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RELIABILITY OF AN EARTH DAM EXCITED BY SPATIALLY VARYING EARTHQUAKE GROUND MOTION

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ABSTRACT

Recent research has shown that significant variation of earthquake ground motion exists over the base dimensions of large structures (Harichandran and Vanmarcke 1986 and many others). The response analysis of an earth dam subjected to spatially varying earthquake ground motion (SVEGM) generally requires a 3-D finite element model in order to accurately represent the spatially varying excitation at the base and sides of the dam. Since SVEGM is usually characterized using probabilistic models, the seismic analysis of the dam is best performed using random vibration analysis, as long as linear response is assumed. A detailed study of the stochastic response of the Santa Felicia earth dam located in California has been performed by the authors (Chen 1995, Chen and Harichandran 1995). This paper examines the reliability of the dam under general SVEGM and identical base excitations using the Mohr-Coulomb failure criterion. Reliability index contours are shown over the entire dam and the results indicate that the effect of SVEGM can have a significant effect on the stability of earth dams.

KEYWORDS

earth dam, spatial variation, random vibration, reliability index, Mohr-Coulomb, three-dimensional, finite element

INTRODUCTION

The Santa Felicia earth dam located on Piru Creek, 65 km northwest of Los Angeles, is 83.8 m (275 ft) high above its rock foundation and 137.2 m (450 ft) long across the valley at the base. The crest has a width of 9.14 m (30 ft) and a maximum length of 388.6 m (1,275 ft). The dam is made of a central impervious core and pervious shell upstream and downstream resting on a stiff layer of gravel and sand down to bedrock as shown in Fig. 1. The core and shell materials are basically alluvial, consisting of clay, sand, gravel and boulders. The Santa Felicia dam has been studied extensively using more conventional ground motion models (Abdel-Ghaffar and Scott, 1979; Prevost *et al.*, 1985). Soil-structure interaction effects, which are not significant for a dam situated on bedrock, were neglected. Due to confining pressure, the shear moduli of the soils vary with depth from the crest of the dam. A three-dimensional inhomogeneous finite element model consisting of 1,004 nodes and 4,140 tetrahedral elements was used in this study. The I-DEAS VI.*i* general purpose finite element com-

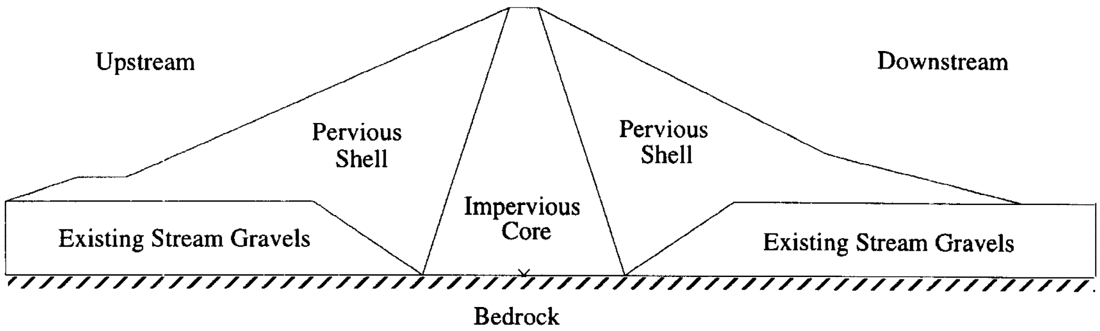

Fig. 1. Santa Felicia earth dam model side view

puter program was used for the modeling and to extract mode shapes and matrices necessary for the random vibration analysis. The fundamental vibration mode computed in I-DEAS is shown in Fig. 2.

GROUND MOTION MODEL

The ground motion model proposed by Harichandran and Vanmarcke (1986) is adopted in this study. The model, which accounts for both incoherence and wave passage effects, is used to specify the base motions in the upstream-downstream direction of the dam. The cross spectral density function (SDF) between the accelerations at two locations *A* and *B* is expressed as the product of the point SDF, coherency function and phase delay:

$$S_{\ddot{u}_A \ddot{u}_B}(\omega) = S_{\ddot{u}_g}(\omega)|\gamma(\nu, \omega)|e^{-i\omega\nu/V} \tag{1}$$

