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Dam response to incoherent ground motions

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ABSTRACT: Response of long dams to spatially incoherent random ground motions is theoretically investigated considering dam-foundation-reservoir interaction. Dam bending and twisting along its longitudinal axis are evaluated for vertically propagating incoherent seismic shear waves and for surface travelling waves. Dam natural frequencies, modes and modal damping ratios stemming from interaction with the foundation and the reservoir are also investigated.

1 INTRODUCTION

Seismic design of gravity dams is usually based on two-dimensional analysis of one monolith or slice. This model provides the basic information but ignores the interaction between dam monoliths or slices, and assumes uniformity of dam cross-section, construction material, soil properties and seismic ground motion along the dam longitudinal axis. For large dams, the uniformity assumption may be inappropriate and a three-dimensional analysis of the dam-foundation-reservoir system may be called for. A complete finite element discretization of the dam, the reservoir and the foundation medium is, however, computationally very expensive, making this type of analysis rather impractical. In this paper, a simplified procedure, complementary to the slice analysis, is presented which reduces the solution of this complex problem to the analysis of a Timoshenko beam.

While the procedure is general and may handle nonuniformity of all parameters, the focus here is on the lateral response of the system to spatially incoherent seismic ground motion. Free vibration analysis of the dam-foundation-reservoir system is also included and the relative effects of the main parameters on the system vibration are investigated. The analysis presented is an extension of the research reported earlier by Ramadan and Novak (1991a). Here, the ground motion is generated in a different, novel way, interaction with the canyon walls is included and free vibration data are presented in full detail.

2 REPRESENTATION OF SPATIALLY INCOHERENT SEISMIC GROUND MOTIONS

The representation of seismic ground motions for a specific application depends on both the structural system and the method of analysis employed. For linear systems, the random vibration analysis employed by, for example, Hindy and Novak (1980) and Novak and Suen (1987), is most suitable because it provides statistical response characteristics. In this analysis, incoherent ground motions are described by their auto- and cross-spectra. The cross-spectrum of a stationary homogeneous random ground motion between any two stations can be written as

$$S_r(r,\omega) = S_\ell(\omega)R(r,\omega)\exp(i\omega r_V/V) \qquad (1)$$

In Eq. 1, r is the separation between the two stations, ω is the circular vibration frequency, $S_\ell(\omega)$ is the local, invariant auto-spectrum and $R(r,\omega)$ is the frequency dependent coherency function; r_v is the separation r projected into the direction of the dominant travelling wave and V is the apparent travelling wave velocity.

For nonlinear systems, time domain analysis based on ground motion time histories may be preferable. The solution is obtained by the numerical integration of the governing differential equations. Time history representation may also be useful for linear systems in special cases and is more convenient for those engineers who are more familiar with deterministic approaches. Due to a paucity in recorded seismic ground motions for specific sites and station separations, artificially generated motions are usually used. In this study, time histories of ground motions are simulated to match ground motion auto-spectra, cross-spectra and coherency using the technique due to Ramadan and Novak (1991b). For the basic case of co-linear stations, the stationary homogeneous ground displacement at a distance x from a reference point is modelled by

$$u_x(t) = \sum_{k=0}^{Nk} \sum_{i=1}^{N} a_k\{\sqrt{S(\omega_{ik})\Delta\omega}\cos[\omega_{ik}(t-\tau_x) + \phi_{ik} + \pi kx/L_i] + \sqrt{S(\omega'_{ik})\Delta\omega}\cos[\omega'_{ik}(t-\tau_x) + \phi'_{ik} - \pi kx/L_i]\} \qquad (2)$$

In Eq. 2, a_k, $k=0,1,\ldots,Nk$ and L_i, $i=1,2,\ldots,N$ depend on the coherency function used and τ_x is the time lag due to surface travelling waves. The motions simulated by Eq. 2 satisfy both the target auto-spectrum and the target coherency function. For the generation of associated ground velocities or accelerations, Eq. 2 is to be differentiated once or twice, respectively. More details and descriptions of the parameters included in Eq. 2 together with its generalization for two- and three-dimensional domains are given in Ramadan and Novak (1991b). This method works very well as is shown in Figure 1 in which target and simulated spectra and coherencies are compared.

