Article

Probabilistic Identification of Seismic Response Mechanism in a Class of Similar Arch Dams

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Abstract: Different numerical models have been proposed for seismic analysis of concrete dams by taking into account the nonlinear behavior of concrete and joints; interaction between the dam, foundation, and reservoir; and other seismic hazard considerations. Less focus, however, has been placed on the real seismic performance of the dams and their relative correlation. This paper investigates the linear and nonlinear seismic performance of two similar high arch dams with relatively different response mechanisms. The response correlation is performed from statistical and probabilistic points of view. Similarities and differences are highlighted, and the best practice to compare the responses in a class of dams is presented. It is found that some demand parameters and seismic intensity measures can reduce the dispersion of the results and increase the correlation. In general, the dam geometry has a direct relation with the deformation and spatial distribution of potential damaged area. However, it is not related to the localized damage at the most critical location. Moreover, the real crack pattern (from nonlinear analysis) is more discrete compared to the continuous overstressed/overstrained regions (from linear analysis).

Keywords: arch dams; probabilistic; nonlinear; seismic; response correlation

1. Introduction

Safety assessment of the existing dams is an important task in risk-based management of infrastructures [1,2]. Risk analysis might be performed on different scales depending on the identified hazard level and importance of the asset. Methods ranging from pure qualitative assessment, to semi-quantitative approaches, to fully quantitative ones might be implemented [3]. Nearly all the quantitative methods depend (to some extent) on a precise numerical model (which reflects the physics-based problem) [4].

In the context of structural engineering, developing an advanced finite element model of the dam and its calibration with site measurements is a first step towards next-generation quantitative risk analysis. Many of the current legislation recognize numerical evaluation as a standard technique for safety assessment of dams [5–9]. The risk-based safety evaluation of dams should consider all the existing hazard scenarios (e.g., flooding, earthquake, aging) and their interactions.

Once the risk associated with a class of dams is evaluated, the repair and rehabilitation can be focused on the high-hazard dams [10]. Therefore, it is important to evaluate the relative capacity of dams subjected to the potential regional hazard scenarios. In the case of co-located portfolio of dams [11], one may evaluate their relative capacity under similar hazard scenarios (whenever possible).

Of the various single- and multi-hazard capacity functions for different dam types, this paper focuses only on seismic hazard for high arch dams. Seismic response of dams is studied from a probabilistic point of view [12]. Various seismic intensity levels (SILs) are also compared, corresponding to different
earthquake return periods. Several linear and nonlinear transient simulations are performed, and the response mechanism of high arch dams is compared. This study helps to identify the relative seismic capacity of different dams when they are subjected to similar hazard scenarios. The results can be used to prioritize the investments on strengthening the low capacity dams.

2. Probabilistic Analysis and Seismic Intensity Levels

Depending on the type of probabilistic analysis, single or multiple SILs might be considered. A single SIL aims to evaluate only a specific annual frequency of exceedance, $\lambda$. A multiple SIL, however, tries to cover a broad range of $\lambda$. Based on the Poisson probability model [13]:

$$P_E = 1 - e^{-\lambda t}$$

where $P_E$ is the occurrence probability during the time life $t$ of structure (generally assumed to be 100 years for dams), and $\lambda$ is inverse of the return period, $T_R$.

International Committee on Large Dams (ICOLD) recommendations are usually adapted for a limited number of SILs, while the Applied Technology Council (ATC) recommendations (for building) are adapted for multiple SIL-based analyses of concrete dams. The following subsections of this paper summarize these recommendations which are mainly adopted from Hariri-Ardebili [14].

2.1. ICOLD Recommendations

There are two basic seismic loads for the design of new dams or the safety evaluation of existing dams [15–17]:

- **Operating Basis Earthquake (OBE)** represents the SIL at the dam site for which only minor (easily repairable) damage is acceptable, and the dam should remain functional. OBE corresponds to the return period of 145 years (50% probability of exceedance in 100 years).

