Assessing Horizontal Seismic Coefficients in Earth Dams with Regards to Expected Deformation

Mohammad Hossein Tavakol
Tarbiat Modares University, Iran

Ali Komak Panah
Tarbiat Modares University, Iran

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ASSESSING HORIZONTAL SEISMIC COEFFICIENTS IN EARTH DAMS WITH REGARDS TO EXPECTED DEFORMATION

Mohammad Hossein Tavakol
MSc graduated of Tarbiat Modares University
Sano C.E., No. 11 Tavanir St., Tehran, Iran

Ali Komak Panah
Associate Professor, Tarbiat Modares University
Jalal-ale-Ahmad Ave., Tehran, Iran

ABSTRACT

One of the most important aspects that must be considered in design of embankment dams is to assess their stability during seismic events. When employing sophisticated approaches like finite element method for dynamic analysis of the mentioned structures against earthquake attacks, difficulties in evaluation of input parameters may exist. Hence employing simpler approaches like pseudo static method are still preferred by some designers.

However, there is a problem associated with the pseudo static procedure as there is no organized technique for precise evaluation of horizontal seismic coefficient. Moreover, this method applies "safety factor" for the stability evaluations which is not a consistent value when different methods of analysis are employed.

In this paper it is tried to suggest a method for a more organized evaluation of horizontal seismic coefficient in the range of empirical values used currently. Meanwhile “displacement” is applied as an appropriate parameter to evaluate the serviceability levels after an earthquake incident. In order to fulfill this goal, typical homogenous embankments were analyzed using shear beam approach to assess their horizontal seismic coefficients. Then Newmark sliding block model was employed to establish a relationship between assessed horizontal seismic coefficients and expected permanent displacements. Moreover, an equation was derived to assess yield acceleration on the basis of shear beam method principles.

LIST OF SYMBOLS

k_h	Horizontal seismic coefficient
$\overline{k}_h$	Average horizontal seismic coefficient
k_c	Yield acceleration in Newmark sliding block model
k_m	Expected peak acceleration to be developed at crest of an embankment to ground acceleration
g	Acceleration due to the gravity
ω_n	Undamped natural circular frequency of an embankment in the nth mode of vibration $\left(\omega_n = (\beta_n/H)\sqrt{\rho/G}\right)$
ρ	Unit mass of embankment material
G	Shear modulus of embankment material
H	Height of embankment
β_n	Zero value of frequency equation $J_0\left(\omega_n H\sqrt{\rho/G}\right) = 0 \rightarrow \beta_1 = 2.404, \beta_2 = 5.52, \ldots$
J_0	Bessel function of first kind
z	Failure depth in vertical direction
q	Failure depth to embankment height (q=z/H)
T_1	Fundamental period of vibration
D_n	Damping ratio of nth mode
$\ddot{u}_g$	Ground induced acceleration due to an excitation
$\ddot{u}_a(t)_{ave}$	Average absolute acceleration for any time instance
U	Horizontal permanent displacement
γ	Unit weight of embankment material
α	Embankment slope in degrees
τ	Shear strength (based on Mohr-Coulomb criterion)
φ	Angle of internal friction for embankment material
C	Cohesion of embankment material
σ	Normal stress
FS	Factor of safety

PREFACE

Every year many earthquakes attack manmade structures in earthquake prone areas and among them embankment dams are very important from safety point of view. They are so huge, costly, complicated in construction, and strategic that any probable damage to them may induce an economic and social deep impact.

Sophisticated methods such as the finite element or the finite difference approaches in dynamic analysis of embankments are expensive, time consuming, difficult in defining input characteristics, and very sensitive to type of constitutive models that are applied. On the other hand, methods such as the pseudo static approach for the design of small dams or even the preliminary design of large dams are simple and fast.

In spite of the simplicity and adequate precision of the pseudo static procedure, it has two prominent disadvantages. First, there is no organized and clear technique in determining the horizontal seismic coefficient and hence it is selected empirically. For instance, in some countries there are some regional maps and empirical relations used in evaluation of the seismic coefficient.

