SEISMIC SPECTRA FOR SAFETY EVALUATION OF CRITICAL DAM APPURTENANT SYSTEMS AND EQUIPMENT

Najib BOUAANANI¹, Sylvain RENAUD², Sayouba TINTA³, Tarik Saichi⁴

ABSTRACT

Appropriate evaluation of seismic demands along the height of hydraulic structures is crucial for the design and safety evaluation of the structure itself, as well as critical appurtenant systems and safety-sensitive equipment. This paper presents an original investigation of acceleration demands in concrete gravity dams for the evaluation of seismic performance or vulnerability of safety-critical appurtenant structures such as bridges, control unit buildings, spillway support structures, gates, hoist bridges, power supply units, and lifting equipment generally located near dam crest where ground motions can be significantly amplified from dam base. Acceleration demands are obtained using coupled dam-reservoir finite element models. The results presented illustrate the sensitivity of seismic demands affecting dam appurtenant structures and equipment to various factors, including hydrodynamic and reservoir bottom wave absorption effects, reservoir modelling assumptions, time signature and frequency content of applied ground motions, vertical earthquake components, 3D effects and dam size.

Keywords: Concrete gravity dams; Appurtenant structures and equipment; Dam safety; Hydrodynamic effects; 3D effects.

1. INTRODUCTION

The appropriate assessment of seismic demands along the height of dam structures is crucial for their design and safety evaluation as well as that of appurtenant systems and safety-critical equipment, such as bridges, control unit buildings, spillway support structures, gates, hoist bridges, power supply units, and lifting equipment. Amplification of seismic demands in dams may cause significant damage as was documented in several cases, such as the 103 m-high Koyna dam (India) after the 1967 M6.3 reservoir induced earthquake, the 105 m-high Hsingfengkiang buttress dam (China) under the effect of a 1962 M6.1 reservoir induced earthquake, and the 106 m-high Sefid-Rud buttress dam (Iran) following a 1990 M7.3 earthquake (Hansen and Roehm 1979, Arcangeli and Ciabarri 1994, ICOLD 2001). In other events, if damage to the dam itself remained marginal, supported equipment and appurtenant structures were severely affected by amplified ground motions which induced offset or cracking of elements such as walls, parapets, or bridge girders (USCOLD 2000, Matsumoto et al. 2011, Nuss et al. 2012). Therefore, modern guidelines focusing on the earthquake response of dams, such as ICOLD (2010), clearly specify that seismic input at the support of equipment or at the base of appurtenant structures should take account of ground motion amplifications. Such practice has not been always uniformly observed however, especially for older dams and appurtenant structures with initial designs that may fail to meet modern safety criteria. Also, dam engineering analysts are required to select most appropriate assumptions for dam-reservoir modeling without having a sense or prior knowledge of the relative impact on the design or safety evaluation of the studied dam or appurtenant structure and equipment. Informed choices are however crucial considering the critical importance and seismic vulnerability that may be associated
with dam-supported appurtenant structures. This paper aims at contributing to feeding such informed choices through an investigation of the sensitivity of seismic demands affecting dam appurtenant structures and equipment to various factors, including hydrodynamic and reservoir bottom wave absorption effects, reservoir modelling assumptions, time signature and frequency content of applied ground motions, vertical earthquake components, 3D effects and dam size. Several case studies of typical dam-reservoir systems with different geometries will be studied to illustrate the methodology adopted and discuss the obtained results.