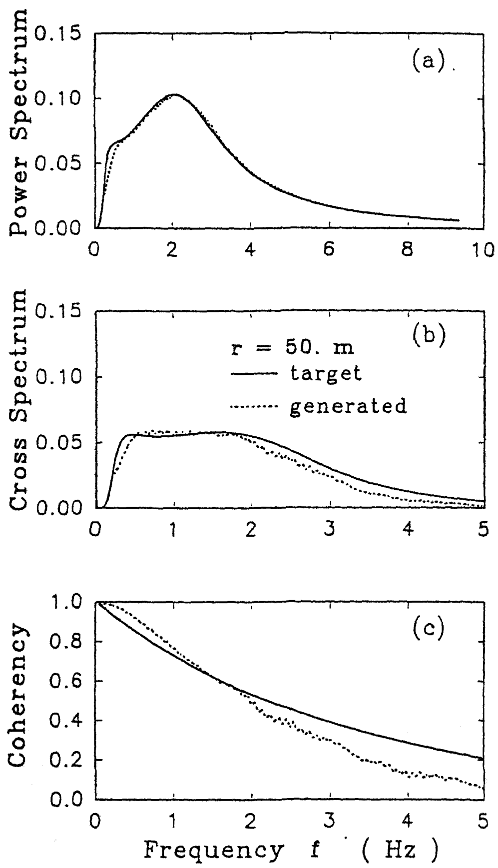

Figure 1. Comparison of target and generated acceleration spectra and coherencies (parameters are given in Section 5).

3 MATHEMATICAL MODEL

Both dam-foundation interaction and dam-reservoir interaction have significant effects on the seismic response of gravity dams and, therefore, should be included in the analysis. In the simplified approach adopted here, the dam length is assumed to be much greater than its cross-section dimensions so that the beam theory may be used to evaluate the variation of the dam horizontal response along the longitudinal axis (Figure 2a). The dam response in the plane of its cross-section is obtained by convqntional analysis of a monolith or slice (Figure 2b). The focus here is on the lateral response of dams with a high aspect (slenderness) ratio, modelled as a Timoshenko beam allowing for bending and twisting along the longitudinal axis, dam-foundation-reservoir interaction and spatial incoherence of the ground motion. Any nonuniformity in the dam cross-section and foundation conditions can also be accommodated. This analysis yields

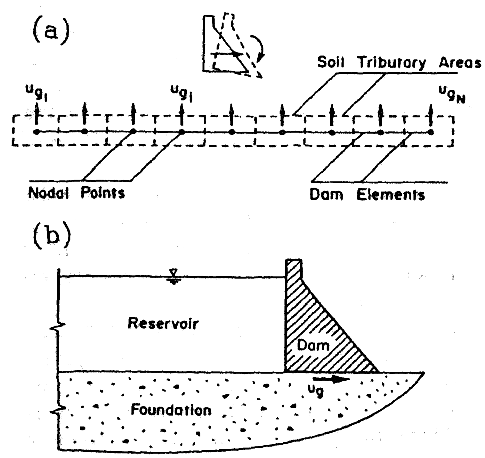

Figure 2. Two-stage analysis of gravity dams:
(a) lateral response; (b) response in cross-sectional plane.

bending and shear stresses of the dam that are totally unforeseen if only the conventional two-dimensional analysis is conducted.

The substructure method is used to evaluate the dam response. In this method, the dam structural stiffness matrix is assembled using the standard structural analysis technique and the foundation stiffness matrix is obtained using the theory of visco-elastic halfspace. The total stiffness matrix is formed by superimposing the two matrices and the reservoir effect. The dam is modeled by N beam elements featuring (N+1) nodes located along the dam longitudinal axis; shear deformation is accounted for. Three degrees of freedom are considered at each node: the horizontal (lateral) translation, rotation in the horizontal plane, and rocking in the vertical plane. The masses associated with horizontal translations and the mass moments of inertia associated with torsion are both lumped at the element nodes. The effect of foundation flexibility is incorporated in the analysis through the complex, frequency dependent stiffness matrix established for the sequence of rectangular tributary areas indicated in Figure 2a. Through foundation coupling between all these areas is considered. The dam is also elastically restrained against rotations and translations at both ends due to interaction with the halfspace, as well as with the walls of the canyon, which is assumed to be rectangular. All stiffness matrices and dam deformations are referred to the dam axis which passes through the centre of gravity of the dam cross-sections.

