Probabilistic Analysis of Concrete Gravity Dams Owing to Foundation Inhomogeneity

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Abstract

Assumption of homogenous foundation would bring about a lot of hazardous consequences, although it might reduce the cost of designing. These hazardous consequences root in inhomogeneity of foundation which is caused by large joints or fissures in it. Even though it is impossible to extend a unique model of inhomogeneity to all dams, the effect of mechanical properties of coupled system of dam-foundation-reservoir would be evaluated by considering one of the most critical geological situations which lead to critical responses. In current study, seismic performance of Pine Flat dam founded on inhomogeneous foundation is studied by probabilistic analysis as case study. In this process, three different level of earthquake's intensity are imposed to the same models. Furthermore, the effect of deconvolution process is studied and the results have demonstrated importance of this process, especially in inhomogeneous cases.

Keywords: Concrete Gravity Dams, Inhomogeneous Rock Foundation, Probabilistic Analysis, Deconvolution, Levels of Earthquake's Intensity.

1. INTRODUCTION

The seismic safety of existing concrete dams and the risks posed by earthquakes has been a growing issue in the last decades. The main potential seismic failure modes of concrete gravity dams are tensile cracking and sliding along jointed sections specifically at the dam-foundation interface [1]. Gravity dams are usually evaluated using deterministic analysis methods; however, probabilistic and reliability methods are preferred due to the sources of uncertainty presented in earthquake ground motions and in parameters describing the structural system. Therefore, the seismic reliability assessment is employed as a useful tool in dam safety. This method of analysis for concrete dams is in its early development stage and examples are scarce [2, 3]. It requires an analytical or numerical model that robustly captures the nonlinear structural behavior, and explicit consideration of important sources of uncertainty. These uncertainties may be aleatoric (inherent randomness) or epistemic (lack of knowledge). In a nonlinear seismic analysis, a primary source of modeling uncertainty lies in definition of the analysis model parameters as compared to the components' actual behavior [4]. Application of seismic reliability analysis to gravity dams requires identification of potential failure modes presented as limit-state (performance) functions, and prediction of the conditional probability of limit-state exceedance under different earthquake events. This exceedance probability can be rigorously estimated using statistical techniques in probability analysis [5].

2. PROPOSED METHODOLOGY

The proposed methodology for deconvolution process would be elaborated in this section. For realizing the effect of foundation's mass in amplifying the free-field motion, this process is required. A schematic view of this process is illustrated in Figure 1. For this purpose, it is needed to be conducted in frequency domain by computing the transfer function TF, of the foundation medium through its finite element analysis by applying the available free-field record $a_{ff}(t)$, at the base and computing the acceleration time-history at the top of the foundation, $a_{top}(t)$. The foundation's TF is obtained as:

$$TF(\omega) = \frac{A_{top}(\omega)}{A_{ff}(\omega)} \tag{1}$$

where $A_{top}(\omega)$ and $A_{ff}(\omega)$ are Fourier transforms of the computed top and the applied free-field records, respectively. Then, the inverse of the foundation's TF is used to obtain Fourier transform of the required input base acceleration record, $A_{inp}(\omega)$:

$$A_{inp}(\omega) = \frac{A_{ff}(\omega)}{TF(\omega)+\varepsilon} \tag{2}$$

where ε is a regularization parameter avoids dividing by very low values. The input deconvolved base record $a_{inp}(t)$, that produces the free-field record at the top of the foundation is computed by the inverse Fourier transform of $A_{inp}(\omega)$. It is noteworthy that the foundation could be homogeneous or inhomogeneous but it should behave linearly so the deconvolution process can be applied. Also, because the properties of the foundation such as rock material properties may change in each sample, the deconvolution process should be repeated for each foundation sample to produce the correct free-field record at the top surface.

