AN INVESTIGATION INTO THE PSEUDOSTATIC ANALYSES OF THE KITAYAMA DAM USING FE SIMULATION AND OBSERVED EARTHQUAKE-INDUCED DEFORMATIONS

T. Akhlaghi[1] and M. Neishapouri[2]

[1] *Assisstant Professor, Dept. of Geotechnical Eng. ,Faculty of Civil Eng.,University of Tabriz, Tabriz, Iran*
[2] *Senior Engineer, Tabriz Urban Railway Org.(TURO),29 Bahman Blvd., Tabriz, Iran*
Email: takhlaghi@tabrizu.ac.ir

ABSTRACT:

Evaluation of seismic slope stability is one of the most important activities in the design of earth structures. A number of analytical techniques, based on limit equilibrium and stress–deformation analyses, are available and proposed for conducting the slope instability analysis. Among them is the pseudostatic analyses which represent the effects of an earthquake by applying static horizontal and/or vertical accelerations to a potentially unstable mass of soil. Representation of the complex, transient and dynamic effects of earthquake shaking by a single constant acceleration is clearly crude and has significant shortcomings. The accuracy of the pseudostatic approach is governed by the accuracy with which the simple pseudostatic inertial forces represent the complex dynamic inertial forces that actually exist in an earthquake. In this study, the Kitayama Dam, which has been designed using the pseudostatic approach and damaged during the 1995 Kobe Earthquake, was investigated and analyzed. The finite element model of the dam was prepared based on the detailed available data and results of in-situ and laboratory material tests. Dynamic analyses were conducted to simulate the earthquake-induced deformations of the Kitayama dam using the computer program Plaxis code. Then the pseudostatic seismic coefficient used in the design and analyses of the dam was compared with the seismic coefficients obtained from dynamic analyses of the simulated model as well as the other available proposed pseudostatic correlations. Based on the comparisons made, the accuracy and reliability of the pseudostatic seismic coefficients are evaluated and discussed.

KEYWORDS: Pseudostatic Analyses, FE Simulation, Kitayama Dam

1. INTRODUCTION

The seismic stability of earth structures has been analyzed by pseudostatic procedures for many decades in which the effects of an earthquake are represented by constant horizontal and/or vertical accelerations. Stability is expressed in terms of a pseudostatic factor of safety calculated by limit equilibrium procedures. Limit equilibrium analyses consider force and/or moment equilibrium of a mass of soil above a potential failure surface.

The results of pseudostatic analyses are critically dependent on the value of the seismic coefficient. Selection of an appropriate pseudostatic coefficient (particularly k_h) is the most important, and the most difficult, aspect of a pseudostatic analysis. The seismic coefficient controls the pseudostatic force on the failure mass, so its value should be related to some measure of the amplitude of the inertial force induced in the potentially unstable material. If the slope material was rigid, the inertial force induced on a potential slide would be equal to the product of the actual horizontal acceleration and the mass of the unstable material. This inertial force would reach its maximum value when the horizontal acceleration reached its maximum value. In recognition of the fact that actual slopes are not rigid and that the peak acceleration exists for only a very short time, the pseudostatic coefficients used in practice generally correspond to acceleration values well below the maximum value. Terzaghi (1950) originally suggested the use of k_h=0.1 for sever earthquakes (Rossi-Forel IX), k_h=0.2 for violent and destructive earthquakes (Rossi-Forel X), and k_h=0.5 for catastrophic earthquakes. Seed (1979) listed pseudostatic design criteria for 14 dams in 10 seismically active countries; 12 required minimum factors of safety of 1.0 to 1.5 with pseudostatic coefficients of 0.10 to 0.12. Marcuson (1981) suggested that appropriate pseudostatic coefficients for dams should correspond to one-third to one-half of the maximum acceleration, including amplification or deamplification effects, to which the dam is subjected. Using shear beams models, Seed and Martin (1966) and Dakoulas and Gazetas (1986) showed that the inertial force on a potentially unstable slope in an earth dam depends on the response of the dam and that the average seismic coefficient for a deep failure surface is substantially smaller than that of a failure surface that does not extend far below the crest. Seed (1979) also indicated that deformations of earth dams constructed of ductile soils with crest accelerations less than 0.75*g* would be acceptably small for pseudostatic factors of safety of at least 1.15 with k_h=0.10 (*M*=6.5) to k_h=0.15 (*M*=8.25). This criteria would allow the use of pseudostatic accelerations as small as 13 to 20 percent of the peak crest acceleration. Hynes-Griffin and Franklin (1984) applied the Newmark sliding block analysis to over 350 accelerograms and concluded that earth dams with pseudostatic factors of safety greater than 1.0 using $k_h = 0.5a_{\max} / g$ would not develop dangerously large deformations.