The second disadvantage is that application of the safety factor for the stability and serviceability evaluation of earthen structures (embankment dams and slopes) is not reliable. For serviceability evaluations of such structures following an earthquake incident, usually permanent displacement offers a better and much suitable criterion.

Of past researchers, Makdisi and Seed proposed a procedure (Makdisi, 1978) based on Newmark sliding block model (Newmark, 1965) which can be used to estimate probable permanent displacements following an earthquake attack. Based on that procedure, Seed (Seed, 1979) presented some values of probable displacements for potential sliding masses through embankments which do not lose more than %15 of their strength during an earthquake shaking (Tables 1 and 2).

Table 1. Probable upper bound displacements for embankment dams subjected to magnitude 6.5 earthquakes-little or no strength loss (Seed, 1979)

	Crest acceleration (x g)	k_m ≈	F.S= 1.15 for k_h=0.05 %15 strength loss $k_c = 0.05$	F.S=1.15 for k_h =0.1 %15 strength loss $k_c = 0.1$	F.S= 1.15 for k_h =0.1 %0 strength loss $k_c = 0.15$
Probable upper bound of accelerations for most earth dams	1.0	0.4	≈4 ft	≈1.8 ft	≈1 ft
	0.75	0.3	≈2.7 ft	≈1.2 ft	≈6 in
	0.50	0.2	≈1.7 ft	≈6 in	≈1 in
	0.25	0.1	≈6 in	0	0

The valuable procedure described by Makdisi and Seed, proposes displacement values for design peak accelerations developing at crest of an embankment. Meanwhile, the method which is discussed in this paper yields in displacement values for design seismic coefficients which are normally applied in the centre of sliding mass running through various heights of embankments.

Table 2. Probable upper bound displacements for embankment dams subjected to magnitude 8.25 earthquakes (little or no strength loss) (Seed, 1979)

	Crest acceleration (x g)	k_m ≈	F.S=1.15 for k_h =0.1 %15 strength loss $k_c = 0.1$	F.S= 1.15 for k_h =0.15 %15 strength loss $k_c = 0.15$	F.S=1.15 for k_h =0.15 %0 strength loss $k_c = 0.2$
Probable upper bound of acceleration for most earth dams	1.0	0.4	≈17 ft	≈7 ft	≈3 ft*
	0.75	0.3	≈10 ft	≈3 ft*	≈8 in*
	0.50	0.1	≈3 ft*	≈4 in*	0*
	0.25	0.1	0*	0*	0*

*Acceptable Performance

Here and based on studies by Tavakol (Tavakol, 2007), it is tried to employ displacement criterion when applying pseudo static method and a horizontal seismic coefficient. To do this some homogenous embankment dam models using shear beam approach were studied and then by applying Newmark sliding block model to them, a graphical relationship between seismic coefficients and permanent displacements was obtained. Moreover, yield acceleration which is used in the Newmark sliding block model was calculated on this basic assumption of the shear beam method that horizontal deformation in an embankment is only due to induced shear forces between adjacent horizontal layers.

The authors believe that by verification, modification, and development of the method described in this paper, it may be possible to have a rational assessment of the horizontal seismic coefficient based on probable earthquake characteristics, embankment geometrical specifications, and material properties.

SEISMIC COEFFICIENT EVALUATION

Generally, there are three approaches for evaluation of pseudo static coefficient (Subba Rao, 2003):

1. A rigid body response would require assuming a coefficient equal to maximum ground acceleration at

all heights. Embankment dams do not behave as rigid bodies. Neglect of viscous damping and of the short duration of force would lead to highly conservative designing while assuming constant seismic coefficients with height may make the top portion unsafe.
2. A more common practice has been to assume an empirical seismic coefficient based on prevalent practice. This is what is followed in the current Indian Standards. Such an assumption of uniform seismic coefficient, irrespective of materials and dimensions of a dam does not sound very rational.
3. Variable seismic coefficient approach based on spectral curves has now found greater acceptance as also adopted in the latest IS 1893(Part I): 2002.

Some empirical values (second approach) are presented in Table 3.