in which the point SDF is the modified Kanai-Tajimi spectrum given by

$$S_{\ddot{u}_g}(\omega) = \left[\frac{\omega_g^4 + 4\omega^2\omega_g^2\zeta_g^2}{(\omega_g^2 - \omega^2)^2 + 4\omega^2\omega_g^2\zeta_g^2}\right]\left[\frac{\omega^4}{(\omega_f^2 - \omega^2)^2 + 4\omega^2\omega_f^2\zeta_f^2}\right]S_o \tag{2}$$

and the empirical coherency function is

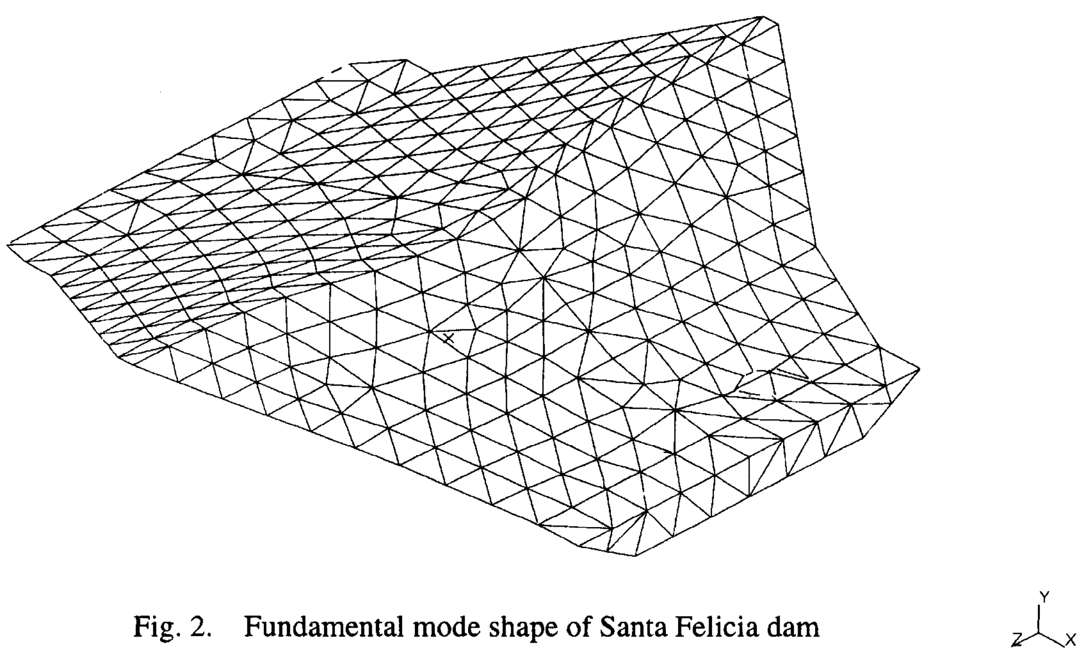

Fig. 2. Fundamental mode shape of Santa Felicia dam

$$|\gamma(\nu, \omega)| = A\exp\left[-\frac{2\nu}{\alpha\theta(\omega)}(1 - A + \alpha A)\right] + (1 - A)\exp\left[-\frac{2\nu}{\theta(\omega)}(1 - A + \alpha A)\right] \quad (3)$$

in which the frequency variation is expressed through the scale of fluctuation

$$\theta(\omega) = k[1 + (\omega/\omega_o)^b]^{-1/2} \quad (4)$$

The parameters ω_g, ζ_g, ω_f and ζ_f were estimated from the El Centro earthquake record by fitting the function expressed in equation (2) to the observed acceleration spectrum. The intensity parameter S_0 was adjusted such that the standard deviation of the ground acceleration is $0.09g$ corresponding to a peak acceleration of about $0.27g$. A, α, k, ω_0 and b are empirical model parameters, ν = separation between locations A and B, and V = apparent wave propagation velocity from A to B. The values used for these parameters are given in Table 1 and correspond to Event 20 recorded by the SMART-1 array in Lotung, Taiwan (Harichandran, 1991).

Two types of excitations are considered in this paper: (a) general excitation with both incoherence and wave passage effects as described by (1); and (b) identical excitation for which all cross-SDFs are given by the point SDF (i.e., no correlation or wave passage effect).