As can be seen from above discussions, there are no hard and fast rules for selection of a pseudostatic coefficient for design. However, it seems that the pseudostatic coefficient should be based on the actual anticipated level of acceleration in the failure mass and that it should correspond to some fraction of the anticipated peak acceleration. Although engineering judgment is required for all cases.

Representation of the complex, transient, dynamic effects of earthquake shaking by a single constant unidirectional pseudostatic acceleration is obviously quite crude. Detailed analyses of historical and recent earthquake-induced landslides have illustrated significant shortcomings of the pseudostatic approach. Results of pseudostatic analyses of some earth dams (e. g., Upper San Fernando dam, Lower San Fernando dam, Sheffield dam, and Tailing dam) show that pseudostatic analyses produced factor of safety well above 1.0 for a number of dams that later failed during earthquakes. These cases illustrate the inability of the pseudostatic method to reliably evaluate the stability of slopes susceptible to weakening instability. Nevertheless, the pseudostatic approach can provide at least a crude index of relative, if not absolute, stability.

Despite the above-mentioned limitations, the pseudostatic approach has a number of attractive features. The analysis is relatively simple and straightforward. Indeed, its similarity to the static limit equilibrium analyses routinely conducted by geotechnical engineers makes its computations easy to understand and perform. It produces a scalar index of stability (the factor of safety) that is analogous to that produced by static stability analyses. It must always be recognized, however, that the accuracy of the pseudostatic approach is governed by the accuracy with which the simple pseudostatic inertial forces represent the complex dynamic inertial forces that actually exist in an earthquake. Difficulty in the assignment of appropriate pseudostatic coefficients and in interpretation of pseudostatic factors of safety, coupled with the development of more realistic methods of analysis, have reduced the use of the pseudostatic approach for seismic slope stability analyses. Methods based on evaluation of permanent slope deformation are being used increasingly for seismic slope stability analysis.

2. KITAYAMA DAM PERFORMANCE AND PROPERTIES

The Kitayama Dam is located on the Rokko granite zone and about 1.5 km from the Ashiya Fault and the Koyo fault. The dam is an earth dam with a height of 25 meters and was completed in 1968.
The Kobe Earthquake of January 17, 1995 in Japan caused slides on the upstream slope of the Kitayama Dam which is an earth dam located about 33 kilometers north-east of the epicenter. It was minor damage that did not affect the structural safety and the water storage functions of the dam. But this was the first time that an embankment dam designed based on design standards and filled by the engineered rolling compaction was damaged in this way in Japan. The sliding failure zone with a depth of 1.5 to 2 m was confirmed and the length in the dam axis direction of the sliding was about 100 m (figure 1).

Figure 1 The sliding failure zone in Kitayama dam caused by Kobe 1995 earthquake

The Kobe University motion is the earthquake motion observed during the Kobe Earthquake. The observation station is located on a weathered rock with an S-wave velocity of 340 m/s and 24 km far from the epicenter. The Kobe University motion has a peak acceleration of 270.4 gal and a duration of 20 seconds. During the earthquake, the reservoir water level was at the top of the sliding failure block. The level differences on the top of the sliding block were 1 to 1.5 m.

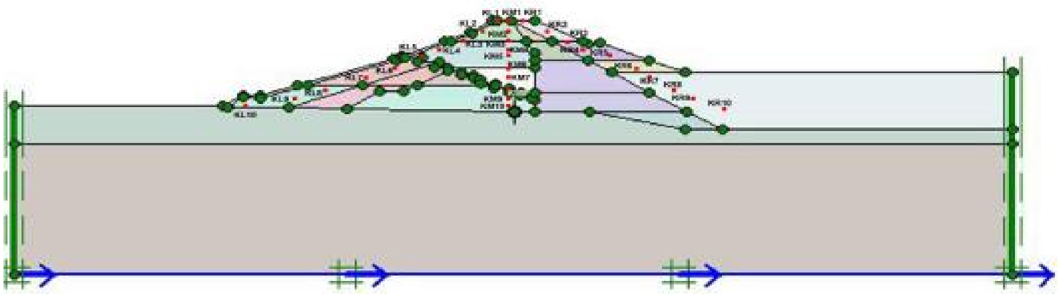

Figure 2 Cross section, layers and stress points of Kitayama dam

As shown in figure 2, the dam is divided into 24 different layers each representing different soil material zone. These soil units of dam section have been classified using Eqn. 2.1 suggested by Sawada. The equation indicates the effect of height variation on the shear wave velocity and shear modulus of the layers materials located at different levels of the dam section.