Table 3. Typical seismic coefficients and factors of safety used in practice

Seismic Coefficient	Suggested Factor of Safety	Remarks and Reference
0.15-0.25	>1.0	Japan (IITK, 2004)
0.05-0.15	>1.0	State of California (IITK, 2004)
0.15	>1.15	With less than 15% strength reduction and for 8.25 magnitude earthquake (Seed, 1979)
0.10	>1.15	With less than 15% strength reduction and for 6.5 magnitude earthquake (Seed, 1979)
$\frac{1}{3}-\frac{1}{2}PGA$	>1.0	(Marcuson, 1981)
$\frac{1}{2}PGA$	>1.0	With less than 20% reduction in strength (Hynes-Griffin, 1984)

The response of an earth dam to a strong earthquake is not only two dimensional including a vertical component of acceleration but also non-elastic and non-linear when true soil properties have to be considered … energy dissipation capacity of the soil masses is not totally viscous and under large deformations and local failures other modes of energy absorption will develop which will dissipate larger amounts of energy and hence decrease the overall response (Ambraseys, 1967).

Thus it seems that using the third approach is more rational and logical since more attention is paid to embankment and earthquake properties in the assessment of horizontal seismic coefficients.

For analytical solutions, Seed and Martin derived the following equations for calculating average seismic coefficients of any optional wedge through an embankment (such as AOB in Fig. 1) using shear beam approach: (Seed et al, 1966)

$$k_h = (1/g)\ddot{u}_g(t)_{ave} = \sum_{n=1}^{n=\infty}\left(\left(4GJ_1\left(\beta_n(z/H)\right)\right)/\left(g\rho H\omega_n z J_1(\beta_n)\right)V_n(t)\right) \quad (1)$$

Where

$$V_n(t) = \int_0^t\left(\ddot{u}_g e^{-D_n\omega_n(t-t')}\sin\left(\omega_n(t-t')\right)\right)dt' \quad (2)$$

Apparently k_h in the above formula is a function of time, properties of embankment materials, embankment height, ground motion characteristics, and fundamental period of vibration. Time-dependency makes use of so-calculated seismic coefficients impractical.

However, as an advantage they are not dependent on the shape of the wedge but on its vertical depth from the embankment's apex.

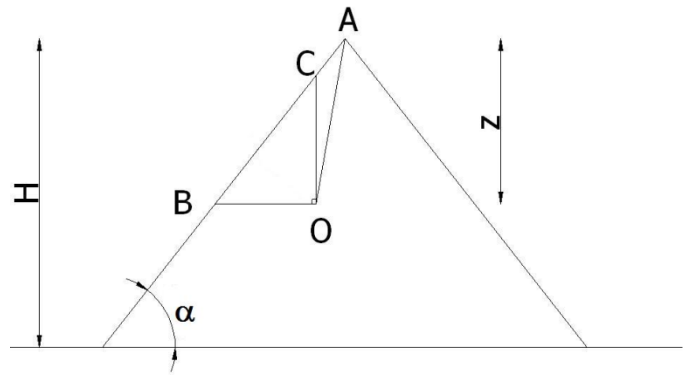

Fig. 1.AOB potential failure wedge in an embankment (for a unit length of the embankment in a right angle to the paper)

An example of k_h values produced by the Eq. (1) is demonstrated in Fig. 3 for the Y-direction component of Tabas earthquake (Fig. 2 shows the Y-Component of Tabas earthquake-Data for this figure is obtained from European Strong Motion Data Centre (Ambrasys et al, 2002)

As it was said above, because the average seismic coefficients are time dependent, there would be no fixed value to be implemented in the stability analysis of embankments. To overcome this problem, the following procedure is adopted to obtain a practical value.

Proposed Approach

First by applying a specific earthquake record (Fig. 2), material properties, and geometry of an embankment in the Eq. 1, the average k_h values of a specific "q" value (e.g. q=0.8) are calculated (Fig. 3).

Then to considering effects of negative and positive acceleration values simultaneously, the graph for absolute values of k_h is plotted (Fig. 4). This is made because of the fact that for any assumed embankment, there are upward and downward slopes but only one side is potentially in danger of failure. Moreover, using absolute values in a single procedure in comparison with two individual calculations for positive and negative values separately and considering the highest one finally, are almost the same.