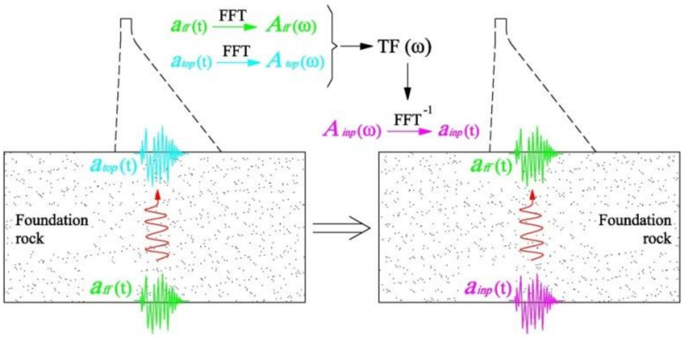
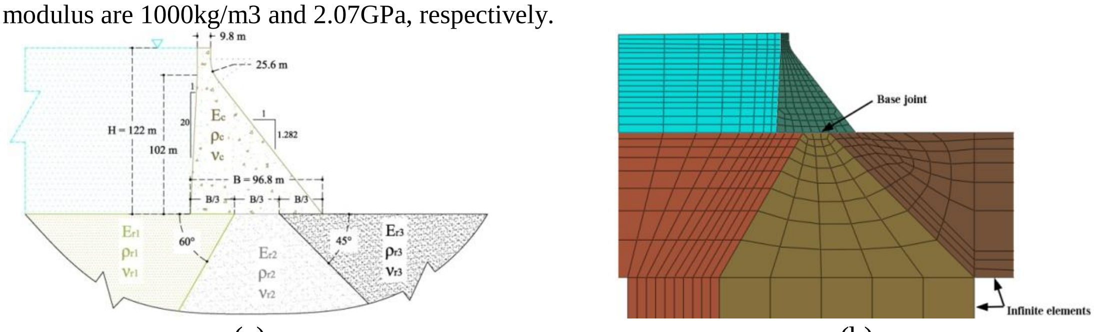


Figure 1. Schematic representation of the deconvolution process

3. APPLICATION EXAMPLE

3.1. PINE FLAT GRAVITY DAM

The proposed methodology is applied to Pine Flat gravity dam as case-study. The tallest non-over-flow monolith of the dam is selected, as shown in Figure 2(a), and numerically analyzed along with a portion of the full reservoir and the rock foundation using the finite element method. All components are modeled with eight-node continuum elements as illustrated in Figure 2(b). In addition, to assess the effects of foundation inhomogeneity on the seismic performance of the dam, three distinct rock regions are considered within the foundation as shown in Figure 2. This illustrative configuration has been selected based on primary analyses showing its high influence on the dam seismic performance. No joint or fault is considered between the rock regions. The inertia, flexibility, and damping of the foundation are taken into account. The radiation damping is modeled using infinite elements at the bottom and lateral sides of the foundation as shown in Figure 2(b) to avoid reflection of seismic waves back to the dam. The reservoir is assumed to be full, and linear elastic materials are used to model the behavior of the concrete, water and foundation rocks. The water's density and bulk modulus are 1000kg/m3 and 2.07GPa, respectively.



(a) (b)

Figure 2. (a) Pine Flat dam with full reservoir and illustrative inhomogeneous foundation, (b) finite element mesh of the dam-reservoir-inhomogeneous foundation system.

There is a single interface in the contact between the dam and the foundation. This base joint which is shown in Figure 2(b) has no tensile strength but it can mobilize shear strength up to some extent. The sliding resistance is defined through the Coulomb model as a function of the friction coefficient μ [6]. The cohesion is neglected and no uplift pressure is considered in the analysis. Firstly, models are statically loaded and then they would be dynamically analyzed under the horizontal component of the deconvolved earthquake ground motions. A typical Rayleigh damping of 5% is used for the dam and the foundation.

3.2. PERFORMANCE FUNCTION

Multiple limit-states, related to structural failure modes, can be of interest for concrete gravity dams. As it was stated, the main potential seismic failure modes of gravity dams are tensile cracking, and movement along the prescribed joint at the dam-foundation interface. Hence, in this study, three different performance functions are defined. The first one is tensile overstressing of the dam body which would result in tensile cracking which is defined through subtracting the envelope maximum (tensile) principal stress within the dam body. The second and the third performance functions are related to the dam-foundation interface and are, respectively, sliding along the base joint, and opening of the base joint in its upstream end adjacent to the reservoir.

3.3. CONTROLLING EARTHQUAKES

For seismic analysis of gravity dams, the USACE guidelines [7] suggest return periods of 144-year, 950-year and 10’000-year for OBE, MDE and MCE records in a common service life of 100 years. Probabilistic seismic hazard analysis of the Pine Flat dam site shows that the peak horizontal accelerations to be expected at the site are 0.18g, 0.27g, and 0.45g corresponding to return periods for the OBE, MDE, and MCE ground motions, respectively [8]. They are proportional to hazard levels of 50%, 10% and 1% in a service life of 100 years, respectively. The Kern County earthquake of 1952 recorded at Taft Lincoln School Tunnel is selected as the free-field ground acceleration. It is scaled into three increasing PGA levels as stated above corresponding to the OBE, MDE and MCE records of the site. Their annual exceedance probability is 0.69%, 0.11% and 0.01%, respectively. The response spectra of the scaled records are shown in Figure 3.