2. BASIC ASSUMPTIONS AND CASE STUDIES

Typical concrete gravity dams as shown in Figure 1 are studied in this paper to illustrate the sensitivity of seismic demands on dam-appurtenant structures and equipment to various factors. For each gravity dam studied, two types of numerical models and analyses are considered: bi-dimensional (2D) and three-dimensional (3D). The geometry of a central dam monolith is defined by the height $h_D$, the width $d_D$ of the base, and the width $e_D$ and thickness $c_D$ of the crest. The 3D geometry corresponding to each dam monolith is generated by extruding the central dam section towards reservoir banks. The lengths at the crest and the base of a generated 3D dam model are denoted by $a_D$ and $b_D$, respectively, as illustrated in Figure 1. The impounded reservoir, of height $h_R$, is assumed to extend to infinity upstream. For this purpose, a special boundary condition is placed at a truncation distance $\ell_R$ far enough from dam face. In the 3D models, the dam face section is extruded to generate the volume occupied by the reservoir, except that the reservoir’s height $h_R$ can be less than the dam section’s height $h_D$. A modulus of elasticity $E_D = 25$ GPa, a Poisson’s ratio $\nu = 0.2$, and a density $\rho_D = 2400$ kg/m$^3$ are adopted as concrete material properties for all the dams studied. The following main assumptions are made: (i) the dam has a linear elastic behavior; (ii) the dam foundation is rigid; (iii) the water in the reservoir, of mass density $\rho_R = 1000$ kg/m$^3$, is assumed inviscid, with its motion irrotational and limited to small amplitudes; (iv) gravity surface waves in the reservoir are neglected; and (vi) the effects of sediments that may be deposited at reservoir bottom are considered. The dam will be referred as dry or wet depending whether the reservoir is empty or not, respectively.

![Figure 1. Geometries of the concrete gravity dams studied: (a) 2D models, (b) 3D models.](image)
Table 1 contains the geometry parameters of the dams studied in this paper. Dam D1 and larger dam D2, with heights of 35 m and 90 m, respectively, are considered to evaluate the effects of dam size on seismic demands. The effects of a wide vs. a narrow impounded reservoir are also investigated by studying the seismic response of models D1-3D-W vs. D1-3D-N and D2-3D-W vs. D2-3D-N.

Table 1. Geometry parameters of the dams studied.

<table>
<thead>
<tr>
<th>Nomenclature</th>
<th>Model and analysis</th>
<th>Geometry parameters (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dam D1-2D</td>
<td>Bi-dimensional (2D)</td>
<td>35 27.5 5.0 5.0  - - 32</td>
</tr>
<tr>
<td>Dam D2-2D</td>
<td>Bi-dimensional (2D)</td>
<td>90 70 5.0 8.0  - - 86</td>
</tr>
<tr>
<td>Dam D1-3D-W</td>
<td>Three-dimensional (3D)</td>
<td>35 27.5 5.0 5.0 350 315 32</td>
</tr>
<tr>
<td>Dam D1-3D-N</td>
<td>Three-dimensional (3D)</td>
<td>35 27.5 5.0 5.0 350 0 32</td>
</tr>
<tr>
<td>Dam D2-3D-W</td>
<td>Three-dimensional (3D)</td>
<td>90 70 5.0 8.0 900 810 86</td>
</tr>
<tr>
<td>Dam D2-3D-N</td>
<td>Three-dimensional (3D)</td>
<td>90 70 5.0 8.0 900 0 86</td>
</tr>
</tbody>
</table>

To investigate the influence of differences in time and frequency characteristics of seismic input, horizontal and vertical ground motion components recorded during Imperial Valley (1940) and Loma Prieta (1989) earthquakes are considered. The corresponding acceleration time-histories and 5%-damped acceleration response spectra are provided in Figure 2, as well as the Peak Horizontal Ground Acceleration (PHGA) and Peak Vertical Ground Acceleration (PVGA).

![Figure 2](image)

Figure 2. Acceleration time-histories and 5%-damped acceleration response spectra of the horizontal and vertical ground motion components considered: (a) and (c) Imperial Valley earthquake (1940) at El Centro; (b) and (d) Loma Prieta earthquake (1989) at Gilroy Array no. 2.

3. NUMERICAL MODELS

Seismic demands within a concrete gravity dam, such as accelerations, can be obtained using a coupled dam-reservoir finite element model as the one illustrated in Figure 1 for a 2D analysis. The gravity dam can be subjected to only horizontal ($u_h$) or to horizontal and vertical ($u_h$ and $u_v$) ground motion components. Seismic demands at a given point P of the dam are obtained by studying the dynamic response of Single Degree of Freedom (SDOF) systems with various vibration frequencies $f_s$ attached to point P, while the dam is excited by a ground accelerations applied at its base. The SDOF systems, can represent dam-supported appurtenant structures or equipment, with mass $m_s$, stiffness $m_s$ and viscous damping $c_s$. The mass of the appurtenant SDOF system is assumed to be too light so that its dynamic response does
not affect that of the dam monolith. The equation of motion of the appurtenant SDOF can be written as

\[ m_s \ddot{u}_s + c_s \dot{u}_s + k_s u_s = -m_s (\ddot{u}_p + \ddot{u}_g) \]  