- **Safety Evaluation Earthquake (SEE)** represents the SIL at the dam site for which a dam must be able to resist without uncontrolled release of the reservoir water. The SEE ground motion can be obtained from a probabilistic seismic hazard analysis (PSHA) and/or a deterministic seismic hazard analysis (DSHA). For large and high consequence dams, SEE is defined as:
  - **Maximum Credible Earthquake (MCE)**: produces the largest expected ground motion at the dam site and is estimated based on DSHA. According to ICOLD [15], the ground motion parameters should be estimated at the 84th percentile level.
  - **Maximum Design Earthquake (MDE)**: corresponds to return period of 10,000 years (1% probability of exceedance in 100 years) and is estimated based on PSHA.

Note that for moderate consequence dams, the SEE ground motion parameters should be estimated at the 50 to 84th percentile level (based on DSHA) and need not have a $\lambda$ smaller than $\frac{1}{1000}$ (based on PSHA). For low consequence dams, the SEE ground motion parameters should be estimated at the 50th percentile level (based on DSHA) and need not have a $\lambda$ smaller than $\frac{1}{10000}$ (based on PSHA).

In addition, the following aspects must be considered [18]:

- The three components of the spectrum-matched acceleration time histories must be statistically independent. One cannot scale one of the acceleration components and use in other direction.
- The duration of strong ground shaking shall be selected in such a way that aftershocks are also covered.
- For the safety check of a dam, at least three different earthquakes shall be considered for the SEE ground motion.
2.2. ATC Recommendations

The Applied Technology Council [19] proposes three types of performance assessments (i.e., intensity-, scenario-, and time-based). Time-based performance assessment (TBPA) evaluates a dam’s performance over a period of time, considering all earthquakes that may occur in that period of time, and the probability that each will occur. TBPA considers uncertainty in the magnitude and location of future earthquakes, as well as the intensity of motion resulting from these earthquakes. SILs and the corresponding ground motions are defined as follows:

- Generate a seismic hazard curve, $\lambda$ vs. $S_a(T_1)$, for the dam site.
- Compute seismic intensity range which covers the dam response from no (or negligible) damage to collapse. As a recommendation, the minimum and maximum spectral acceleration can be assumed as: $S_{a,\text{min}}(T_1) = 0.05g$, and $S_{a,\text{max}}(T_1) = S_a(T_1)|_{\lambda=0.00002/\text{yr}}$, where $\lambda = 0.00002/\text{yr}$ corresponds to $T_R = 50,000$ years.
- Split the $[S_{a,\text{min}}, S_{a,\text{max}}]$ range into $N_{IM}$ equal intervals; calculate and record $\Delta \lambda_i$ in each interval; identify the midpoint spectral acceleration in each interval and the corresponding $\lambda_i$. For 2D model of gravity dams, $N_{IM}$ is recommended to be 8; while for 3D arch dams it may be reduced to 4.
- Develop a target response spectrum, $S_{a,\text{trg}}(T)$, based on data collected from each midpoint. Three types of response spectra are acceptable: (1) uniform hazard spectra, (2) conditional mean spectra, and (3) conditional spectra.
- For each target response spectrum, select and scale suites of $n$ ground motion triplets as follows:
  - Select a candidate suite of ground motion triplets from available recorded motions (e.g., PEER [20]).
  - For each ground motion triplet, construct the geomean spectrum for the horizontal components over a period range of $(T_{\text{min}}, T_{\text{max}})$ as $S_{a,\text{geo}}(T) = \sqrt{S_{a,1}^H(T) \times S_{a,2}^H(T)}$, where $T_{\text{min}}$ and $T_{\text{max}}$ can be selected as 0.2$T_1$ and 2.0$T_1$, respectively. $T_1$ is the fundamental period of dam-reservoir-foundation system.
  - Compare $S_{a,\text{geo}}(T)$ and $S_{a,\text{trg}}(T)$, and select those ground motion horizontal pairs which are similar in shape to the target response spectrum within the period range of $(T_{\text{min}}, T_{\text{max}})$.
  - Amplitude-scale all three components of each ground motion triplet by the ratio of $S_{a,\text{trg}}(T_1)$.$S_{a,\text{geo}}(T_1)$.