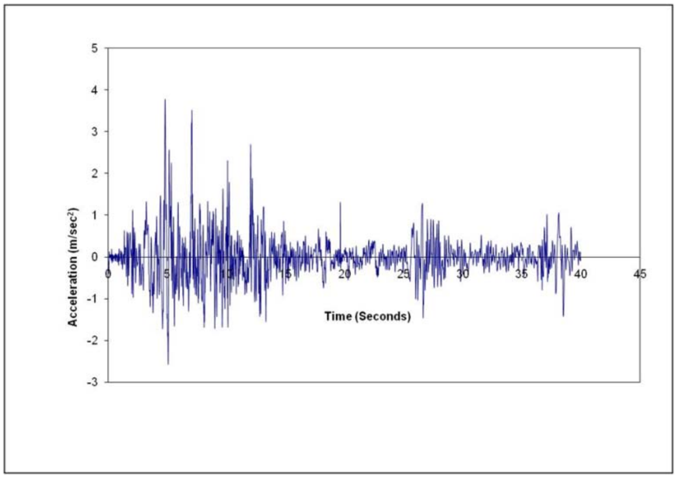

Fig. 2.Y-direction component of Tabas earthquake

In the next step and in order to remove paltry values and simplify the record, the maximum value of the absolute data is identified and all the values less than 0.05 times of the maximum k_h are omitted (Fig. 5). This "0.05" value is adopted from the definition of "Bracketed Duration" (Bolt, 1969).

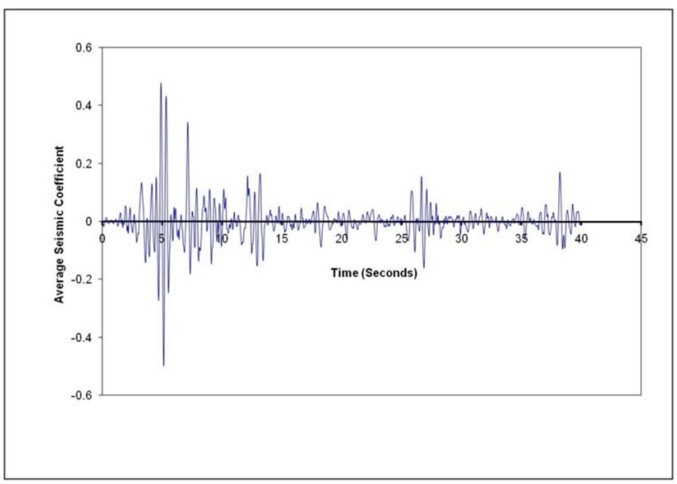

Fig. 3.k_h average values through the record length for the Y-direction component of Tabas earthquake using Seed and Martin equation (Seed, 1966). This figure is calculated for q=0.8, H=20m, shear wave velocity=129 m/sec, T_1=0.40 sec, D=%20 (Tavakol, 2007)

Later, the area beneath diagram (Fig. 5) is calculated for the remaining time steps. This area is then divided by the corresponding time lengths (of remaining time steps) which yields the average fixed k_h value:

$$\overline{k}_h = \sum\left(k_{h_i} \times \Delta t_i\right)/\sum \Delta t_i \quad \left(\forall i: k_{h_i} \neq o\right) \tag{3}$$

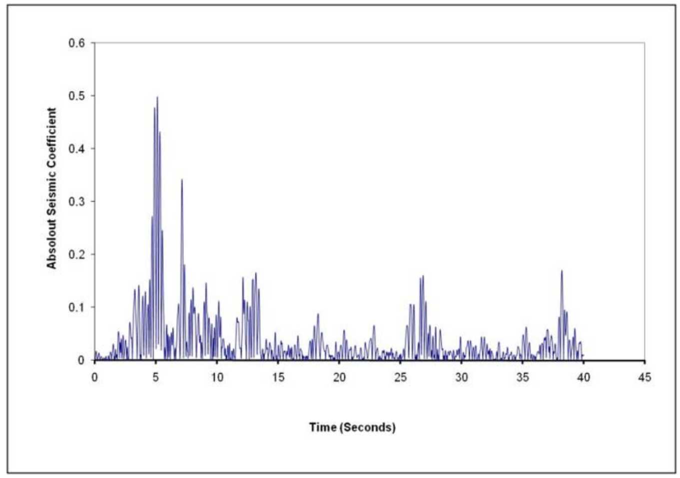

Fig. 4.Absolute values of Fig. 3(Tavakol, 2007)