$$V_S = 140Z^{0.34} \tag{2.1}$$

where V_S is the shear wave velocity and Z is the dam height below the crest.
The layers' soil parameters associated with the 24 soil zones obtained using the above equations and detailed results given by Sakamoto et al. (1973) are listed in table 2.1.

Table 2.1 Soil properties of the Kitayama dam

Layer No.	g_{dry} $[kN/m^3]$	g_{wet} $[kN/m^3]$	E $[MN/m^2]$	V_S $[m/s]$	c $[kN/m^2]$	j $[Degree]$	n	K $[m/day]$
1	19.5	20.60	8.519 E+5	405.8	2	86	0.3	0.06
2	19.5	20.60	8.390 E+5	401.3	2	48	0.309	0.06
3	19.5	20.60	6.238 E+5	347.2	2	85	0.3	0.06
4	19.5	20.60	6.689 E+5	359.6	2	48	0.3	0.06
5	19.5	20.60	6.881 E+5	364.7	2	46	0.3	0.06
6	19.5	20.60	8.562 E+5	406.8	2	46	0.3	0.06
7	19.5	20.60	7.884 E+5	386	2	44.3	0.3	0.09
8	19.5	20.60	5.079 E+5	313.3	2	44.6	0.3	0.09
9	19.5	20.60	2.048 E+5	215.7	2	44.3	0.3	0.09
10	27.5	29.1	3.655 E+5	215.7	63.7	-	0.3	86.4
11	27.5	29.1	6.472 E+5	297.8	63.7	-	0.3	86.4
12	27.5	29.1	8.792 E+5	347.1	22.5	39	0.3	82
13	27.5	29.1	1.201 E+5	408.7	22.5	39	0.3	82
14	19.3	20.4	2.385 E+5	216.8	63.7	-	0.3	0.06
15	19.2	20.5	2.472 E+5	210	63.7	-	0.33	0.06
16	19.3	20.4	4.186 E+5	280.6	63.7	-	0.35	0.06
17	19.2	20.5	4.119 E+5	273.1	63.7	-	0.36	0.06
18	19.3	20.4	7.105 E+5	366.9	63.7	-	0.34	0.06
19	19.2	20.5	6.944 E+5	357.1	63.7	-	0.36	0.06
20	19.3	20.4	9.007 E+5	408.6	63.7	-	0.37	0.06
21	19.3	20.4	1.023 E+6	437.1	63.7	-	0.36	0.06
22	19.2	20.5	9.404 E+5	420.1	63.7	-	0.35	0.06
23	21.7	23.64	1.61 E+6	521.6	2	30	0.34	0.06
24	23.1	24.53	9.36 E+6	1222	2	28	0.33	0.1

3. STUDY METHODOLOGY AND DYNAMIC ANALYSIS

Most engineers consider the seismic coefficient as a means of designating the magnitude of a static force which is equivalent in effects (i.e. produces the same deformations of the earth dam) to the actual dynamic inertial forces induced by the earthquake. But how would the seismic coefficient denoting this equivalent static force be determined? It would seem that the determination of an appropriate value would necessarily involve two steps:
1) Determination of deformations and degree of instability of dam induced by the earthquake, and
2) Evaluation of equivalent static force with the capability to make the same displacements or instabilities.
It would appear that any attempt to select a final value of such a seismic coefficient without going through step one and without a large back-log of experience to guide the selection could have little reliable basis.

In order to determine exact results for stage one, it will be preferable to utilize dynamic analyses based on finite element method, and hence the Plaxis software seems to be an appropriate choice. High accuracy of dynamic analysis puts it at high point of view. The results obtained from two-dimensional dynamic analysis of Kitayama dam under Kobe 1995 earthquake, such as horizontal and vertical displacements, almost justify the observed displacements. Then an equivalent static force is determined for each layer and seismic coefficient is obtained for those layers. The importance of this study shines in evaluating varying seismic coefficient for Kitayama dam and that is relevant to differentiation of each layer's coefficient. Assuming a constant seismic coefficient would be applicable for rigid structures and using this current method for earth dams which have not rigid-body response is not rational. In this study, the equivalent seismic coefficients for different soil zones of Kitayama dam have been determined using two-dimensional dynamic analyses and large back-log, and then the results have been compared with the design seismic coefficient (0.15) of the dam.

