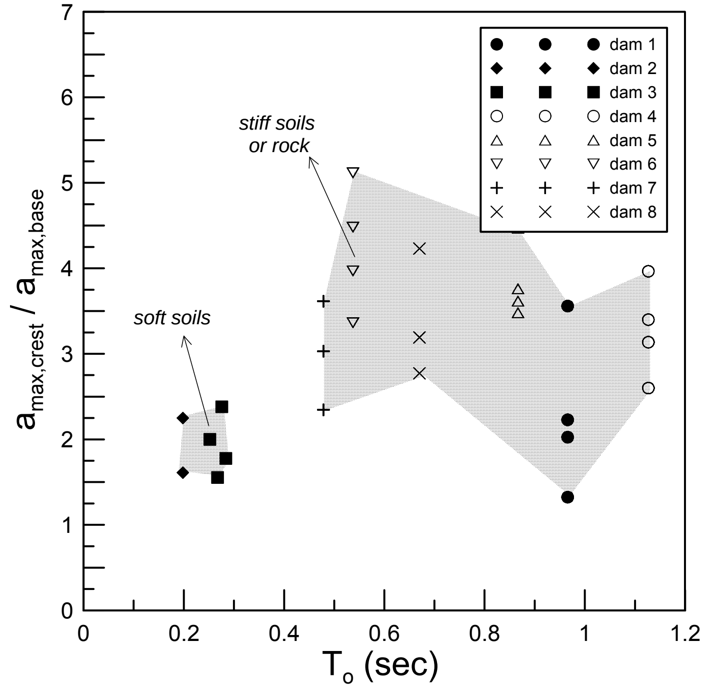

Fig. 7a   Correlation of $a_{max,crest}/a_{max,base}$ ratio to the (first) dam eigenperiod T_o, on the basis of numerical data

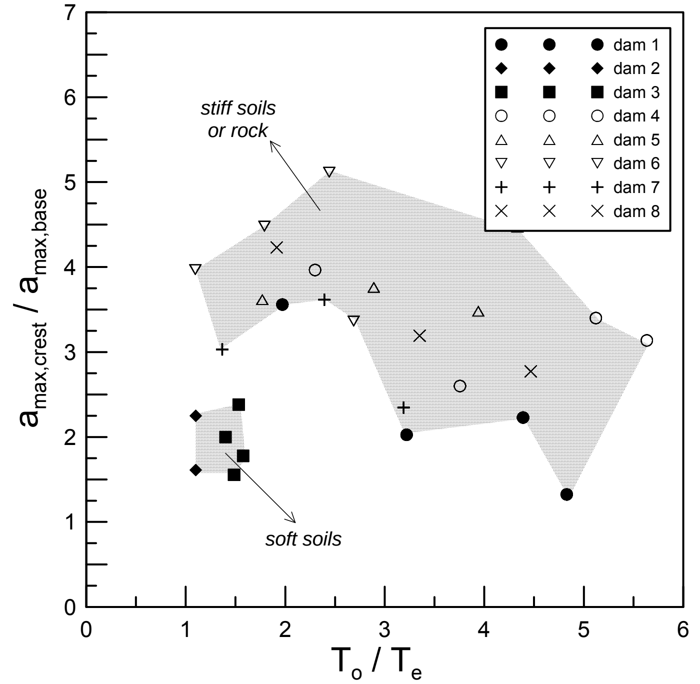

Fig. 7b   Correlation of $a_{max,crest}/a_{max,base}$ ratio to the normalized eigenperiod T_o/T_e, on the basis of numerical data