RELIABILITY ANALYSIS

The Mohr-Coulomb criterion is usually used to determine the strength of soil. It has the form

$$\tau_s = c + \tau' \tan\phi \quad (5)$$

in which c = cohesion, ϕ = friction angle, τ' = effective normal stress and τ_s = shear strength of the soil. If the shear stress due to loading exceeds the shear strength given by (5), then the soil would fail locally. In terms of principal stresses, (5) can be written as

$$\frac{\tau_1 - \tau_3}{2} = \frac{\tau_1 + \tau_3}{2}\sin\phi + c\cos\phi \quad (6)$$

in which τ_1 and τ_3 are the major and minor principal stresses, respectively. The effect of the intermediate principal stress in 3-D problems is neglected by the Mohr-Coulomb criterion. When the stresses τ_1 and τ_3 and/or the material parameters c and ϕ are random, safety against failure may be expressed by the safety margin failure criterion

$$Y = \frac{\tau_1 + \tau_3}{2}\sin\phi + c\cos\phi - \frac{\tau_1 - \tau_3}{2} \quad (7)$$

Failure occurs when $Y < 0$, and hence the probability of failure is $p_f = P[Y < 0]$. Rather than using p_f, a popular measure that is used to assess the reliability of engineering structures is the reliability index defined as

$$\beta = \frac{\bar{Y}}{\sigma_Y} \quad (8)$$

Table 1. Ground motion model parameters

ω_g (rad/s)	ζ_g	ω_f (rad/s)	ζ_f	A	α	k (m)	ω_0 (rad/s)	b	V (m/s)
15.0	0.55	3.0	0.60	0.636	0.0186	31,200	9.49	2.95	4,270

where $\bar{Y}$ and σ_Y are the mean and standard deviation of Y, respectively. For a non-linear failure criterion the magnitude of the reliability index is dependent on the form of the failure criterion. For example, a failure criterion written in terms of the factor of safety can yield a different β-value than a criterion written in terms of the safety margin. A comprehensive reliability analysis should use the Hasofer-Lind reliability index (Madsen *et al.*, 1986), which is invariant with respect to different but mechanically equivalent formulations of the failure criterion. However, to keep this illustration simple, the reliability index based on the safety margin is used.

One method of assessing safety against local failure in a finite element analysis of earth dams is to compute the β-value at each node. The principal stresses τ_1 and τ_3 should include the effects of both gravity and seismic loads. In this illustration, the material parameters c and ϕ are assumed to be deterministic (i.e., constant). Typical friction angles of 36° and 42° are adopted for the shell and gravel, respectively. The determination of the consolidated-undrained shear strength of clay is described in Lowe (1967) and Johnson (1975), and based on their suggestions a typical friction angle of 20° and cohesion of 100 kN/m^2 are used for the clay core. The nodes of the finite element model located at the interface between the core and shell or between the core and gravel were assigned to have the properties of the core, and the nodes between shell and gravel were assigned to have properties of the gravel material. For simplicity, these properties are assumed to be homogeneous for each material region. The excess dynamic pore water pressure induced during an earthquake is neglected.

Soil strength depends on the effective stress. The Santa Felicia dam has a free board of 20 ft. A full water level is assumed so that on the upstream side the effective vertical normal stress due to gravity load is determined by

$$\tau' = (\text{height of the soil column}) \times \gamma' \qquad (9)$$

where $\gamma' = \gamma_{sat} - \gamma_w$ equals the submerged unit weight of soil. The gravity-induced stresses were computed using I-DEAS by performing a static analysis of the Santa Felicia dam subjected to an acceleration of $1g$ in the vertical direction. The computed horizontal normal stresses were about 50% to 60% of the vertical normal stresses τ'.

Linear random vibration analysis was used to compute the statistics of the Cartesian stresses at each node due to the general and identical random ground motion models (Chen and Harichandran, 1995; Chen, 1995). The Cartesian stresses due to gravity effects are deterministic and constitute the means of the total stresses, since the mean Cartesian stresses due to the ground motion are zero. The principal stresses τ_1 and τ_3 are non-linearly related to the Cartesian stresses and hence so is the safety margin Y. The statistics of Y, and hence the value of β at each node, were computed by simulating 10,000 realizations of the Cartesian stresses at each node consistent with the statistics of these stresses, computing Y for each realization, and then computing the statistics of the 10,000 Y-values (Chen, 1995).