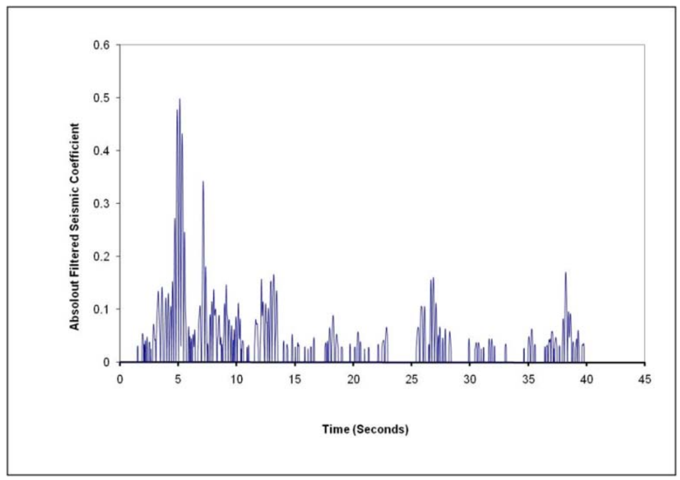

Fig. 5.Filtered values of Fig. 4 (all the values less than 0.05 times of the maximum k_h are omitted) (Tavakol, 2007)

By following the above described procedure for few values of embankment heights and material properties (different T_1 values), it is possible to produce a spectral graph representing k_h versus T_1 for any specific earthquake. In Fig. 6, an example of such graphs for the Y-direction component of Tabas earthquake is presented while considering %15 of critical damping for the embankment material.

The obtained values are within a range of empirical values used currently (for some of empirical values please refer to Tab. 3), and furthermore are calculated based on an analytical and organized procedure.

In order to obtain a k_h from such graphs, first T_1 should be calculated using Eq. 4 and then based on the “q” value, the corresponding k_h value is assessed:

$$T_1 = (2\pi/\omega_1) \ where \ \omega_1 = (\beta_1/H)\sqrt{G/\rho} \quad (\beta_1 = 2.404) \tag{4}$$

Of pivotal characteristics of the calculated k_h by the discussed procedure, these might be mentioned:

- procedure of calculation is organized,
- geometrical configuration and material properties of embankment are considered, and
- attention is paid to motion properties.

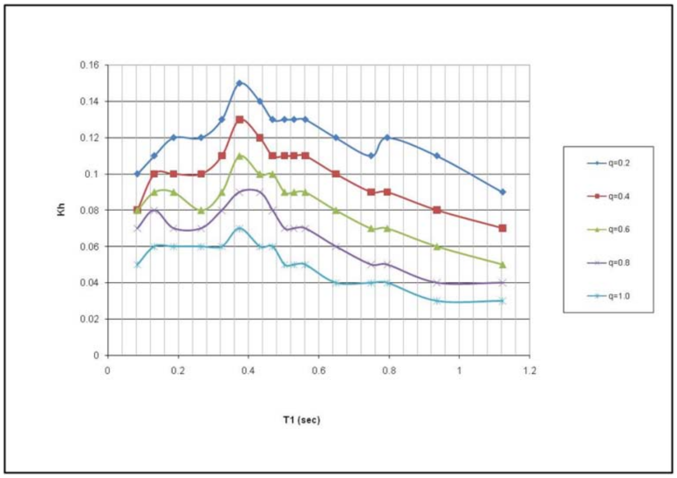

Fig. 6.k_h versus T_1 plot for Y-direction component of Tabas with % 15 of critical damping for the embankment's material (Tavakol, 2007)

DISPLACEMENT CRITERION

As it was said earlier, the main objective of this research is to establish a relationship between horizontal seismic coefficients and expected permanent displacements. Hence, in the following section the formulation for the threshold acceleration (k_c) and calculation of probable permanent displacements are presented.