(1)

where \( \ddot{u}_p \) denotes the horizontal acceleration at point P of the dam relative to dam base, and \( u_s, \dot{u}_s \) and \( \ddot{u}_s \) the horizontal displacement, horizontal velocity and horizontal acceleration of the secondary SDOF system relative to point P, respectively. The seismic acceleration demand \( \Gamma(z_P) \), at a point P of coordinate \( z_P \), is defined hereafter as the maximum absolute acceleration response \( |\ddot{u}_s + \ddot{u}_p + \ddot{u}_g| \) of the secondary SDOF system for a given vibration frequency \( f_s = 1/(2\pi) \sqrt{k_s/m_s} \) and damping coefficient \( c_s \) or equivalent damping ratio \( \xi_s = c_s/(4\pi m_s f_s) \). We denote by \( \Gamma^*(z_P) \) the maximum acceleration at a point P of coordinate \( z_P \) over the whole range of frequencies \( f_s \) considered, i.e. from 0 to 50 Hz in this paper.

![Figure 3. Methodology for the computation of seismic acceleration demands within a gravity dam.](image)

The acceleration \( \ddot{u}_p \) at point P relative to the dam base required in Eq. (1) can be obtained using a coupled dam-reservoir finite element model. For this purpose, the dam structure is modelled using conventional 2D or 3D solid finite elements, while 2D or 3D potential-based fluid finite elements, formulated based on water velocity potentials, are used to model the impounded reservoir. Under seismic loads, dam vibrations cause water motions normal to the dam-reservoir interface, and the induced-pressure within water cause additional hydrodynamic pressure to act on dam upstream face. The dynamic responses of the dam and the reservoir are then coupled at dam-reservoir interface through compatibility between dam accelerations and hydrodynamic pressures. Such coupling can also be enforced through compatibility between water velocity potentials and dam normal velocity at dam-reservoir interface. This procedure is known as the \( \phi - U \) formulation, since it is based on displacements \( U \) in the dam and velocity potentials \( \phi \) in the reservoir. The main equation governing the coupled dynamic response of the dam-reservoir system can then be written as (Zienkiewicz and Newton 1969, Olson and Bathe 1985, Bouaanani and Lu 2009)

\[
\begin{bmatrix}
M_{DD} & 0 \\
0 & -M_{RR}
\end{bmatrix}
\begin{bmatrix}
\ddot{U} \\
\Phi
\end{bmatrix}
+ 
\begin{bmatrix}
C_{DD} & C_{DR} \\
C_{RD} & C_{RR}
\end{bmatrix}
\begin{bmatrix}
\dddot{U} \\
\dot{\Phi}
\end{bmatrix}
+ 
\begin{bmatrix}
K_{DD} & 0 \\
0 & -K_{RR}
\end{bmatrix}
\begin{bmatrix}
U \\
\Phi
\end{bmatrix}
= 
\begin{bmatrix}
-M_{DD} \ddot{u}_g(t) \\
-C_{RD} \ddot{u}_g(t)
\end{bmatrix}
\]  

(2)

where \( U \) and \( \Phi \) are vectors containing the nodal displacements relative to the ground and the nodal velocity potentials, respectively, \( M_{DD} \) and \( K_{DD} \) are the structural mass and stiffness matrices of the dam, respectively, \( C_{DD} \) is a damping matrix of the dam structure that can be determined using a Rayleigh damping, equivalent to a modal damping \( \xi_D \) or hysteretic damping \( \eta_D \), \( M_{RR} \) and \( K_{RR} \) are the potential and kinetic energy matrices of the impounded reservoir, respectively. \( C_{RD} \) is a matrix coupling the velocity potential to displacements on the dam-reservoir interface, matrix \( C_{RR} \) accounts for damping due
to energy dissipation at reservoir bottom and far upstream boundary of the reservoir, \( \ddot{u}_g \) and \( \dot{u}_g \) are prescribed ground accelerations and velocities, and \( \mathbf{1} \) is a column vector with the same dimension as \( \mathbf{U} \), containing ones when a translational degree of freedom corresponds to the direction of earthquake excitation, and zero otherwise. The following boundary conditions are considered: (i) a free surface boundary condition, (ii) a compatibility boundary condition at the vibrating dam-reservoir interface (Fenves and Chopra 1984, Bouaanani and Lu 2009), (iii) a radiation boundary condition to prevent reflection of waves at the far upstream of the reservoir (Sommerfeld 1949, Zienkiewicz and Newton 1969, Bouaanani and Lu 2009), and (iv) a boundary condition accounting for energy dissipation due to sedimentation at reservoir bottom Hall and Chopra 1982, Fenves and Chopra 1984, Bouaanani and Lu 2009). The last two boundary conditions can be modeled using infinite fluid elements placed at the upstream end of the reservoir and by viscous dampers placed at reservoir bottom as illustrated in Figure 4. More details about the modelling methodology can be found elsewhere (Bouaanani and Renaud 2014). The coupled finite element of dam D1-2D is shown in Figure 5 for illustration purposes. The same methodology is extended to construct the 3D finite element models.