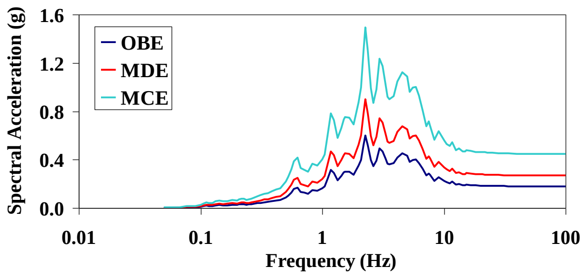


Figure 3. Response spectra of the selected earthquake ground motions

3.4. RANDOM VARIABLES

The random variables chosen are listed in Table 1 with their associated probability distribution function, mean, and coefficient of variation. The related domains are shown in Figure 2(a). As limited material investigations are available for the Pine Flat dam, most probability distributions are defined from empirical data of similar dams. The uncertainty in modeling parameters is mainly considered to be epistemic because of this lack of knowledge. The elastic moduli of the rock regions are related to the dam’s one using the defined α ratios. The random variables are all assumed to be uncorrelated.

3.5. MONTE-CARLO WITH LHS

For Monte-Carlo simulation, statistically significant samples of the dam-foundation coupled system are derived using Latin Hypercube sampling (LHS). There is no predefined sample size N to achieve a certain confidence level, however, some formulas have been presented for various applications. One of the simplest formulas is [9]:

$$\lambda < 1 - e^{-N.P_f} \tag{3}$$

where λ is the confidence level, and P_f is the exceedance probability of limit-state. Assuming a confidence level of λ = 98% and $P_f = 10^{-3}$, which is a reasonable value in dam engineering [10], about 4000 samples are required based on this convenient formula. To assess the number of samples, the LHS method is employed to obtain three different sets with 1000, 2000, and 4000 dam-reservoir-foundation samples by sampling the modeling parameters in Table 1. Each sample is then analyzed under the selected scaled ground motions. As there are three different earthquake records scaled to the given seismic intensities, total number of simulations is 42'000.

Table 1. Selected random variables; related domains and parameters are shown in Figure 2. (a)

Domain	Random variable	Unit	Distribution function	Mean	Coefficient of variation
Dam	Density, ρ_c	kg/m^3	Lognormal	2400	0.1
	Elastic modulus, E_c	GPa	Lognormal	30	0.2
Rock region 1	Density, ρ_{r1}	kg/m^3	Lognormal	2600	0.1
	Elastic modulus ratio, $\alpha_1 = E_{r1} / E_c$	---	Uniform	0.875	0.4
Rock region 2	Density, ρ_{r2}	kg/m^3	Lognormal	2600	0.1
	Elastic modulus ratio, $\alpha_2 = E_{r2} / E_c$	---	Uniform	0.875	0.4
Rock region 3	Density, ρ_{r3}	kg/m^3	Lognormal	2600	0.1
	Elastic modulus ratio, $\alpha_3 = E_{r3} / E_c$	---	Uniform	0.875	0.4
Base joint	Fiction coefficient, μ	---	Normal	1	0.2

4. RESULTS AND INTERPRETATION

The histograms of maximum seismic responses of the three sample sets under the OBE, MDE, and MCE ground motions are illustrated in Figure 4. Also shown is the best distribution fit using one of normal, lognormal, beta, and logistic fits. The distributions' mean and standard deviation, for each histogram is presented in Table 2. Regardless of the number of samples, the same distribution fit with very similar mean and standard deviation is obtained. Therefore, only one fit for the 1000-sample set is shown in Figure 4. As it is expected, when the applied earthquake becomes stronger, higher mean responses are generally captured. The mean responses are about 0.2, 0.7, and 4.8MPa for tensile stress under the OBE, MDE, and MCE, respectively. However, even up to 8MPa tensile stress may be observed under the MCE. The corresponding standard deviations also increase by increasing the earthquake intensity. So it seems that the tensile stresses spread in a wider range. But, the coefficient of variation of the tensile stress is about 1.10, 0.38, and 0.23 under the OBE, MDE, and MCE, respectively. So by increasing the earthquake intensity the lower dispersion of distribution is obtained.