It is worth noting that in the Newmark sliding block model used to calculate probable permanent displacements, the upward movement of the wedge was neglected to be on the side of safety. Moreover, the reducing acceleration in the movement of sliding wedge was assumed to be equal to k_c. For detailed information on the Newmark sliding block model applied, please refer to Kramer (Kramer, 1996).

Evaluation of Yield Acceleration (k_c)

In order to calculate permanent displacements based on the Newmark sliding block model, yield (threshold) acceleration must be evaluated. As the method adopted for the calculation of horizontal seismic coefficient was the shear beam method, it was also used to derive the related equation for the yield acceleration. The detailed steps of the derivation of threshold acceleration formula (Eq. 5) are discussed in Appendix A.

$$k_c = (2.C/(q.H.\gamma)) + (OB.\tan\alpha.\tan\varphi/(q.H)) \tag{5}$$

Permanent Displacement Calculations

In order to implement permanent displacement as the desirable criterion for evaluation of horizontal seismic coefficient, the following process is followed. Having calculated k_h and k_c values for wedges of specific "q" values, the corresponding permanent displacements for each wedge based on the Newmark sliding block model are calculated under the attack of the very same earthquake record.

Now a graphical relationship between k_c/k_h values and probable permanent displacements is possible. A typical example of such graphical relationships is presented in Fig. 7.

For any earthquake, a graph is possible to be made by analyzing several embankments and evaluating their horizontal seismic coefficients, threshold acceleration, and expected permanent displacements for potential sliding wedges.

After providing such graphs for any earthquake, the following guidelines may be used to find suitable horizontal seismic coefficients and corresponding expected permanent displacements for potential sliding wedges:

1. Values for "H", "G", and "ρ" are determined.
2. Using the values of previous step and Eq. 4 the value for T_1 is obtained.
3. For desirable value of "z", "q" equal to z/H is calculated.
4. From the k_h versus T_1 graph of a certain earthquake and for specific "q" and damping values, k_h is determined (an example of such graphs is shown in Fig. 6)
5. In order to maintain static stability and based on known values of "φ", and "C", slope's gradient (α) is calculated with a reasonable factor of safety. Now the k_c value for any optional sliding wedge is calculated using the following equation:

$$k_c = (2.C/(q.H.\gamma)) + (OB.\tan\alpha.\tan\varphi/(q.H)) \tag{5-Repeated}$$

6. Applying the calculated values obtained in steps 4 and 5, k_c/k_h is obtained.
7. Now using k_c/k_h vs. $U/(T_1.g.k_h)$ graph for a specific earthquake, an estimation of expected permanent displacements is possible (an example of such graphs is shown in Fig. 7).
8. Obtained displacements in step 7 can be compared to allowable limits.

The following section contains an example on how to apply above steps.

An Illustrative Example

On September 16th of 1978 a devastating quake hit "Tabas" northeast of Iran, leaving more than 20 000 lives dead. This earthquake with a magnitude 7.4 is known to be the greatest one shaking Iran in recent decades. Time history for Y-direction component of Tabas earthquake is shown in Fig. 2.

Based on the approach discussed earlier, a spectral graph for k_h versus T_1 for this quake is prepared and shown in Fig. 6. Meanwhile Fig. 7 shows the k_c/k_h against $U/(T_1.g.k_h)$.