RESULTS AND DISCUSSION

The reliability indices computed at each node for identical excitation are displayed as shaded image contours over the entire dam in Fig. 3. The figure shows the contours on the vertical XY cutting plane at the mid-length of the dam. The region in which the reliability index is less than zero is the potential failure zone. It is seen that the material in the bottom half of the dam has high reliability against local failure. However, the regions at about two-thirds the height of the dam in the shell as well as in the core show local failure. When the potential failure zones are narrow and isolated, sliding failure does not develop. However, if the potential failure zones expand to some extent and are connected with each other, sliding of the slope as shown in the figure may occur (Okamato, 1984). Note that the zigzag shape of contour boundaries is due to the coarse resolution of the finite element mesh.

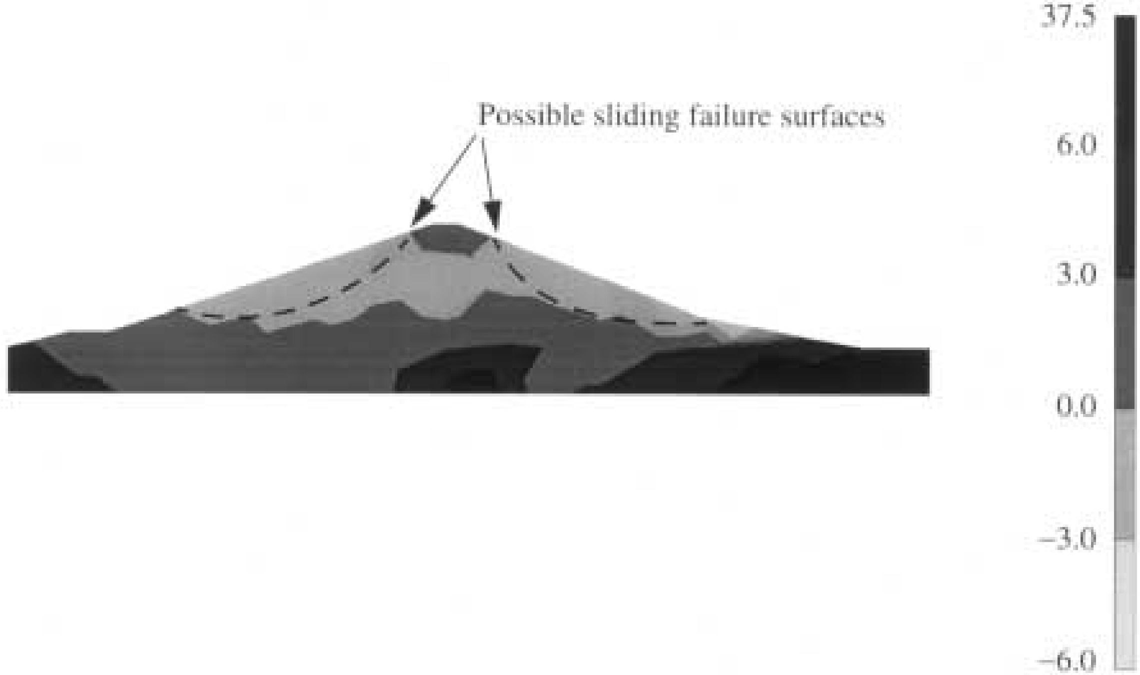

Fig. 3. Reliability index contour on the vertical cross section at mid-length for identical excitation

Figure 4 displays the reliability index contours on the vertical XY cutting plane at the mid-length of the dam for the general excitation case. Larger regions of local failure appear to occur in the dam due to the general excitation than due to identical excitation, especially in the gravel stream-bed. It would appear that a greater variety of sliding failures could occur due to the SVEGM than due to identical excitation. In particular, sliding masses could include parts of the gravel stream-bed.