The reservoir effect is represented by the resultant hydrodynamic pressure force and its moment about the dam axis. These forces are obtained analytically by solving the wave equation assuming an infinite reservoir with uniform cross-section. For more information about the mathematical model and the governing equations of motion, the reader is referred to Ramadan and Novak (1991a).

4 FREE VIBRATION ANALYSIS

The system natural frequencies, vibration modes and damping ratios are obtained by solving the associated eigenvalue problem. For the dam-foundation-reservoir system the eigenvalue problem is nonclassical and nonlinear. Nonlinearity is due to the frequency dependency of both the soil stiffness parameters and the hydrodynamic pressures. In addition, the use of frequency-independent, hysteretic material damping for the soil and the dam materials together with the non-proportional damping generated by the reservoir also contribute to the nonclassical nature of system damping. The eigenvalue problem is solved by means of iteration and the complex eigenvalue analysis. To obtain the k^{th} natural mode, an initial value of the undamped natural frequency is assumed and used in evaluating the complex soil stiffness matrices and the reservoir effects. A complex eigenvalue analysis is then performed to obtain damped and undamped natural frequencies together with the vibration modes. Then, the k^{th} undamped natural frequency obtained is compared with the initial frequency assumed and the analysis is repeated until the two frequencies approach each other with the required degree of accuracy. The whole procedure is then repeated for each vibration mode.

For numerical applications, an 853 m long concrete gravity dam with a constant cross-section similar to that of the Koyna in Western India is chosen. The dam is 103 m high and has a base width of 68.5 m. Its mass density, Young's modulus and Poisson's ratio are 2300 kg/m^3, 30000 MPa and 0.2, respectively. The halfspace foundation is basalt with mass density, shear wave velocity and Poisson's ratio of 2400 kg/m^3, 1218 m/s and 0.3, respectively. Hysteretic material damping ratio is assumed to be 0.02 and 0.05 for the dam and the soil, respectively. The reservoir is assumed to be infinitely long with a constant cross-section and a rigid base. The velocity of sound in water, c, the mass density of water and the water depth, H, are 1440 m/s, 1000 kg/m^3 and 90 m respectively. Effects of sediments are not considered. The dam was discretized into 10 elements featuring 11 nodes. Free vibration analysis was performed to evaluate the first ten vibration modes for four different cases (case 1 to case 4). Through-rock-coupling (interaction between foundation tributary areas) was considered in cases 3 and 4 only while the reservoir was assumed to be empty in cases 1 and 3. The obtained natural frequencies and damping ratios are displayed in Figure 3 (the discrete data are connected to emphasize different cases). Ignoring the through-rock-coupling of foundation results in underestimation of the system damping at low frequencies and overestimation at high frequencies when the reservoir is empty. When the reservoir is full, the damping is underestimated at some frequencies and overestimated at others, depending on the dam-foundation-reservoir interaction. The sudden changes in the trend of natural frequencies and damping ratios for cases 2 and 4 beyond the seventh mode are due to first resonance of the reservoir at its fundamental frequency, $\pi c/2H=25.13$ rad/s. Modifications of the system complex vibration modes due to both through-rock-coupling of foundation and reservoir-interaction were also observed.

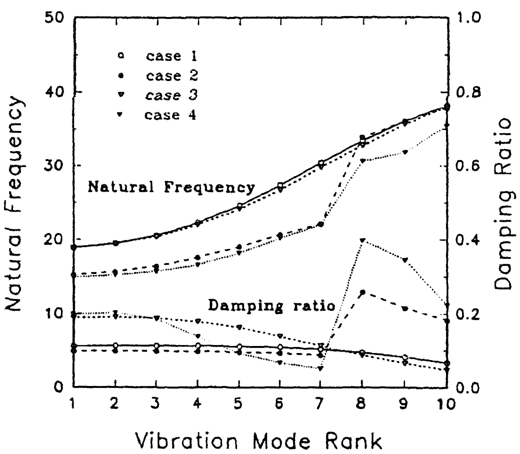

Figure 3. Natural frequencies and damping ratios of the dam-foundation-reservoir system.