Most of the problems encountered in the area of geotechnical engineering such as retaining walls, tunnels, earth dams and embankments are studied using two-dimensional dynamic analyses based on the finite element method (FEM) which is one of the available powerful numerical methods. One of the current methods used to solve the movement equation is Newmark step-by-step method. Newmark provided this method for dynamic analysis of earthquake loading. In this method displacement and velocity are determined using the following equations:

$$u_{t+\Delta t} = u_t + \dot{u}_t \, \Delta t + [(\frac{1}{2} - \alpha)\ddot{u} + \alpha \, \ddot{u}_{t+\Delta t}]\Delta t^2 \tag{3.1}$$

$$\dot{u}_{t+\Delta t} = \dot{u}_t + [(1-\beta)\ddot{u}_t + \beta \, \ddot{u}_{t+\Delta t}]\Delta t \tag{3.2}$$

where Δt is time pace and α and β are controlling parameters for numerical integration accuracy, according to the implicit Newmark scheme. In order to obtain a stable solution, these parameters have to satisfy the following condition:

$$\beta \geq 0.5 \quad ; \quad \alpha \geq 0.25(0.5+\beta)^2 \tag{3.3}$$

Considering standard values (β=0.5) which is called average acceleration method, leads the calculations to rational results. Despite Newmark's damping method, taking advantage of β=0.6 and α=0.3025 values, in this study average acceleration method is being used to solve movement equations, as well as Newmark's method.

Special boundary conditions have to be defined in order to avoid the spurious reflections of the waves on the model boundaries. These boundaries are based on the Lysmer-Kohlmeyer model. According to this model, the normal and shear stress components absorbed by a damper are determined as follows:

$$\sigma_n = -c_1 \rho V_P \dot{u}_x \tag{3.4}$$

$$\tau = -c_2 \rho V_S \dot{u}_y \tag{3.5}$$

where ρ = mass density, V_S = shear wave velocity, V_P = longitudinal wave velocity, $\dot{u}_x$ and $\dot{u}_y$ = velocity of particle motion in the direction of x and y, respectively. c_1 and c_2 are relaxation coefficients used to improve the wave absorption on the absorbent boundaries. c_1 Corrects the dissipation in the direction normal to the boundary and c_2 in the tangential direction. The research and experience findings recommend to choose c_1=1 and c_2 = 0.25 for best results (Brinkgreve and Vermeer, 1998).

The simulation begins with specifying the clusters (24 clusters) and defining the properties relevant to each cluster along with definition of upstream water level and phreatic line.
The Numerical calculations using Plaxis software for this study involve 3 Phases. First phase is dam plastic analysis conducted for the time when the construction is over. Second phase includes dam plastic analyses under own body load and finally the last phase consists of dynamic analysis under Kobe 1995 earthquake loading. The third step loading is applied in the form of a file (accelerogram) input to the program. The whole deformations, horizontal and vertical displacements are obtained in the output of the program.

4. DETERMINATION OF EQUIVALENT FORCE

Before starting calculation step, stress points are chosen on cross section of the dam. These points are located in the *x* direction with three points at each level as shown in figure 2 (one point upstream side, one point middle part, and one point downstream side). 30 points constituting 10 lines parallel with the *x* direction are specified. Stress (s_{xx})-time curves (30 curves) can be obtained from CURVE step of the program. Owing to the generation of numerous curves and for the sake of space saving, only some stress-time curves related to points at various levels of the section are provided. Figures 3 to 8 show these envisaged curves.