About the base joint opening and sliding, the mean responses increase by increasing the earthquake intensity, but the standard deviation is the most under the MDE, then MCE, and then OBE. Moreover, same as tensile stress, more earthquake intensity causes lower coefficient of variation. Rather high values of the coefficient of variation show that large uncertainties are controlling the results, no matter what number of samples is being used. Even with low coefficient of variation, the base sliding more than 60cm may occur under the MCE. Based on the results obtained, it is important to increase the amount of information about the sources of uncertainty affecting the seismic responses.

From the histograms, it is possible to compute the exceedance probability of the limit-states Pf, for varying threshold values using equation (3). The varying threshold values describe various damage levels. The obtained exceedance probability (EP) curves are plotted in Figure 5 for the defined performance functions. As it is observed, the EP curves are very similar regardless of the number of samples used. As the earthquake record becomes more intensive, the related EP curve will be shifted to the right side which shows higher exceedance probabilities. For example, considering threshold value of 2.25MPa for the overstressing performance function, the exceedance probabilities would be 2%, 37% and 93% for the OBE, MDE and MCE, respectively. This threshold is a common value of the dynamic tensile strength of dam concrete [11], so exceeding this value means tensile cracking of the dam body, however it is not coincident with the total failure of the dam.

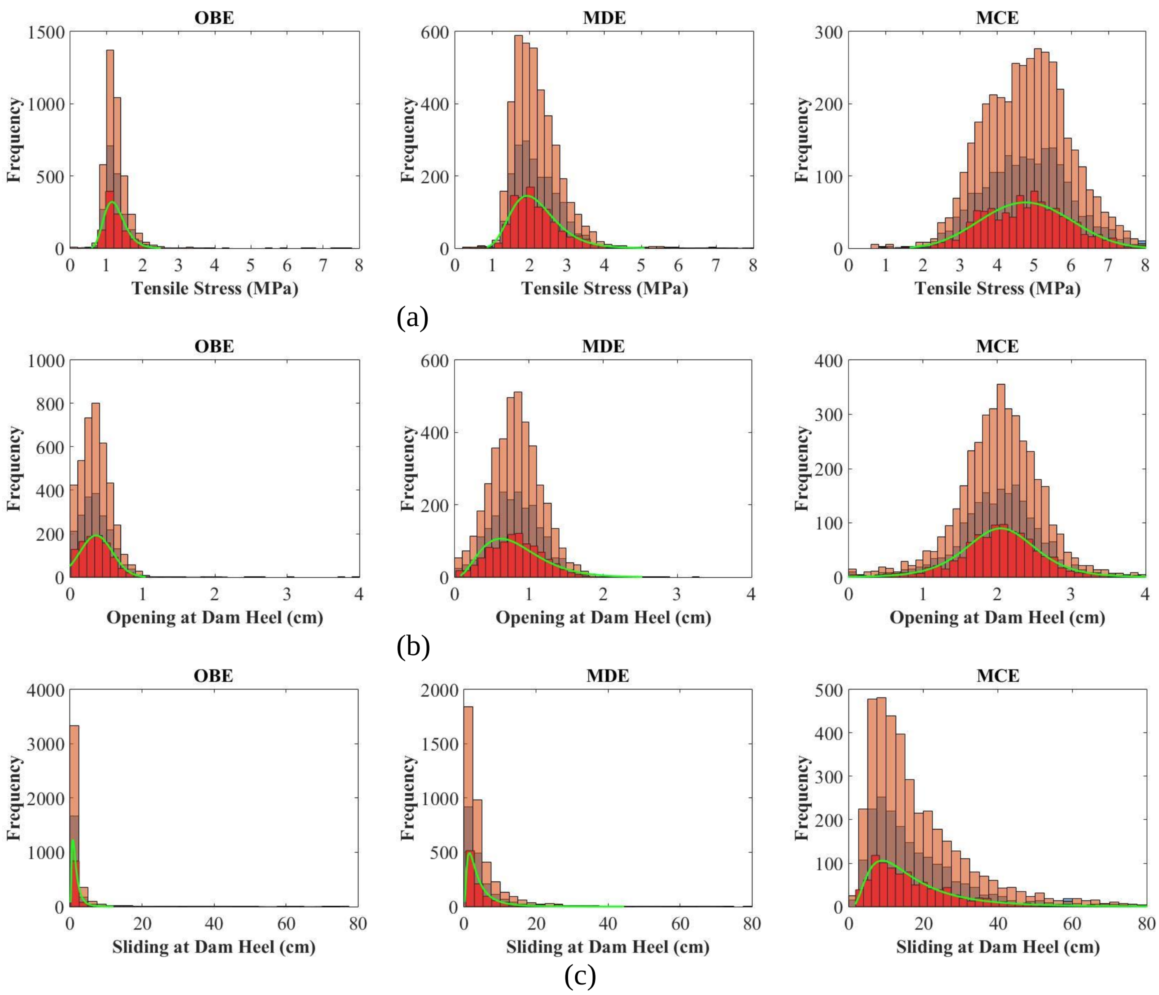