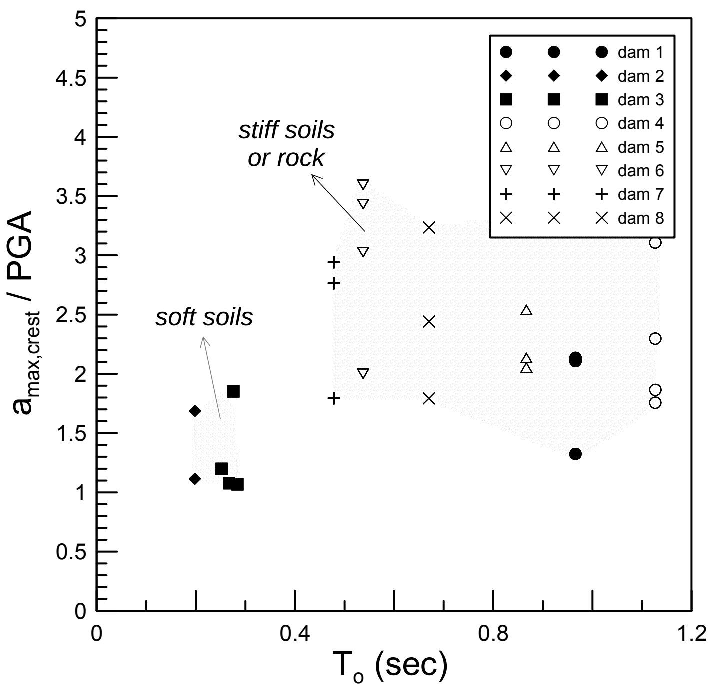

Fig. 8a   Correlation of $a_{max,crest}/PGA$ ratio to the (first) dam eigenperiod T_o, on the basis of numerical data

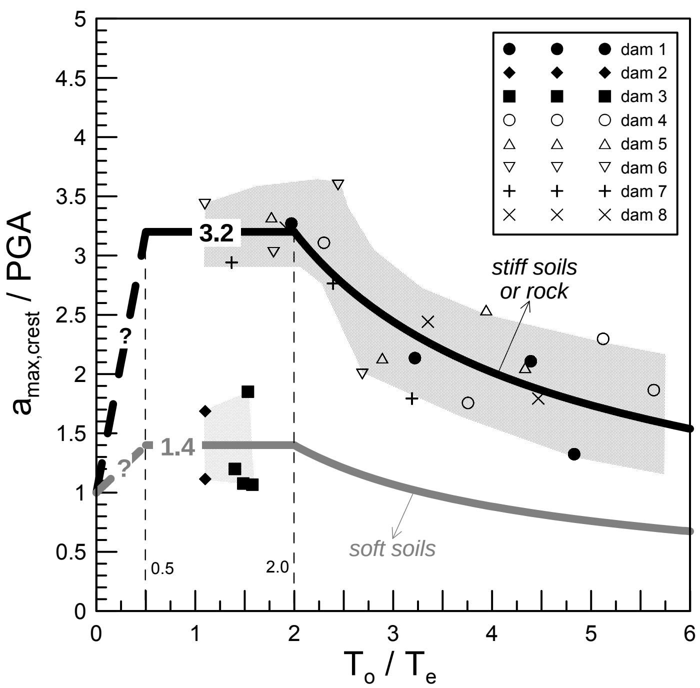

Fig. 8b   Design chart for $a_{max,crest}$ as a function of PGA, the dam foundation soil conditions and the normalized eigenperiod T_o/T_e, on the basis of numerical data

PROPOSED METHODOLOGY

Based on all the above, as well as those described in detail by Bouckovalas and Papadimitriou (2006), the proposed methodology for estimating seismic coefficients consists of five (5) steps that are described below:

Step 1: Estimation of PGA and predominant period T_e of the seismic excitation

The seismic hazard study for the earth dam in question proposes values for the peak ground acceleration (PGA_b) and the elastic response spectrum (for 5% damping) at the outcropping bedrock for the various design earthquakes (MDE, OBE, RIE). For any of the design earthquakes, the predominant period T_e may be estimated as the structural period (or the range of structural periods) leading to the peak spectral accelerations. The estimation of PGA is based on PGA_b, but should take into account the potential local amplification due to the foundation soil. Therefore, one may outline two cases:

(a) the earth dam is founded on rock, and therefore PGA = PGA_b,
(b) the earth dam is founded on a soil layer overlying rock (e.g. as shown in Figure 1).

In the second case, the estimation of PGA may be performed either via a numerical analysis [e.g. the equivalent-linear method employing SHAKE91 (Idriss & Sun 1992)] or using a simplified methodology, as for example the following relations, which are based on a simplification of the approximate methodology of Bouckovalas & Papadimitriou (2003):

$$PGA = PGA_b \frac{1 + 0.85\left(\frac{PGA_b}{g}\right)^{-0.17}\left(\frac{T_s}{T_e}\right)^2}{\sqrt{\left(1-\left(\frac{T_s}{T_e}\right)^2\right)^2 + 1.78\left(\frac{T_s}{T_e}\right)^2}} \qquad (5)$$

where T_S is the non-linear eigenperiod of the foundation soil layer, that is estimated via:

$$T_s = \left(\frac{4H_s}{V_{ss}}\right)\sqrt{1 + 5330\, V_{ss}^{-1.3}\left(\frac{PGA_b}{g}\right)^{1.04}} \qquad (6)$$

where:
H_S is the thickness of the soil layer (m), and
V_{SS} is the average (small-strain) shear wave velocity in the soil layer (m/s)

Step 2: Estimation of first eigenperiod of dam T_o

Simplifying the related analytical relations of Dakoulas and Gazetas (1985), the following relation is proposed for the estimation of the first eigenperiod T_o of a 2D dam having a trapezoidal cross section:

$$T_o = (2.6 + 2r)\frac{H}{V_s} \qquad (7)$$

where:
H is the height of the dam (m),
V_S is the average shear wave velocity of the dam (with emphasis in the central part of the cross section), and
r is the ratio of the width of the dam crest over the width of the base of the dam in the cross section at hand.