Time(s)

Figure 3 Stress-time curve for point KM2

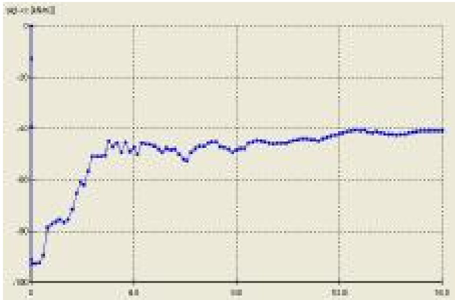
Time(s)

Figure 4 Stress-time curve for point KM6


Time(s)

Figure 5 Stress-time curve for point KR2


Time(s)

Figure 6 Stress-time curve for point KR4


Time(s)

Figure 7 Stress-time curve for point KL7


Time(s)

Figure 8 Stress-time curve for point KL9

For each curve a maximum value is deliberated for a period and is considered due to its conservative value. Though the maximum value of each curve is multiplied by 0.7 (Bathurst & Cai, 1995) then distribution of stress along the height of model is approximated to be linear for all points located in the upstream (U/S), downstream (D/S) and middle (M) parts. The results are shown in figures 9 to 11.

In addition, the equivalent force for each part of layer across stress point is determined using the product of height of layer to approximated stress value. The results are illustrated in figures 12 to 14.

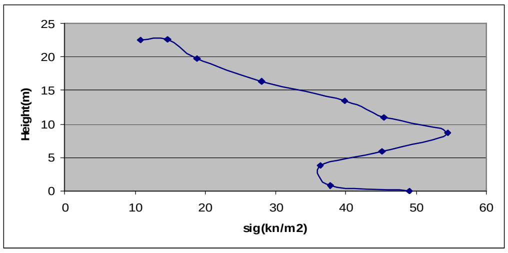

Figure 9 Approximated stress curve for U/S part of dam

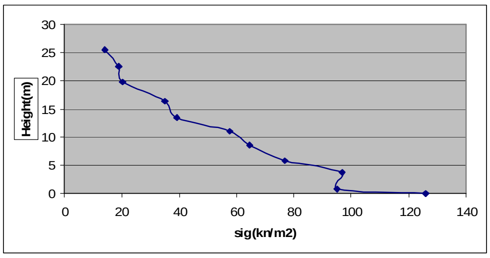

Figure 10 Approximated stress curve for M part of dam

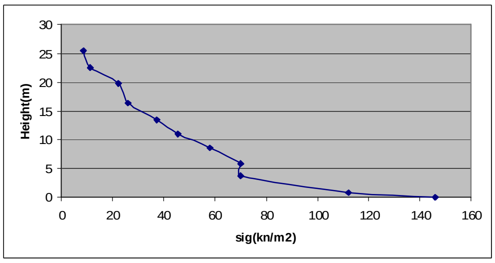

Figure 11 Approximated stress curve for D/S part of dam

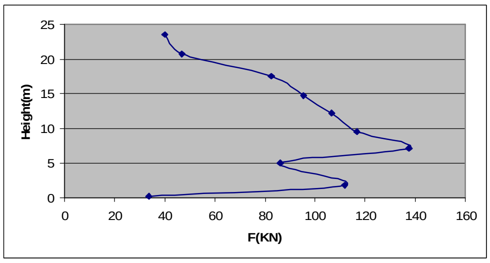

Figure 12 Equivalent force curve for U/S part of dam

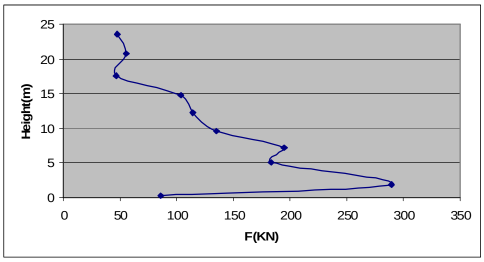

Figure 13 Equivalent force curve for M part of dam

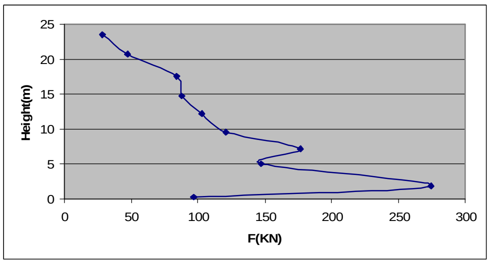

Figure 14 Equivalent force curve for D/S part of dam

Then each layer's equivalent force can be determined by employing the following equation:

$$F = \frac{\sum_{i=1}^{3} F_i L_i}{L} \tag{4.1}$$

in which F_i is force of the part (upstream, middle, downstream), L_i is the effective length of the part, and L is the layer's length. Finally seismic coefficient is calculated by dividing layer force to its weight. The results are summarized and listed in Table 4.1