Figure 4. Histograms of seismic analysis output for the three sets of samples under different ground excitations. (a) maximum tensile stress, (b) base joint opening, and (c) base joint sliding. The results of 1000, 2000, and 4000-sample sets are shown in red, brown, and light brown, respectively.

Table 2. Mean and standard deviation of distribution fits on the output histograms

Seismic output	Ground motion	Distribution	Number of samples					
			1000		2000		4000	
			Mean	STDEV	Mean	STDEV	Mean	STDEV
Tensile stress	OBE	lognormal	0.197	0.232	0.204	0.221	0.197	0.217
	MDE	lognormal	0.733	0.276	0.739	0.266	0.734	0.263
	MCE	beta	4.787	1.149	4.808	1.152	4.802	1.132
Base joint opening	OBE	normal	0.349	0.204	0.349	0.201	0.349	0.198
	MDE	beta	0.820	0.346	0.818	0.333	0.820	0.334
	MCE	logistic	2.032	0.316	2.020	0.306	2.001	0.330
Base joint sliding	OBE	lognormal	0.261	0.676	0.288	0.701	0.284	0.702
	MDE	lognormal	1.130	0.850	1.139	0.861	1.140	0.868
	MCE	lognormal	2.674	0.710	2.683	0.718	2.648	0.787

The base joint opening is observed even for low seismic intensities. Because the opening is measured at the upstream end of the base joint adjacent to the reservoir, it may result in water to penetrate inside the base joint that endangers the dam stability. The probability of maximum opening to exceed 1cm is 0%, 29% and 97% under the OBE, MDE and MCE, respectively. Minor, moderate and severe damage will be imposed to the dam's drain system for incipient, 2.5cm and 5cm base sliding, respectively [12]. The moderate and severe damage probabilities are 16% and 8% under the OBE, respectively, while they are 53% and 29% under the MDE, and more than 90% under the MCE. A sliding displacement of 15cm would cause unacceptable differential movements with the adjacent monoliths and could, eventually, cause loss of reservoir control [2]. The probability of exceeding from this high sliding is 3%, 8% and 50% for the OBE, MDE and MCE, respectively.