5 RESPONSE ANALYSIS

To investigate the effects of ground motion incoherency on long gravity dams, the lateral response of the dam-foundation-reservoir system to simulated ground motions is analyzed. The seismic ground motions are simulated using the technique outlined above. The random field comprising the ground motion time histories is assumed to be homogeneous and therefore the dam response statistical parameters should be symmetrically distributed with regard to the dam midpoint if repeated runs are made. The local power spectrum used is the modified Kanai-Tajimi spectrum (Clough & Penzien 1975) with the parameters $\omega_g = \omega_f = 5\pi$, $\zeta_g = \zeta_f = 0.6$ and $s_o = 0.01$ m^2/s^3. The coherency function is chosen in the simple exponential form, i.e. $R(r,\omega) = \exp[-c(\omega r/V)]$. The ratio c/V is estimated as 0.001 and this value agrees quite well with the radial motions of event 20 of the SMART-1 seismic array. The motions are assumed to be caused by vertically propagating shear waves and, therefore, the wave passage effect is absent. With these parameters, the length scales, defined in Hindy and Novak (1980), assume the values of 3259, 447 and 97 m for the ground displacement, velocity and acceleration respectively.

The simulated accelerations are shown in Figure 4. Nonstationarity characteristics were introduced by superimposing a sine-time-envelope function to the simulated stationary time histories. A 0.02 s time step was used in the simulation. Figure 4 shows the variation of seismic input from station to station. The maximum ground acceleration is 20% of the gravitational acceleration. The target auto-spectrum, cross-spectrum for a separation r=50 m and the coherency function are compared with those of the generated motions in Figure 1. The match is quite good. The variation of the simulated motions along the dam length at five discrete time instances is shown in Figure 5.

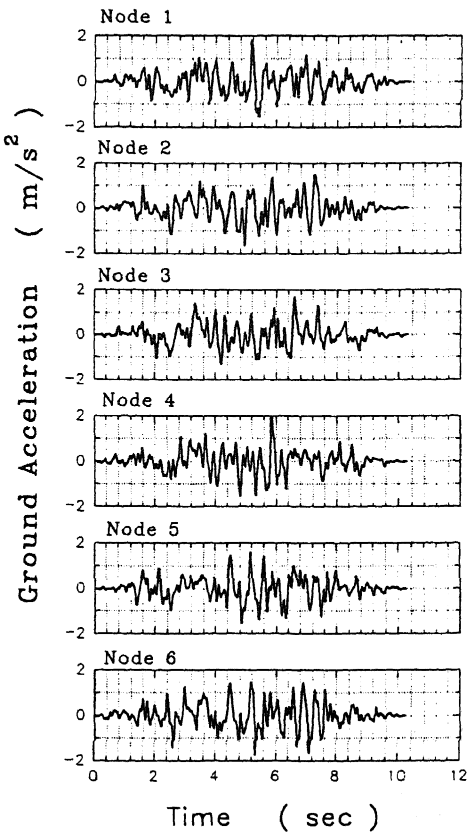

Figure 4. Simulated ground accelerations at dam nodes 1 to 6.

To evaluate the dam response to the simulated motions accounting for the frequency dependence of the foundation stiffnesses and hydrodynamic pressures, the complex response analysis is used. In this procedure, the ground motions are first transferred to the frequency domain using the FFT. The response analysis is then performed in the frequency domain and the results are finally transferred back to the time domain through the inverse FFT. For the response analysis presented here, the reservoir is assumed full, H=90 m, and the through-foundation-coupling is considered. All parameters for the dam, the foundation and the reservoir are as in section 4. The dam lateral response to the spatially correlated (SC) ground motions is shown in Figure 6 in which the response to fully correlated (FC) ground motions is also