The Santa Felicia earth dam located in southern California experienced the Northridge earthquake of January 17, 1994. The ground acceleration of the Northridge earthquake had a peak value of about $0.27g$ in the up-

Fig. 4. Reliability index contour on the vertical cross section at mid-length for general excitation

stream-downstream direction, which is close to the peak acceleration of the simulated El Centro earthquake used in this analysis. However, no severe damage except a $\frac{1}{8}$-inch transverse crack in fill placed to connect an access road to the crest of the dam at the abutment was reported due to the Northridge earthquake (Stewart, 1994). The analysis results therefore appear to be unrealistic. There are several reasons why the results of the analysis are expected to be over-conservative:

- Linear analysis of the dam was performed by assuming that the entire construction took place in a single operation. In reality, earth dams are constructed through a definite sequence of operations. In embankments, the behavior of soil at a particular stage of loading is dependent upon the state of stress and the stress history. Thus the stresses in the final configuration depend on the sequence of intermediate configurations and loadings (Desai and Abel, 1972). Nonlinear finite element analysis can be used to compute the stresses in the final configuration of the slope due to the construction and compaction sequence (Dunlop and Duncan, 1970). The computed horizontal normal stresses at a node in this analysis were typically about one-half of the vertical stress at the node. However, for over-consolidated dam soil the lateral earth pressure may be larger than the vertical stress (i.e., the lateral earth pressure coefficient can be greater than unity), and the shear strength of the soil is expected to be larger than assumed.

- To determine sliding failure, the number of cycles of peak shear stresses required to cause failure at points along a potential sliding surface should be taken into account (Peakcock and Seed, 1968). The Mohr-Coulomb criterion assumes that failure occurs even under a single excursion of the shear strength during dynamic response. However, in reality, several excursions of the shear strength are required during dynamic response to cause sliding failure; the required number of excursions is a function of confining pressure, void ratio, relative density, etc. Thus, a more realistic failure criterion should include not only the shear strength of soil but also the number of excursions of the shear strength by the dynamic load induced shear stress.

- Earthquake induced sliding failure in the shell has been found to be related to the gradient of the dam slope (Okamoto, 1984). The intensity of vibration necessary to cause sliding failure is lower for steeper slopes. The local yielding failure criterion governed by the Mohr-Coulomb relationship does not account for this effect. For global failure, the effect of slope gradient on the sliding of the slope should be taken into account.

- Earthquakes, in reality, are non-stationary random processes. For the Northridge earthquake ground motion recorded at the base of the Santa Felicia dam, the duration of strong shaking was only about 4 seconds. In this study, stationary earthquake excitation and response were assumed. The assumption of stationary response is conservative; a dam that would fail under stationary response may not fail under non-stationary, transient response. Consequently, the reliability analysis based on stationary response is conservative.

A final key issue is that for SVEGM, the stress response in the stiff gravel material at the base of the dam is sensitive to the coherency model used. The use of several coherency models was investigated (Chen, 1995), and for most models SVEGM generated large τ_{max} values near the base, with significant variation from one earthquake event to another. However, for the coherency model proposed by Abrahamson (1993), the critical τ_{max} value at the base was only slightly (5%) larger than that due to identical excitation, and consequently the effect of SVEGM on the dam response is much less significant. For the ground motion model used in this paper, very high stresses near the base occur due to incoherence in the ground displacement. The ground displacement is dominated by motions with frequencies less than about 0.5 Hz. Abrahamson's model is highly coherent at this very low frequency range while most other models investigated displayed some incoherence; hence the discrepancy. This observation highlights the crucial need to accurately characterize the low frequency variation of coherency for SVEGM. However, this task is compounded by the fact that the coherency at such low frequencies cannot be reliably estimated from short duration seismograph records using conventional spectral estimation methods, owing to the loss of resolution resulting from smoothing techniques that are employed.

SUMMARY AND CONCLUSIONS

As a simple illustration of the failure analysis of earth dams subjected to a combination of gravity load and SVEGM, the reliability against local shear failure is assessed. This paper illustrates how the earth dam responses computed through random vibration analysis can be used for this assessment. The effective shear strength of soils is determined by the Mohr-Coulomb criterion and the static pore water pressure on the upstream side is accounted for. The reliability index against local failure is computed at each node and displayed as shaded image contours over the entire dam for identical and general excitation models. The soil regions on the upstream and downstream surfaces and at around two-thirds the height of the dam violate the Mohr-Coulomb criterion due to identical excitation having a r.m.s. acceleration of 0.09g. For SVEGM, the potential failure zones become larger, and include the base gravel material because higher stresses occur in this region for the ground motion model used in this study. However, stresses in the base gravel material are sensitive to the low-frequency variation of the coherency model used for SVEGM, and an accurate characterization of this needs to be done. Reasons why this simplistic illustration is likely to be over-conservative are outlined.

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