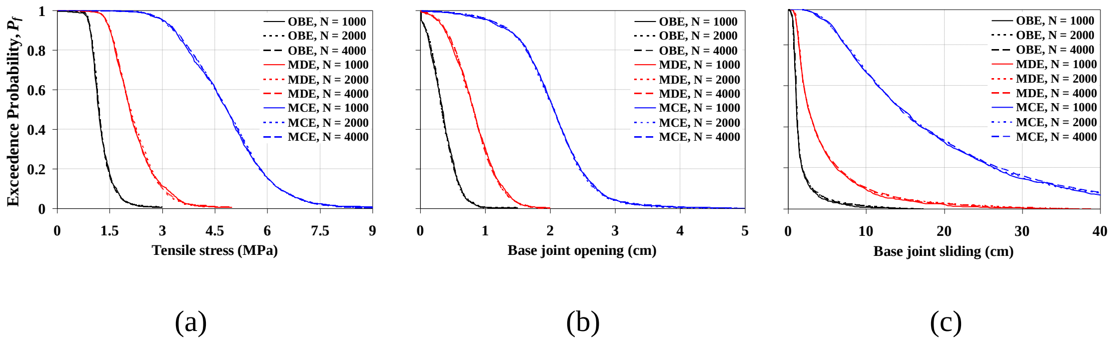


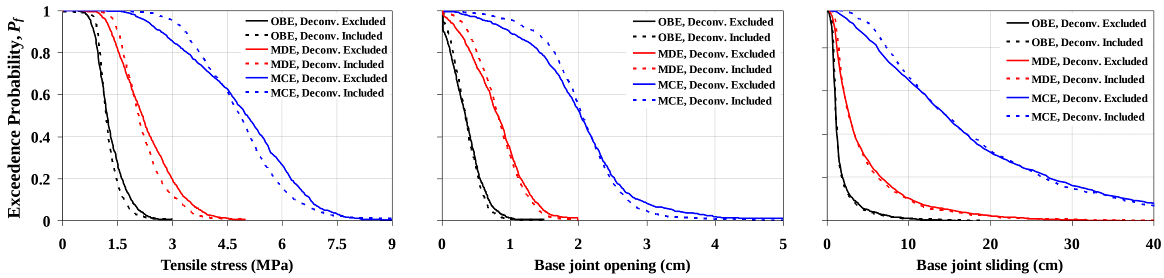
Figure 5. EP curves of the three sets of samples under increasing ground excitation intensities for: (a) overstressing, (b) base joint opening, and (c) base joint sliding performance functions.

All in all, considering two seismic failure modes of tensile overstressing and movement along joints, the dam will undergo severe damage level with the probability more than 90% under the MCE ground motion, while this probability is below 5% for the OBE. Approaching coincidently to two failure modes will probably cause the total failure of the dam under intense earthquake ground shakings; however, the dam will safely survive light earthquakes in order of OBE. It should be noted that the computed conditional probabilities do not provide much information on dam total safety because it has to be multiplied by the probability of the seismic loadings [5]. In this way, the total annualized probability of failure is computed by summing the products of the probability of the seismic event by the conditional probability of failure for all potential failure modes.

5. SENSITIVITY TO EARTHQUAKE DECONVOLUTION PROCESS

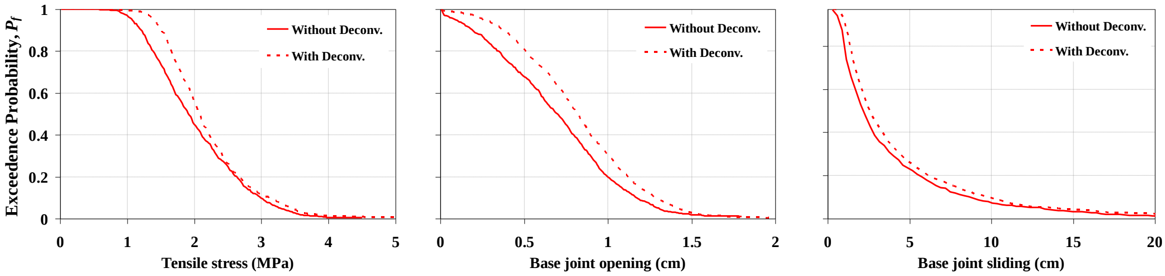
To investigate the effects of the deconvolution process, the 1000-sample set is re-analyzed under the OBE, MDE and MCE deconvolved using the foundation with mean values for the random variables of the rock regions. The deconvolution process is done only once for each earthquake, and it is not repeated for every sample. So the same ground motion is applied to all samples. The new computed EP curves are compared with obtained curves in last section in Figure 6. The difference between the EP curves increases by increasing the earthquake intensity. So the deconvolution process is more important for larger earthquakes. However, the maximum difference between the estimated P_f values in all threshold levels is up to 12% for the defined performance functions. Excluding the deconvolution process for each sample flattens the EP curve, and the mean values are shifted into lower values. It shows that the obtained results are more sensitive when the deconvolution process is considered for each sample.

In the next step, the 1000-sample set is re-analyzed under the MDE which is applied without any deconvolution in the free-field condition to the model base. The resulted EP curves are illustrated in Figure 7 against the EP curves obtained considering the deconvolution of the MDE. Ignoring the deconvolution and applying the ground motion in the free-field condition totally underestimates the exceedance probabilities in entire threshold range for all failure modes. The difference is more for lower threshold values. It is expected that magnifying the earthquake intensity would increase this underestimation. Hence, the earthquake records should be deconvolved when analyzing dam-foundation systems considering inertia and inhomogeneity of the rock.