![Figure 4. Finite element model of a dam-reservoir system and corresponding boundary conditions.](image)

![Figure 5. Finite element model of dam D1-2D impounding a rectangular reservoir.](image)

**4. SELECTED RESULTS AND DISCUSSIONS**

The coupled 2D and 3D finite element models described earlier were built using the finite element
software ADINA (2017) and are used next to investigate the sensitivity of acceleration demands within the studied gravity dams to various factors. Due to space limitations, only selected results are presented and discussed next. Seismic demands within the dams are evaluated through two types of responses: (i) floor acceleration spectra $\Gamma(h_D)$ at the crest of the dam expressed as a function of frequency $0 \text{ Hz} \leq f_s \leq 50 \text{ Hz}$, and (ii) profiles of maximum acceleration demands $\Gamma^*(z_p)$ computed along dam height, i.e. for $0 \leq z_p \leq h_D$, over the whole range of frequencies $f_s$ varying between 0 and 50 Hz. In this paper, the profiles of maximum acceleration are determined considering that point $P$ moves along the dam cross-section’s middle line made of two segments relating points A, B and C as indicated in Figure 5 for dam D1-2D. To facilitate comparisons, acceleration demands are non-dimensionalized with respect to the PHGA of the applied earthquake.

4.1 Effects of hydrodynamic loads and reservoir bottom wave absorption

Figure 6 shows the seismic acceleration demands within dam D1-2D considering reservoir bottom surfaces from fully reflective, i.e. $\alpha = 1.0$, to highly absorptive, i.e. $\alpha = 0.2$. A reservoir truncation length $L_B = 4h_D$ was found to be far enough from vibrating dam face to ensure non-reflection of outgoing waves at reservoir upstream under horizontal earthquake excitation. Accelerations demands within the dry dam are also shown for comparison purposes. We observe that hydrodynamic effects are negligible for relatively flexible appurtenant structures or equipment, i.e. low natural frequencies up to about 5 Hz in the present case. Beyond this frequency range, hydrodynamic effects alter acceleration demands either in terms of amplitudes or frequency content. For example, it is found that such effects can lead to amplification or reduction of acceleration demands with respect to the empty reservoir case. This interesting result shows that, in some situations, depending on the dam, the earthquake applied, and the natural frequency of secondary structure, maximum acceleration demands may correspond to the dry dam, and can then be obtained without recourse to reservoir modeling and fluid-structure interaction analyses. Dam crest accelerations indicate that maximum effects of energy dissipation at reservoir bottom are generally concentrated around the main resonant segments of the curves, i.e. near 8-10 Hz. It is observed that acceleration demands are higher as reservoir bottom wave absorption is lower, with the full reflection case, i.e. $\alpha = 1.0$, corresponding to the highest acceleration amplifications. The profiles of maximum acceleration demands along dam height show that these demands increase with lower reservoir bottom wave absorption, and that the lowest peak acceleration demands correspond to an empty reservoir. Maximum amplifications vary from about 10 under the effect of Loma Prieta (1989) ground motion to about 22 under the action of Imperial Valley (1940) earthquake. Overall, it is clearly seen that profiles of maximum acceleration demands are very sensitive to the geometry of the dam studied and applied earthquake.