There is no magic rule for the number of selected ground motions in each level; however, when there is a significant scatter in spectral shape of the selected records or a poor fit to the target spectrum, $n = 11$ or more triplets of motions may be needed. The use of fewer than $n = 7$ motion pairs is not recommended [21].

3. Criteria for Performance Evaluation

Since two sets of linear elastic and nonlinear damage analyses are performed for the dams, two sets of different criteria are required to interpret the results. This paper adopts the criteria proposed by Ghanaat [22], USACE [23] for the linear elastic analysis, and by Hariri-Ardebili et al. [24] for nonlinear simulations.

3.1. Linear Analysis

Results of the linear elastic analysis should be interpreted with respect to some predefined criteria; therefore, some indices are introduced first [14]:

- Demand Capacity Ratio (DCR): This local index refers to the ratio of the calculated stresses or strains in a dam body to the tensile strength of mass concrete or its equivalent strain.
- Cumulative Inelastic Duration (CID): This local index refers to the total duration of stress (or strain) excursions above a stress (or strain) level associated with a certain DCR.
- Damage Spatial Distribution Ratio (DSDR): This global index refers to the ratio of the overstressed (or overstrained) region to total dam area at the specific DCR.
The aforementioned local indices should be computed for at least one critical node. Then, the dam performance should be evaluated using the plots shown in Figure 1, which presents a threshold surface for local indices. The vertical axis, CID (or DSDR), varies between zero and CID* (DSDR*). For arch dams, a CID* = 0.4 and DSDR* = 15% are recommended by USACE [23]. Finally, the evaluation strategy can be summarized as follows:

- If DCR ≤ 1.0 for all NSIL, the dam response is in linear elastic range. No (or minor) damage is expected.
- If 1.0 < DCR < 2.0, the dam response is in nonlinear phase. The status of the critical nodes should be checked as follows:
  - If even one of the critical nodes (completely or partially) exceeds the threshold surface, being in zone B, significant damage is expected. In this condition, performing detailed nonlinear analysis is required.
  - If all the critical nodes are in zone A, application of the linear elastic procedure is allowed only if the DSDR does not exceed the threshold, zone A (of right plot).
  - If all the critical nodes fall below the threshold curve in CID–DCR plot (i.e., zone A), but DSDR exceeds the threshold in DSDR–DCR plot (i.e., zone B), severe damage is expected in the dam body. Performing detailed nonlinear analysis is required.
- If DCR ≥ 2.0, severe damage (at least localized damage) is expected. If it is accompanied by considerable spatial extension, zone B in the right plot, global damage is also expected. In this case, performing detailed nonlinear analysis is required.

Figure 1. Seismic performance evaluation of arch dams based on linear analysis.

Application of these linear analysis based criteria on concrete dams (gravity, arch, and buttress) and their partial calibration with nonlinear models can be found in [24–32].

3.2. Nonlinear Analysis

Results of nonlinear analyses can be directly correlated with safety of the dam [33]. There is no established criteria/metric for nonlinear evaluation of concrete arch dams. This means that there is no limit state (LS) function from nonlinear analyses to correlate its performance with observed damage. However, multiple indices can be considered, such as concrete cracking (and crushing), joint opening/sliding, and the dissipated energy during damage. Different researchers combined these indices with the damage index (DI) concept to present the expected damage in a more quantitative form [34–37]. This paper does not directly use the concept of DI for arch dams. The objective is to compare the critical value of nonlinear damage metrics in two dams which are subjected to similar seismic hazard scenarios. Moreover, a correlation among different damage metrics will be investigated.
4. Case Study Dams

Two high arch dams are used as case studies in this paper. The first one is Dez Dam (hereafter referred to as Dam-1), and the second one is Karun-III Dam (hereafter referred to as Dam-2). The heights of Dam-1 and Dam-2 are 203 and 205 m, respectively; while their crest lengths are 240 and 462 m. General information, as well as the finite element model calibration for these dams, can be found in [38,39].