Now suppose we have a typical embankment with the following specifications (Fig. 1):

G=64 000 KPa, D=%15, $\gamma = 19.65\ KN/m^3$, and H=30 m

Also for providing static stability, let us take the following properties which lead to a static safety factor equal to 1.38:

C=20 KPa, $\varphi = 30°$, and $\alpha = 33.7°$ (1V:1.5H)

From Eq. 4:

$$\omega_1 = (\beta_1/H)\sqrt{G/\rho} =$$
$$(2.404/30)\sqrt{64000/(19.65/9.806)} = 14.32\ rad/sec$$

So the first period is:

$$T_1 = 2\pi/\omega_1 = 2\pi/14.32 = 0.438\ sec$$

From Fig. 6 and for $T_1 = 0.438$ the values for k_h are as follow:

For q=0.2 : $k_h = 0.14$; For q=0.4 : $k_h = 0.12$
For q=0.6 : $k_h = 0.10$; For q=0.8 : $k_h = 0.08$
For q=1.0 : $k_h = 0.06$

Also From Eq. 5 for q=1 and OB=2.5 m:

$$k_c = (2C./(q.H.\gamma)) + (OB.\tan\alpha.\tan\varphi/(q.H)) =$$
$$((2 \times 20)/(1 \times 30 \times 19.65)) +$$
$$((2.5 \times \tan 33.7 \times \tan 30)/(1 \times 30))=0.100$$

Now, applying this value into Fig.7 leads to a probable permanent displacement equal to 17.2 cm (roughly 20 cm). Consequently, by applying other “q” and “OB” values in the above formula and using Fig.7, more values of probable permanent displacements are achieved (Tab. 4).

From the tabulated data, it is evident that a potential wedge with q=0.8 has the greatest value (U=30.2 cm) among wedges with OB=2.5 m (considerable mass of failure). This might be used when evaluating serviceability levels of the embankment following the earthquake occurrence.

Furthermore a k_h = 0.08 is an appropriate value to be considered if pseudo static method is applicable.

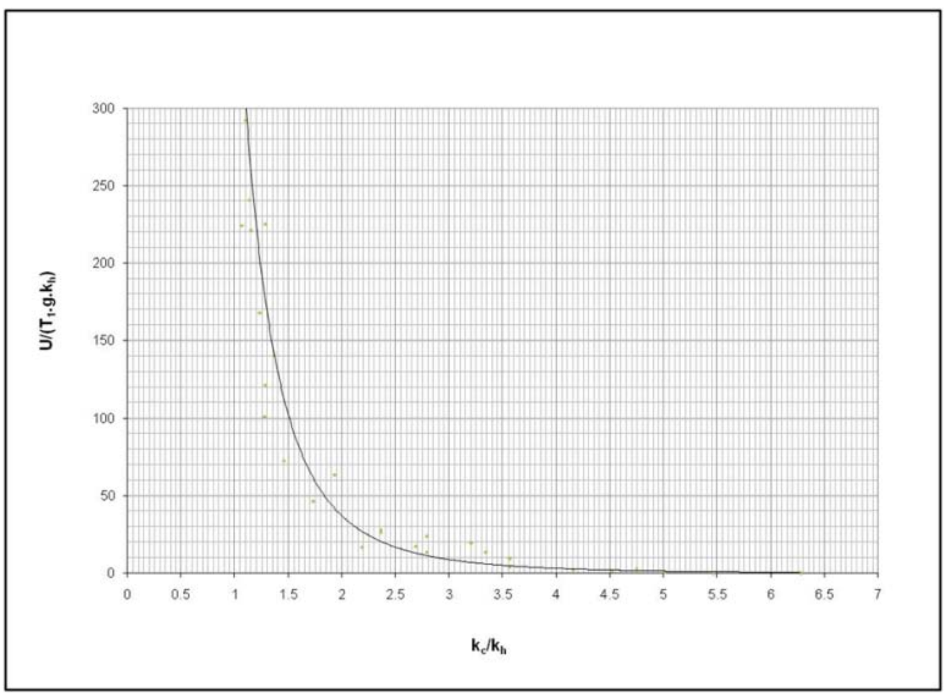

Fig. 7.Graphical relationship between k_h, k_c, T_1, and expected permanent displacements. The graph is produced for Y-direction component of Tabas earthquake and % 15 of critical damping for the embankment’s material (Tavakol, 2007)