Figure 6. Effects of reservoir bottom wave absorption on seismic spectra within dam D1-2D: (a) and (b) Imperial Valley (1940) earthquake; (c) and (d) Loma Prieta (1989) earthquake.
4.2 Effects of water modelling assumptions

The previous results were obtained assuming that water in the reservoir is compressible. In this section, we investigate the effect of this assumption by comparing the results to cases: (i) where water in the reservoir is assumed incompressible by considering a very large bulk modulus, and (ii) where hydrodynamic loads are modeled using Westergaard’s added mass formulation (Westergaard 1933). According to the latter formulation, the effect of the reservoir is equivalent to inertia forces generated by a body of water of parabolic shape moving back and forth with the vibrating dam which is assumed rigid. The Westergaard added mass \( m_i^{(W)} \) to be attached to a node \( i \) belonging to dam-reservoir interface can be obtained as

\[
m_i^{(W)} = \frac{7}{8} \rho_R S_t \sqrt{h_R(h_R - z_i)}
\]

where \( z_i \) denotes the height above dam’s base of node \( i \) of the dam-reservoir interface and \( S_t \) the transverse surface area associated to node \( i \) considering a unit-thick slice of the studied gravity dam. Figure 7 compares the results obtained using an incompressible water assumption and added mass formulation to those corresponding to a fully reflective reservoir containing compressible water. The acceleration demands in the dry structure are also plotted for comparison purposes. These figures clearly show the high sensitivity of acceleration demands to reservoir modeling assumptions, and illustrate that water compressibility might affect the dynamic response of appurtenant structures differently depending on the dam and frequency ranges considered. The incompressible water assumption induces the highest acceleration demands at the crest of the dam and generally maximum acceleration amplifications over dam height. Acceleration demands at dam crest corresponding to the three water modelling assumptions are practically identical in the lower frequency range up to about 8 Hz. At higher frequencies, acceleration demands corresponding to incompressible water assumption and added mass formulation are generally closer, as dam flexibility effects diminish. The profiles of maximum acceleration demands confirm the sensitivity of the responses to reservoir modeling assumptions, with differences generally decreasing as the position where seismic demand is computed is lower. The largest differences between the results corresponding to compressible and incompressible water assumptions are obtained at the crest of the dam.

Figure 7. Effects of water modelling assumptions on seismic spectra within dam D1-2D: (a) and (b) Imperial Valley (1940) earthquake; (c) and (d) Loma Prieta (1989) earthquake.
4.3 Effects of earthquake vertical component

The previous analyses considered only the effects of horizontal earthquake components. In this section, the seismic demands on structures and equipment appurtenant to dam D1-2D are investigated under the effect of both horizontal and vertical ground motion components. It is important to note that the reservoir truncation length \( \ell_R \) found to be far enough from dam face to simulate non-reflection of outgoing waves upstream under the effects of horizontal ground motions only, is generally not sufficient when the dam-reservoir system is excited both horizontally and vertically. In the present case, a reservoir truncation length \( \ell_R = 50h_R \) was found to yield appropriate results. Figure 8 illustrates the seismic demands in dam D1-2D obtained including or excluding the effects of vertical earthquake components. It is seen that such effects are negligible for relatively flexible secondary structures with natural frequencies up to about 8 Hz. Beyond this frequency range, larger acceleration demands are obtained when both horizontal and vertical earthquake components are applied, with the highest amplifications corresponding to appurtenant structures or equipment vibrating at nearly 10 to 15 Hz. The profiles of maximum spectral accelerations clearly show that vertical earthquake components induce significantly larger demands, with peak amplifications varying from twice to three times those obtained excluding the effects of vertical earthquake component of Imperial Valley (1940) and Loma Prieta (1989) events, respectively. The largest differences between the results including and excluding vertical earthquake component effects are obtained at the crest of the dam, while diminishing towards its base.

![Figure 8. Effects of vertical earthquake component on seismic spectra within dam D1-2D:](image)

(a) and (b) Imperial Valley (1940) earthquake; (c) and (d) Loma Prieta (1989) earthquake.