Dam-1 includes a Pulvino (and peripheral joint), while Dam-2 has massive concrete blocks on its sides. The finite element program ANSYS [40] is used for all simulations. The dams are modeled using six- and eight-node structural elements, Figure 2. Foundation rock is modeled as a mass-less medium, and the reservoir water is included using the pressure-based fluid elements. In the nonlinear analyses, the concrete is modeled based on the extended rotating smeared crack model [41], and the contraction (and peripheral) joints are simulated using the node-to-node contact elements. Lift joints were not considered in this paper. The total numbers of elements in Dam-1 are 792, 3770, and 3660 for the body, foundation, and reservoir, respectively. The numbers of elements in Dam-2 are: 3958, 21,848, and 23,022.

(a) Dam-1  (b) Dam-2

Figure 2. (a,b) Finite element mesh of two case study dams.

The applied loads on the system are: dam body dead load (based on several construction stages), hydrostatic load in normal water level (with gradually impounding the reservoir), silt pressure, thermal loads (with summer temperature condition; Figure 3), and, finally, seismic load (with three component ground motion records). Up to 10 and 4 construction stages were considered in Dam-1, and Dam-2, respectively. In the applied method, the concrete weight for $i$th stage is first applied, followed by grouting of the joints in $i$th stage, and then, concrete weight again for $(i + 1)$th stage. This is continued to construct the entire dam, and to grout all the joints. Material properties can be summarized as follows: Modulus of elasticity in concrete is 40 and 30 GPa for Dam-1 and Dam-2, respectively; Poisson’s ratio is 0.2; mass density for concrete is 2400 kg/m$^3$; and concrete compressive strength is 35 and $[25 – 35]$ (inner and outer parts) for Dam-1 and Dam-2, respectively.

(a) Dam-1  (b) Dam-2

Figure 3. Summer temperature distribution on the dam face resulted from thermal transient analysis.

According to the PSHA of Dam-1, three SILs are identified, and nine site-specific real ground motion records are also selected from NGA-WEST2 list [20]. Each of nine ground motions are scaled to three SILs. Subsequently, $3 \times 9 = 27$ three-component scaled records are produced for each dam. Finally, 27 records are applied to 3 types of finite element models: linear elastic model, nonlinear model with joints, and nonlinear model with concrete damage. This requires a total of $27 \times 3 = 81$ (linear and nonlinear) transient analyses for one dam. The results presented in Section 5 summarize $81 \times 2 = 162$
simulations. Considering the complexity of models, each analysis took from 6 to 60 h to complete in a standard 8 GB-RAM workstation.

5. Results and Discussions

Results are presented in two sections. Section 5.1 presents the response of the linear and nonlinear systems from a statistical point of view. The outputs are presented for each dam separately and compared as a function of SIL. In Section 5.2, the outputs from both dams are paired with different ground motion intensity measure (IM) parameters and compared from a probabilistic point of view.

5.1. Statistical Response Comparison

Figure 4 presents the maximum displacement of dams along the crest and height of the crown cantilever. For each SIL, the mean, \( \mu \), value of nine ground motion records, as well as their lower and upper bounds (i.e., \( \mu \pm \eta \)), where \( \eta \) is standard deviation) are presented. Clearly, increasing the SIL increases the drift response. In general, the drift response of Dam-2 is about 2.5 times that of Dam-1. Their height is about the same; however, the crest length in Dam-2 (including the massive concrete blocks) is about twice that of Dam-1.

Next, the performance of the dams is compared based on their CID–DCR curves, See Figure 5. These plots are based on the stress (or strain) time history in the most critical point within the dam body. Some major observations are:

- The stress-based and strain-based CID–DCR metrics are not identical. In fact, the stress-based CID–DCR metric shows higher CID values for Dam-1, while the opposite is true for Dam-2.
- According to the stress-based criteria, SIL-2 and SIL-3 exceed the threshold in Dam-1, while SIL-3 exceeds the threshold only at DCR = 1 and 2 in Dam-2.
- The strain-based CID–DCR curves for both dams share more similarities. In both cases, the mean curve exceeds the threshold extensively for SIL-3, partially for SIL-2 (only DCR = 1), and never for SIL-1.
- Moreover, the curves associated with Dam-2 decay faster than the curves belonging to Dam-1. This means that the localized damage risk at higher DCR values is lower for Dam-2.