In practice, the r value in Equation (7) is generally small for earth dams and may become noteworthy (e.g. $r \geq 0.05$) only for tall embankments. The value given to V_S in Equation (7) depends on the type of the cross section (zoned earth dam or tall embankment), the height of the dam (due to the increase in effective stresses and thus shear stiffness with depth) and the amount of soil non-linearity introduced by strong seismic motions. In general, for small-strain conditions (low intensity motions) the V_S ranges from 250 to 350m/s for earth dams with cohesive core (the highest values for tall dams) and may exceed 350m/s for tall rockfill dams. An approximate relation for estimating average small-strain V_S values for zoned earth dams, which was based on the numerical results hereby compiled, is:

$$V_s\,(m/s) = 100\, H(m)^{0.25} \qquad (8)$$

The foregoing values of V_S must be reduced if one wants to take into account indirectly the soil non-linearity introduced by high intensity shaking. The amount of reduction may reach 50% for extremely strong seismic motions (e.g. PGA = 0.5g) that introduce resonance effects, but a reduction of 20 – 30% is more representative for most cases with strong ground motion.

Step 3: Estimation of $a_{max,crest}$ at the crest of the dam

The maximum acceleration at the crest of the dam $a_{max,crest}$ is estimated as a function of PGA (Step 1), the predominant period T_e of the seismic excitation (Step 1) and the first eigenperiod T_o of the dam (Step 2) according to:

- Equation (3) for dams founded on rock or stiff soil, and
- Equation (4) for dams (or most probably embankments) founded on soft soils.

The differentiation between stiff and soft foundation soil may be performed roughly on the basis of the site investigation data, or better on the basis of the ratio of PGA/PGA_b that was estimated in Step 1, via Eq. (5). More specifically, being conservative, a foundation soil may be considered soft, only if $PGA/PGA_b < 1.0$, which means that in most cases in practice Equation (3) is to be used, especially for dams. The opposite may occur for really soft soil conditions, as for example in the design of breakwater embankments.

Step 4: Estimation of the maximum value of the seismic coefficient k_h

The maximum value of the seismic coefficient k_h is estimated as a function of $a_{max,crest}$ (Step 3) and the normalized depth z/H (of the maximum depth z of the failure surface normalized over the height H of the dam) according to Equation (2). This value corresponds to the maximum value of the acceleration characterizing the vibration of the sliding mass and is therefore appropriate for use only for the estimation of seismic displacements using the sliding block method of analysis (e.g. Newmark 1965, Richards and Elms 1979).

Step 5: Estimation of the effective value of the seismic coefficient k_{hE}

As presented in the introduction, the ratio of the effective k_{hE} over the maximum value of the seismic coefficient k_h (Step 4) ranges between 0.5 and 0.8 in the literature. The designer must select an appropriate value of this ratio for use, and in practice, the most commonly used value is 2/3. This effective value is appropriate for use in pseudo-static slope stability analyses of earth dams, with concurrent requirement for $FS_d \geq 1.0$. It is underlined that such a design process leads to "small" (practically zero) downslope displacements. In general, the lower the k_{hE}/k_h ratio, the larger the expected slope displacements. Therefore, a correlation of the k_{hE}/k_h ratio to the allowable slope displacements would be a useful tool for future enhancements of the proposed methodology.

Reliability and limitations of methodology

In order to study the reliability of the proposed methodology, it was applied for an *a posteriori* estimation of the values of k_h for the 112 failure surfaces in the compiled database of numerical results. Hence, Figure 9 presents the effect of the normalized eigenperiod of the dam T_o/T_e on the ratio of the estimated k_h value using the proposed methodology over the respective k_h that was computed from detailed numerical analyses for all 112 cases in the database. Based on this figure it is deduced that the proposed methodology is accurate, with relatively small scatter in the results, since the standard deviation of the relative error in the estimation of k_h is equal to ±24%. In addition, the same figure also shows that the scatter is practically uniform and independent of the value of T_o/T_e, as well as other problem parameters (height H, dam eigenperiod T_o and normalized depth z/H), as shown in detail by Bouckovalas & Papadimitriou (2006).