Table 4.1 Seismic coefficients for Kitayama dam under 1995 Kobe loading

Layer	*Force* $\overline{F}$ *(kN)*	*Height of layer (m)*	*Weight W (kN)*	*Seismic coefficient* $C = \overline{F}/W$
1	386	2.9	891.3	0.47
2	497	2.8	1420	0.35
3	877	3.5	3043.5	0.29
4	957	2.8	3385.2	0.27
5	1070	2.5	3742.5	0.27
6	1252	2.33	4891.7	0.25
7	1716	2.76	5765.6	0.25
8	1419	2.11	5234.8	0.26
9	2163	3.62	7846.6	0.23
10	681	0.78	3242.8	0.21

5. CONCLUSIONS

The results obtained from the analyses conducted for investigating Kitayama dam behavior under Kobe earthquake loading indicate that the seismic coefficient increases with increasing of the height. The ratio of seismic coefficient at the crest over seismic coefficient at the base of the dam is about 2. The design seismic coefficient was 0.15 but the minimum calculated seismic coefficient for lower layer of this dam is 0.21. In this case the crest of the dam settled 0.43 m and moved 0.72 m downstream. The maximum amount of horizontal displacements was about 0.98 m. The results indicate that the constant seismic coefficient used in designing of Kitayama dam was not applicable and in case of using constant seismic coefficient, it must be between the minimum value of 0.21 and the maximum value of 0.47. The following results are gained by comparing the design seismic coefficient to the calculated seismic coefficients:

- The maximum value of seismic coefficient offered by JCOLD is 0.25 while this coefficient is not applicable in the case of this dam.
- Although the minimum acceleration submitted by seismograph has a value about 0.221g but considering the rigid body response method (k=0.221) in this case which has a maximum seismic coefficient of 0.47 is not reliable.
- The maximum and minimum values determined using Ambraseys's method are 0.27, 023 respectively. It seems that the rational value for seismic coefficient is existed between these evaluated values.
- This study shows that considering inconstant seismic coefficient in earth dam design is more realistic and rational than considering a constant seismic coefficient. Furthermore the procedure employed in this study can be utilized for evaluation of design seismic coefficient of constructed earth dams designed using pseudostatic analyses.

REFERENCES

Ambraseys, N.M. (1974). Engineering seismology and earthquake engineering. *Misc.*, 7-15.

Brinkgreve, R.B. and Vermeer, P.A. (1988). Plaxis Ver. 7.2 Manual. *Finite element code for soil and rock analyses*. Balkema, Rotterdam, Brookfield.

Dakoulas, P. and Gazetas, G. (1986). Seismic shear strains and seismic coefficients in dams and embankments. *Soil Dynamics and Earthquake Engineering*, **5:2**, 75-83.

Hynes-Griffin, M.E. and Franklin, N.G. (1984). Rationalizing the seismic coefficient method. *Miscellaneous Paper GL-84-13*, U.S. Army Corps of Engineers Waterways Experiment Station, Vicksburg, Mississippi, 21p.

Bathurst R.J. and Cai, Z. (1995). Pseudo-static seismic analysis of geosynthetic-reinforced segmental retaining walls. *Geosynthetics international* **2:5**.

Makdisi, F.I. and Seed, H.B. (1978). Simplified procedure for estimating dam and embankment earthquake-induced deformations. *ASCE Journal of Geotechnical Engineering Division*, 104p.

Marcuson, W.F. III. (1981). Moderator's report for session on 'Earth dams and stability of slopes under dynamic loads'. *Proceedings, International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*, St. Louis, Missouri, **Vol.3**, p. 1175.

Newmark, N.M. (1964). Effect of earthquake on dams and embankments," *Geotechnique*, **15**, 135-160.

Sakamoto T., Yoshida H., Yamaguchi Y., Satoh H. and Tomoya I. (2002). Numerical simulation of sliding of an earth dam during the 1995 Kobe earthquake. The 22nd USSD Annual Meeting and Conference Pre-conference Workshop 3rd U.S.-Japan Workshop on Advanced Research on Earthquake Engineering for Dams.

Seed, H.B. (1979). Considerations in the earthquake-resistant design of earth and rockfill dams. *Geotechnique*, **29:3**, 215-263.

Seed, H.B. and Martin, G.R. (1966). The seismic coefficient in earth dam design. *Journal of the Soil Mechanics and Foundation Division*, ASCE**, 92, No. SM3**, 25-58.

Terzaghi, K. (1950). Mechanisms of landslides. *Engineering Geology* (Berkley) Volume, Geological Society of America.