(a) (b) (c)

Figure 6. Effects of the earthquake deconvolution process on the EP curves for (a) overstressing, (b) base joint opening, and (c) base joint sliding performance functions



(a) (b) (c)

Figure 7. Comparison of the EP curves of the 1000-sample set under the MDE with and without deconvolution: (a) overstressing, (b) base joint opening, and (c) base joint sliding

6. CONCLUSIONS

In this study, a methodology was proposed to evaluate the seismic reliability of gravity dam-reservoir-foundation coupled systems. The uncertainties associated with modeling parameters were incorporated in nonlinear time history analysis to realistically quantify their effects on the seismic performance of the system. The system was analyzed under the OBE, MDE, and MCE records of the dam site. The deconvolved-base-rock input model was utilized as earthquake input mechanism. The same distribution fit with very similar parameters and approximately the same exceedance probability curves were obtained for seismic output results regardless of the number of samples. So in this type of analysis, the number of simulations can be reduced in order of four, from 4000 to 1000, with the same estimation of probability. Higher mean responses were obtained under larger earthquake, but the coefficient of variation decreased by increasing the earthquake intensity. However, high values of the coefficient of variation showed that large uncertainties are controlling the results. It was shown that considering two seismic failure modes of tensile overstressing and movement along joints, the dam will undergo severe damage level with the probability more than 90% under the MCE, while this probability is below 5% for the OBE. Approaching coincidently to two failure modes will probably cause the total failure of the dam under intense earthquake ground shakings; however, the dam will safely survive light earthquakes in order of OBE.

About the deconvolution process, it was found that it is more important for larger earthquakes. Excluding the deconvolution process for each sample flattened the EP curve, and the mean values were shifted into lower values. The obtained results were more sensitive when the deconvolution process was considered for each sample. Ignoring the deconvolution and applying the ground motion in the free-field condition totally underestimated the exceedance probabilities for all failure modes. Hence, the earthquake records should be deconvolved when analyzing dam-foundation systems considering inertia and inhomogeneity of the rock.

7. REFERENCES

1. ICOLD (2001), Design features of dams to resist seismic ground motion: Guidelines and case studies. Bulletin No. 120, Paris.
2. Bernier C, Padgett J.E, Proulx J, and Paultre P. (2016), "*Seismic Fragility of Concrete Gravity Dams with Spatial Variation of Angle of Friction*", Journal of Structural Engineering, **142** (5).
3. Alembagheri M, and Seyedkazemi M. (2015). "*Seismic performance sensitivity and uncertainty analysis of gravity dams*", Earthquake Engineering and Structural Dynamics, **44** (1), pp 41-58.
4. Liel A.B, Haselton C.B, Deierlein G.G, and Baker J.W. (2009), "*Incorporating modeling uncertainties in the assessment of seismic collapse risk of buildings*", Structural Safety, **31** (2), pp 197-211.
5. Altarejos-García L, Escuder-Bueno I, Serrano-Lombillo A, and de Membrillera-Ortuño M.G, (2012), "Methodology for estimating the probability of failure by sliding in concrete gravity dams in the context of risk analysis", Structural Safety, **36-37**, pp 1-13.
6. Alembagheri M, and Ghaemian M. (2013), "*Incremental dynamic analysis of concrete gravity dams including base and lift joints*", Earthquake Engineering and Engineering Vibration, **12** (1), pp 119-134.
7. USACE (2003), Time-history dynamic analysis of concrete hydraulic structures. EM-1110-2-6051, Washington, DC.
8. USBR (2003), Upper San Joaquin River Basin Storage Investigation, Raise Pine Flat Dam. Surface Storage Option Technical Appendix to the Phase 1 Investigation Report. In Coordination with: The California Bay-Delta Authority.
9. Broding W.C, Diederich F.W, and Parker P.S. (1964), "*Structural optimization and design based on a reliability design criterion*", Journal of Spacecraft and Rockets, **1** (1), pp 56-61.
10. Chauhan S.S, and Bowles D.S. (2003), "*Dam safety risk assessment with uncertainty analysis*", Proceeding of Australian Committee on Large Dams Risks Workshop, Australia.
11. 11.Harris D.W, Mohorovic C.E, and Dolen T.P. (2000), "*Dynamic properties of mass concrete obtained from dam cores*", Materials Journal, **97** (3), pp 290-296.
12. 12.Fenves G.L, and Chavez J.W. (1996), "*Evaluation of earthquake induced sliding in gravity dams*", 11th World Conf. on Earthquake Eng., International Association for Earthquake Engineering, Tokyo.