The improved methodology is proposed for the estimation of seismic coefficients for cases of two dimensional (2D) earth dams and tall embankments that fall within the range of cases that were used for this study. In particular, the improved methodology is proposed for cases of:

- Earth dams and tall embankments with height H = 20 – 120m, triangular or trapezoidal cross section, with crest to base width ratio r = 0 – 0.1 (usual range), founded on variably different soil conditions,
- Seismic excitations with predominant period T_e=0.13–0.49s (usual range for Greece) and peak seismic acceleration at outcropping bedrock PGA_b=0.27-0.37g,
- Potential failure surfaces that pass through the central part of the cross section (core in zoned earth dams) and reach significant depths within the body of the dam, not allowing their simulation as infinite slopes.

In cases that the 2D assumption for the dam is not realistic (i.e. for narrow canyons), then the eigenperiod T_o of the dam is expected to be smaller leading to a stiffer overall dam response (see Gazetas 1987). Moreover, the 3D topography of the canyon is expected to amplify the seismic motion in a manner that cannot be captured by 2D seismic response analyses.

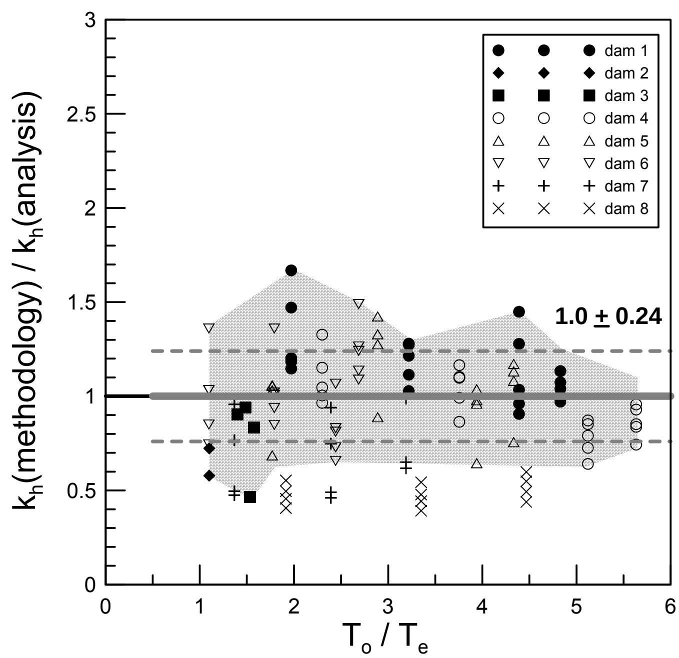

Fig. 9. Effect of the normalized eigenperiod T_o/T_e on the accuracy in the prediction of the maximum value of the seismic coefficient k_h according to the proposed methodology

CONCLUSIONS AND DISCUSSION

This paper presents an improved methodology for estimating seismic coefficients for the pseudo-static slope stability analyses of 2D earth dams and tall embankments. The methodology is based on a statistical analysis of numerical data from 28 2D seismic response analyses of pertinent geostructures. The methodology is applied in five (5) successive steps and estimates the maximum and effective seismic coefficients, k_h and k_{hE}, as functions of: a) the maximum acceleration of the free-field surface of the foundation soil PGA, b) the predominant period T_e of the seismic excitation, c) the first eigenperiod T_o of the dam, d) the dam foundation conditions, and e) the normalized depth z/H of the maximum depth z of the failure surface over the height H of the dam. The improved methodology offers satisfactory accuracy, with standard deviation of the relative error equal to ±24% as compared to numerical results.

In comparison with existing methodologies or design guidelines, the proposed methodology alleviates over-simplifications and increases the accuracy of estimation of the seismic coefficients. Nevertheless, there are still issues that need to be resolved in order to enhance the accuracy and broaden the limits of applications of the proposed methodology. For example, an exact estimation of the effect of the non-linear soil response on the estimation of the first eigenperiod T_o of the dam (Step 2) and of the soil foundation conditions on the value of the $a_{max,crest}$/PGA ratio (Step 3) are issues that still require investigation. Moreover, the effect of the reservoir on altering the value of k_h, pending on whether the failure surface is downstream or upstream, as well as other geometric characteristics of the failure surface on top of maximum depth z, are issues that still need to be resolved and would affect the accuracy of Step 4 of the methodology. Finally, the aforementioned correlation of the k_{hE}/k_h ratio to allowable displacements (Step 5) would provide a much more rational methodology of design, without loss of accuracy or safety. All foregoing issues are currently being investigated as part of research project funded by the Public Power Corporation of Greece.

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