4.4 3D effects

Concrete gravity dams have usually been studied using 2D numerical models. In this work, the effects of such simplification on the seismic demands affecting appurtenant structures and equipment are studied. For this purpose, the results obtained using 2D dam models are compared to those from 3D dam analyses including impounded reservoirs from wide to narrow. Figure 9 provides an example of such comparison, where crest acceleration demands and profiles of maximum accelerations along dam height are obtained using dams D1-2D, D1-3D-W and D1-3D-N. It is seen that the 2D numerical model generally gives the lowest acceleration demands, while highest acceleration amplifications correspond to the dam impounding the narrowest reservoir. It is important to note that the differences between these results stem from a combination of two types of 3D effects: (i) differences between the dynamic responses of the dry dams, i.e. structural 3D effects, and (ii) differences in hydrodynamic responses of the reservoirs, i.e. hydrodynamic 3D effects.
4.5 Effects of dam size

In this section, the effects of dam size on the previous results and observations are investigated. For this purpose, the same seismic analyses conducted on dams D1-2D, D1-3D-W and D1-3D-N are carried out on larger dams D2-2D, D2-3D-W and D2-3D-N. The results obtained are provided in Figures 10 to 13. It is globally shown that dam size is an important factor and that the results and observations obtained previously for dam D1 cannot be generalized. For example, Figure 10 shows that the dry structure of dam D2-2D develops the highest acceleration demands near the crest under both earthquakes. This observation opposes what was found for dam D1-2D, and emphasizes the complexity of the seismic behavior of appurtenant structures and equipment, as it results from the interaction of various factors including the earthquake frequency content, the natural frequencies of the dam and appurtenant structures, and hydrodynamic effects.
The effects of reservoir bottom wave absorption on crest acceleration demands within dam D2-2D subjected to Imperial Valley (1940) earthquake follow practically the same trends as dam D1-2D. However, a different behavior is observed when dam D2-2D is subjected to Loma Prieta (1989) ground motion, i.e. crest acceleration demands are found to decrease with reservoir bottom wave absorption contrary to what was observed previously.

The profiles of maximum acceleration demands within dam D2 also present different features. First, maximum amplifications under the effect of Imperial Valley (1940) earthquake are almost identical for all reservoir bottom wave absorption levels except the fully reflective case. Maximum accelerations are found to increase with lower reservoir bottom wave absorption under the effects of Loma Prieta (1989) earthquake, as opposed to observations applying to dam D1-2D.

Other evidences of dam size effects can be seen by comparing the effects of water modeling assumptions, vertical earthquake component or 3D modelling. For example, Figure 11 (b) shows that added mass formulation yields the highest crest acceleration demands within dam D2-2D under the effect of Imperial Valley (1940) earthquake as opposed to what was found for dam D1-2D (Figure 7(b)). Figure 12(a) reveals that dam D2-2D is less affected by the vertical earthquake component of Imperial Valley (1940) ground motion than dam D1-2D (Figure 8(a)). According to Figure 13(d), 3D effects are more predominant for dam D2-3D-N under the effects of Loma Prieta (1989) earthquake than dam D1-3D-N (Figure 9(d)).

5. CONCLUSIONS

This paper presented an original investigation of the sensitivity of seismic acceleration demands within concrete gravity dams to various factors, including hydrodynamic and reservoir bottom wave absorption effects, reservoir modelling assumptions, time signature and frequency content of applied ground motions, vertical earthquake components, 3D effects and dam size. The methodology adopted and obtained results were illustrated through several case studies of typical dam-reservoir systems with different geometries. The dam-reservoir systems were subjected to ground motions with various time- and frequency-domain characteristics.
The following main conclusions could be drawn: (i) Hydrodynamic effects are generally significant in the evaluation of seismic acceleration demands within gravity dams, except for very flexible appurtenant structures or equipment. Such effects were found to lead to either amplification or reduction of acceleration demands with respect to the empty reservoir case, depending mainly on the geometry of the dam-reservoir system, the earthquake applied, and the natural frequency of secondary structure. (ii) The dynamic response of a dam-supported appurtenant structure can be affected by reservoir bottom wave absorption. Maximum effects are however generally concentrated around the main resonant segments of the acceleration demands. (iii) Water compressibility was found to affect acceleration demands differently depending on the dam and frequency ranges considered. Minimum effects were observed for very flexible appurtenant structures or equipment. (iv) Vertical earthquake components were shown to
generally considerably affect seismic demands at near the crest of the studied gravity dams, and (v) 3D effects due to a combination of structural and hydrodynamic 3D responses are generally more predominant for narrower dam-reservoir geometries. Overall, this work clearly emphasizes the complexity of the seismic behavior of appurtenant structures and equipment, as it results from the interaction of various factors, and warns against the temptation of generalizing the results to any dam or appurtenant structure. Each case should indeed be studied thoroughly and the corresponding results interpreted carefully in light of the modeling assumptions adopted.

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7. REFERENCES