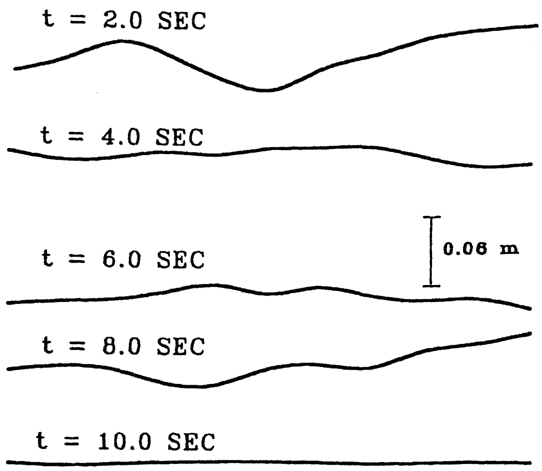

Figure 5. Variation of simulated ground displacements along dam axis.

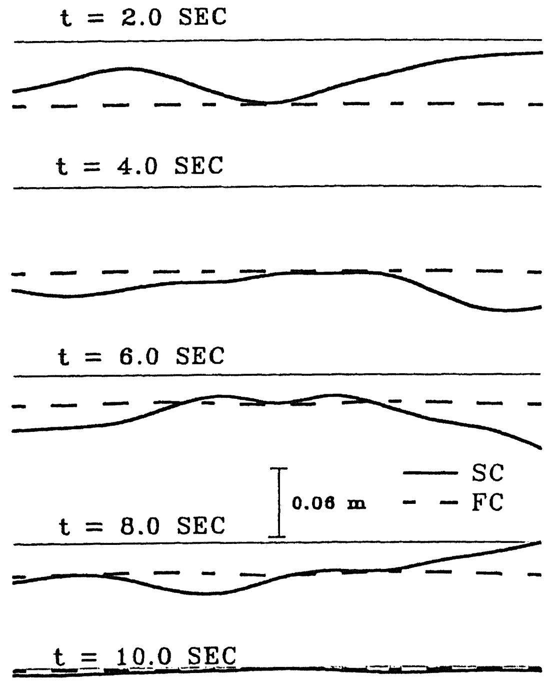

Figure 6. Dam lateral response at different time intervals.

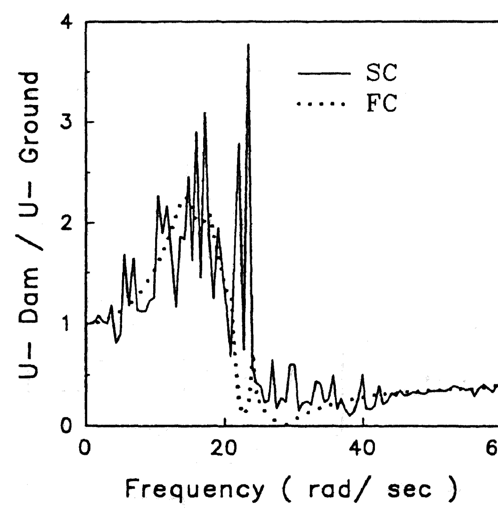

Figure 7. Ratio of dam absolute motion to ground motion at dam midpoint.

shown for comparison. For the FC case, the ground motions at all dam nodes were taken as identical. While the dam responds almost as a rigid body under the FC motions, its bending under the SC motions is quite marked. To demonstrate the dam-foundation-reservoir interaction effects, the Fourier amplitudes of the absolute dam displacement normalized by those of the ground displacement are shown in Figure 7. This figure presents the data at the dam midpoint. Note that more higher vibration modes participate in the response to the SC ground motions than in the case of the FC ground motions. The resonance of the reservoir is pronounced at its fundamental frequency of 25.13 rad/s. Distributions of dam bending and torsional moments are shown in Figures 8 and 9, respectively. The bending and torsional moments developed under the FC motions are due to the dam end conditions. They are higher near the dam ends and decrease towards the dam midpoint. For an infinitely long dam, these stresses would disappear. The maximum bending stress under the SC motions is 7.26 MPa which is very significant. The actual stress level for a specific case will depend on several parameters including the design acceleration, relative structure to foundation stiffnesses, the frequency contents of the ground motion, and also the reservoir resonance frequencies and other factors.