![Figure 4](image-url) (a) Dam-1
![Figure 4](image-url) (b) Dam-2

Figure 4. (a,b) Maximum displacement response of arch dams subjected to three seismic intensity levels.
Cumulative Inelastic Duration–Demand Capacity Ratio (CID–DCR) of arch dams subjected to three seismic intensity levels.

So far, the localized potential damage is studied in the context of CID–DCR curves. The global behavior can be investigated in the context of DSDR–DCR plots, See Figure 6. Again, results are presented in terms of overstressed and overstrained regions on upstream (US) and downstream (DS) faces of the dams. Major findings can be summarized as follows:

- The stress-based and strain-based responses have a similar pattern for Dam-1, while they are different for Dam-2.
- In Dam-2, the strain-based DSRD is higher than stress-based one for lower DCR values.
- For Dam-1, the US face is a bit more critical than DS one. However, Dam-2 has a different mechanism and the DSDR in DS is much higher.
- In Dam-1, only SIL-3 exceeds the threshold at DCR = 1. In Dam-2, both SIL-3 and SIL-2 exceed the threshold at several DCRs (especially at the DS face).
Figures A1 and A2 (in Appendix A) provide a visualization of the DSDR in Dam-1 and Dam-2, respectively. As opposed to Figure 6, which presents the statistics of DSDR at various DCRs, Figures A1 and A2 present the individual DSDR only at DCR = 1. Moreover, the graphical presentation is only provided for the US face of Dam-1 and the DS face of Dam-2 (which are critical face). In Dam-1, the central parts of the body near the crest are the most vulnerable regions during the seismic excitation. In Dam-2, the left and right quarter points in the upper half of the dam are the most vulnerable regions.
The predicted potential damaged area from linear analyses in Figures A1 and A2 are further validated based on a series of nonlinear simulations with the concrete smeared crack model, see Figure 7. Only the results of SIL-3 (which is the critical level) are provided for both the US and DS faces. In general, there is an acceptable consistency between the potential damaged area and the real cracked element for Dam-1. Nonlinear simulations show some limited extra damage next to the dam-foundation interface. The resulting damaged area in Dam-2 is less than the expected damage from linear analysis. The crack pattern is more discrete compared to the continuous overstressed/overstrained regions. The nonlinear simulations, however, show a considerable amount of damaged area at the dam-foundation interface. This is partially identified by linear analysis. Note that the concrete near the foundation is reinforced (to some degrees), and these peripheral cracks may not appear in the real dam.

**Figure 7.** Crack profile of both dams from nonlinear analysis subjected to seismic intensity level-3 (SIL-3) ground motions.

Finally, Figure 8 presents the joint opening and sliding in terms of crack opening displacement (COD) and crack sliding displacement (CSD). Major observations can be summarized as follows:

- In general, the mean CSD is twice and three times the COD for Dam-1 and Dam-2, respectively.
- In general, joint opening is more critical than joint sliding (since the water can penetrate inside the joint and increases the pressure on the inner walls). The COD appears to be well-controlled in both dams.
- The CODs in both dams are very similar: limited to no more than 1 mm for the first 150 m of height. Moreover, there is practically no difference between three SILs up to height 150 m. The major joint opening occurs at the top 50 m (i.e., upper quarter of dam).
- On the other hand, the joint sliding has a more smoothed behavior along the height (especially for Dam-2). Again, the most critical zone is the upper quarter of dam height.
5.2. Probabilistic Response Correlation

Once the relative responses of different dams are identified under three SILs, it is also important to establish a direct relation between the individual ground motion records and their linear/nonlinear response. In this section, a probabilistic model is developed for dam response under ground motion IM parameters. The outcome of this section can also be expanded to develop a so-called analytical response surface meta-model for the case study dams [42] (which is not discussed herein).

Several intensity-, frequency-, and duration-based IM parameters can be extracted from a ground motion record [42]. They might have scalar or vector format. These IM parameters can then be correlated with the response parameters to develop a so-called probabilistic seismic demand model (PSDM). Investigation of the most optimal IM parameter is not the focus of this paper; the existing literature is used to identify the three most important IMs. According to Hariri-Ardebili et al. [24], Hariri-Ardebili and Boodagh [42], peak ground acceleration (PGA), acceleration spectrum intensity (ASI), and the first-mode spectral accelerations ($S_a(T_1)$) are among top IM choices.