Table 4: Probable permanent displacements values for the illustrative example

q	0.2		0.4		0.6		0.8		1.0	
OB[a] (m)	1.5	2.5	1.5	2.5	1.5	2.5	1.5	2.5	1.5	2.5
k_c	0.435	0.500	0.218	0.250	0.145	0.167	0.109	0.125	0.087	0.100
k_h	0.14		0.12		0.10		0.08		0.06	
k_c/k_h	3.10	3.57	1.81	2.08	1.45	1.67	1.36	1.56	1.45	1.67
$\frac{U}{T_1 g k_h}$	8	5	53	42	112	70	140	88	112	67
Probable Permanent Displacement (cm)	4.8	3	27.3	21.6	48.1	30.1	48.1	30.2	28.8	17.2

[a] "OB" values were considered so to represent considerable potential sliding masses

CONCLUSIONS

The proposed procedure described in this paper in assessing horizontal seismic coefficient in embankment dams, yields the results that are not only in good accordance with empirical values but also calculated based on an organized procedure. In this method, a graphical relation between k_h and T_1 was produced while attention was paid to all of the following prominent factors in the seismic responses of embankments:

- Properties of embankments material
- Geometrical configuration of embankments
- Motion characteristics and ingrained properties of design earthquakes

Furthermore, using a simple definition based on one of the basic assumptions of shear beam approach in which it is assumed that there is only shear resistance between horizontal layers of an embankment cross section, an equation for the yield acceleration was developed. Then by using values obtained from this equation and applying the Newmark sliding block model, probable permanent displacements for few potential sliding wedges following a specific earthquake attack were calculated.

Then, by presenting threshold acceleration to horizontal seismic coefficient ratios against a function of probable permanent displacements in a graphical form, a design graph was achieved. Using this graph and $k_h$versus T_1 graph, it is possible to estimate the probable permanent displacements that will occur under the considered earthquake. However it seems that defining a band instead of a single line for the displacement graph (Fig. 7) is more suitable.

Finally it should be noticed that the earthquake records used in this study were chosen to be outcrop records. Studying how to cope with different geologically originated records is extremely essential. Probably, changes may be required in removing trifling parts of an earthquake record in softer sites.

APPENDIX A

Based on one of the basic assumptions of the shear beam approach, any deformation in an embankment occurs horizontally and only if induced shear forces exceed shear resistance between horizontal layers of an embankment section. As a result, if the AOB wedge in Fig. 1 starts sliding, the only resisting force would be along OB line. Hence we have:

$$FS_{Sliding} = F_{Resisting\ Forces}/F_{Driving\ Forces} = (\tau.OB/(W_{AOB}.k_h)) \quad \text{(A-1)}$$

The weight of AOB wedge is

$$W_{AOB} = (z.OB/2)\gamma = (q.H.OB.\gamma/2) \quad \text{(A-2)}$$

In addition, the shear resistance is assumed to follow Mohr-Coulomb criterion:

$$\tau = C + \sigma \tan\varphi \quad \text{(A-3)}$$

So

$$FS_{Sliding} = (C + \sigma \tan\varphi).OB/\big((q.H.OB.\gamma/2)k_h\big) = \big(2(C + \sigma\tan\varphi)\big)/(q.H.\gamma.k_h) \quad \text{(A-4)}$$

Also the normal stress is calculated from the weight of OBC wedge:

$$\sigma = W_{OBC}/OB = (OC.OB.\gamma/2)/OB = OC.\gamma/2 = OB.\tan\alpha.\gamma/2 \quad \text{(A-5)}$$

The latter yields the following equation for the factor of safety:

$$FS_{Sliding} = \big(2(C + OB. \tan\alpha . \gamma . \tan\varphi/2)\big)/(q.H.\gamma.k_h) \quad \text{(A-6)}$$

The wedge starts sliding when $k_h = k_c$. For this case and based on limit equilibrium method, the safety factor would be equal to the unity:

$$k_h = k_c \Rightarrow FS_{Sliding} = 1.0 \quad \text{(A-7)}$$

Thus from Eq. 10 we have

$$1 = 2(C + OB. \tan\alpha . \tan\varphi . \gamma/2)/(q.H.\gamma.k_c) \quad \text{(A-8)}$$

And finally

$$k_c = (2C/(q.H.\gamma)) + (OB. \tan\alpha . \tan\varphi/(q.H)) \quad \text{(A-9)}$$

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