The dam was assumed to be continuous over its entire length in this study. This may be adequate for some dams like the Old Aswan Dam in Egypt (2142 m long) and the Willow Creek Dam in Oregon, U.S.A. (543 m long) which were built without expansion joints. For dams having expansion joints, the stresses may be reduced, depending on the nature of the joints but the along-the-dam response would still be significant.

The dam response to fully coherent travelling waves was also considered. In this case, the ground motions are assumed to be the same at all dam

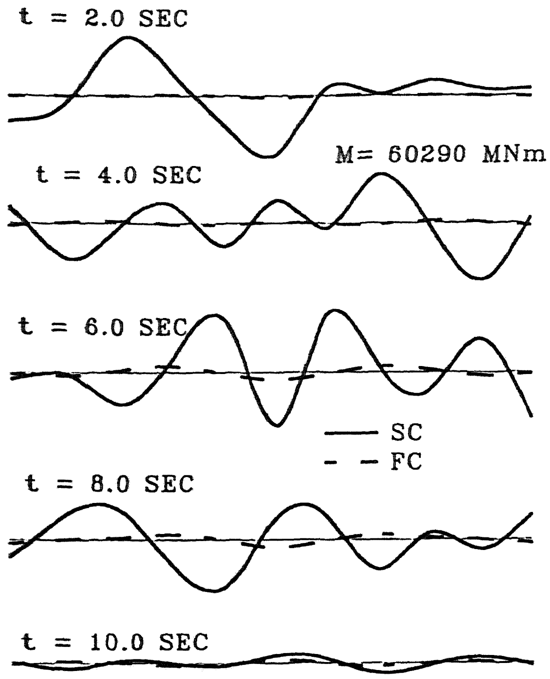

Figure 8. Distribution of dam bending moments.

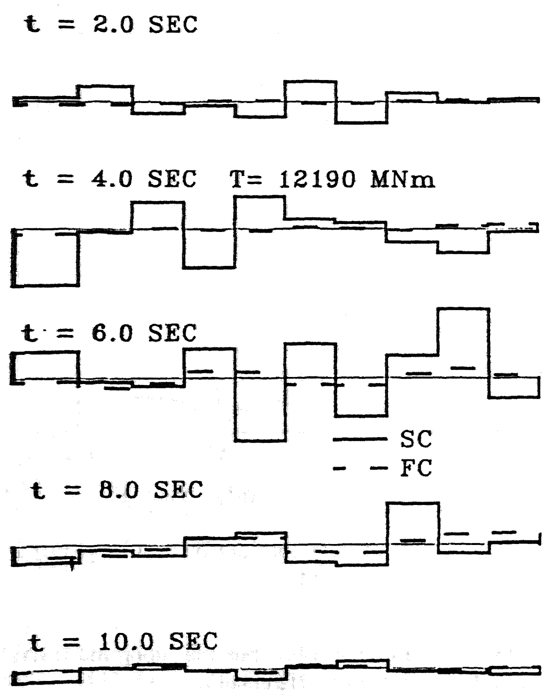

Figure 9. Distribution of dam torsional moments.

nodes but with different time lags calculated from the wave propagating velocity. For waves propagating along the dam longitudinal axis at 4.265 km/s, the maximum bending stress is 1.74 MPa. This bending stress becomes 3.37 MPa when the propagation velocity is taken as 2.133 m/s. This suggests that, for the system considered, the dam stresses due to ground motion incoherence are higher than those due to travelling waves.

6 CONCLUSIONS

Response analysis of a dam-foundation-reservoir system to spatially incoherent seismic ground motions is presented, leading to the following observations:

(1) The dam response is strongly affected by the incoherence of ground motions. The dam responds almost as a rigid body to fully correlated motions but bends and twists significantly under incoherent ground motions.

(2) The stresses caused by the spatial variability of seismic ground motions are not insignificant even under moderate lack of coherence and can be more severe than those caused by travelling waves.

(3) The stresses due to ground motion incoherence remain completely unforeseen in the usual design methods; they should be considered in the design of large structures such as large dams, tunnels, pipelines, and long bridges.

(4) Dam interaction with both the foundation and reservoir has a significant effect on the dam response.

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