On the other hand, since three-component ground motions are used in this paper, the scalar IM parameters for different components need to be combined with an appropriate method. The square-root-of-sum-of-squares technique is used to combine them as:

$$IM^{combo} = \sqrt{IM_{H_1}^2 + IM_{H_2}^2 + IM_V^2}$$  (2)

where $H_1$, $H_2$, and $V$ are two in-plane directions and one vertical direction.

Figure 9 provides PSDM for maximum crest displacement as a function of different IM parameters including the goodness-of-fit in terms of root mean square error (RMSE). Major observations can be summarized as follows:

- PGA and ASI are structure-independent IM parameters; therefore, both curves in Figure 9a,b have identical IM range. $S_a(T_1)$, however, is a structure-dependent parameter. Since the fundamental period of these two dams is different, the spectral values will also be different, See Figure 9c.
- The confidence interval for Dam-2 is larger than Dam-1.
- The slope of the curve in Dam-2 is higher than Dam-1, which shows more correlation between the input and output parameters.
- In order to increase the accuracy of the PSDM, it is possible to develop a multiple IM model, See Figure 9d. Although one may apply a polynomial with different degrees, a planer one is selected in this paper. It is already found that higher order models over-fit the results Hariri-Ardebili et al. [24]. In this plot the upper plane belong to Dam-2.

![Graph](image1.png)

(a) Peak ground acceleration; RMSE\textsuperscript{1} = 6.86; RMSE\textsuperscript{2} = 16.73

(b) Acceleration spectrum intensity; RMSE\textsuperscript{1} = 5.06; RMSE\textsuperscript{2} = 14.39

![Graph](image2.png)

(c) First-mode spectral acceleration; RMSE\textsuperscript{1} = 5.05; RMSE\textsuperscript{2} = 13.77

(d) Multiple intensity measures

Figure 9. (a–d) Correlation between the dam displacement and the seismic intensity measures.

A similar procedure can be applied in nonlinear analyses for the joint response. Figure 10 provides PSDMs for the maximum COD and CSD. Again, three IM parameters are contrasted. The most important finding is that the mean curves and the confidence intervals (from two dams) are nearly parallel (specially for ASI). This proves that there is a high correlation in nonlinear response of these two dams from a probabilistic point of view. Furthermore, this shows that ASI might be a good candidate to compare the joint capacity of different arch dams.
Finally, one may try to correlate different dam responses together (and not in the context of seismic intensity measures). Such an attempt is shown in Figure 11. In order to develop a smooth curve or joint probability model, a relatively large number of simulations is required. Since the initial finite element analyses are limited (because they are computationally demanding), a procedure is used to expand the number of engineering demand parameters (EDPs). This method, which was originally developed by Yang et al. [43], is briefly explained in Algorithm 1. The input is the matrix of analytically determined EDPs (e.g., displacements, joint opening/sliding), $X$, and the output is the matrix of statistically determined EDPs, $W$. 

**Figure 10.** (a–c) Correlation between the joint opening and sliding with seismic intensity measures.
Joint movement [mm]

Cum. Dist. Func. (CDF)

Opening

Sliding

Figure 11. (a,b) Uncertainty quantification of joint opening and sliding.

This method is used to expand the initial simulations, and, subsequently, a total of 10,000 correlated EDPs are generated. Figure 11a shows the cumulative density function (CDF) of the joint opening and sliding in Dam-1 separately. On the other hand, Figure 11b presents the joint probability density function (PDF) of the COD and CSD for Dam-2. Both of these plots are very useful for identifying the relation of various EDPs and developing an integrated risk-based decision model.

Algorithm 1 Generating correlated EDPs from initial finite number of simulations

Inputs: $X_{n \times k}$ $\triangleright$ EDP matrix, where $k$ is the number of different EDPs.

Output: $W$

1. procedure
2. Compute $Y_{n \times k} = \ln X_{n \times k}$ $\triangleright$ It has a joint normal distribution
3. Compute the $M_Y = (\text{mean } (Y))^t$ $\triangleright$ Mean vector
4. $D_Y = \text{diag } (\text{std } (Y))$ $\triangleright$ Diagonal standard deviation matrix
5. $R_{YY} = \text{corrc0ef } (Y)$ $\triangleright$ Correlation coefficient matrix
6. $\Sigma_{YY} = D_Y R_{YY} D_Y$ $\triangleright$ Covariance matrix
7. $L_Y = (\text{chol } (R_{YY}))^t$ $\triangleright$ Lower-triangular decomposition
8. Generate $U$ with $\mu = 0, STD = 1$ $\triangleright$ A vector of uncorrelated standard normal RVs
9. $Z = D_Y L_Y U + M_Y$ $\triangleright$ A linear transformation and translation from $U$ to $Z$
10. $W = \exp (Z)$ $\triangleright$ Transfer back the generated joint normal logarithmic EDPs
11. end procedure

6. Conclusions

Risk-based management of a large portfolio of dams requires information about the relative performance of all the assets and particularly dams. Often, several dams are co-located or within a short distance from each other; thus, similar seismic scenarios might be applied to all of them. This paper presents the finite element results of two arch dams assuming they are subjected to similar seismic hazard scenarios.

Both dams are over 200 m in height; however, one of them has a larger crest length. In addition, one of them was constructed with Pulvino technology, while the other one is in direct contact with the foundation and has two massive concrete blocks on its left and right sides. The finite element model of the dam-foundation-reservoir coupled system is developed with both linear and nonlinear assumptions. For the nonlinear models, the concrete damage is modeled based on a rotating smeared crack approach, while the contraction (and peripheral) joints are simulated with tension-free contact elements. Thermal loads are also taken into account before dynamic analyses. Three seismic intensity levels are studied based on PSHA, and nine ground motion records are scaled in each one; subsequently, a total of over 160 transient analyses are performed for dams. Results are extracted and compared in terms of displacements, DCR, DSDR, CID, joint openings and sliding, and crack pattern.

The geometry of the dam (especially its crest length) found to be directly related with the deformation (See Figure 4). Moreover, it has a direct relation with the spatial distribution of the
potentially damaged area on both the upstream and downstream faces of the dam (See Figure 6). It is not, however, related to the localized damage at the most critical point of the dam (See Figure 5). In fact, it has been shown that the localized potential damage between two dams can be well-correlated with strain-based criteria on CID (See Figure 5, right column).

Next, the crack pattern from nonlinear analyses is compared with potential overstressed/overstrained area from linear simulations. It is found that the real crack pattern is more discrete compared to the continuous DSDR. Finally, the joint opening/sliding displacement from both dams are compared. Although the opening displacements were in the same range, the sliding displacement from the wider dam was twice that of the other dam (See Figure 8).

Last but not least, thanks to the multiple dynamic analyses, a PSDM is proposed for both dams. Surprisingly, the COD-based PSDMs on ASI were nearly identical. Moreover, PSDMs from CSD had parallel lines. This shows that in spite of all differences in geometry, the joint opening capacity of the dams is very close. Finally, a mathematical model is used to expand the initial engineering demand parameters and generate a large set of correlated EDPs. Using this new dataset (which has similar statistical parameters to the initial set), one can easily develop any joint PDF and CDF between the outputs.

Future research can be directed on developing a generalized PSDM for arch dams which is function of not only IM and limit state (LS) but also the geometry variables [44] in the form of $EDP = f(IM, LS, Geo)$.

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Appendix A. Detailed DSDR Plots

![DSDR Plots](image-url)

Figure A1. Matrix of over-stress and over-strain area on Dam-1 upstream face; Note: Each matrix includes 9 plots associated with 9 ground motions. In each matrix, top-left to bottom-right are #1 to #9.
Figure A2. Matrix of over-stress and over-strain area on Dam-2 downstream face; Note: Each matrix includes 9 plots associated with 9 ground motions. In each matrix, top-left to bottom-right are #1 to #9.

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