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SEISMIC FRAGILITY ASSESSMENT OF CONCRETE GRAVITY DAMS USING NONLINEAR DYNAMIC ANALYSIS WITH MASS FOUNDATION

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ABSTRACT

Maintaining of concrete gravity dams in good conditions as one of the important infrastructures is significant concern for dam owners. These dams should able to continue their function after a disaster such as earthquake. But most of them are aged dams with some located near faults. Indeed, there are some concerns regarding the performance of these dams under the effect of seismic loads. In recent years some criteria about linear performance of these dams have been developed. But the same cannot be said about nonlinear behavior of these dams, which seem to lack well-developed criteria. In this paper we shall try to illustrate seismic fragility curves for concrete gravity dams by using nonlinear dynamic analysis and a continuum crack propagation model, smeared crack model. For this purpose largest monolith of Pine Flat dam has been used. The fragility curves show the dam is very vulnerable when an earthquake strikes it with the peak ground motion more than 0.19g.

Introduction

In recent years, the growing knowledge of seismic hazard and improvement in designing techniques of dams have caused an increased awareness and concerns regarding the performance and reliability of aged concrete gravity dams under the effect of seismic loads. As a result, dam engineers seek to develop the most reliable methods of investigating safety issues affecting concrete gravity dams before beginning a rehabilitation process for these dams.

In the last, two methods have been used for illustrating fragility curves for concrete gravity dams. One of them is based on ATC-13 report and damage probabilities matrices. This method was used by Lin and Adams for illustrating fragility curves of Canadian hydropower components (Lin, 2007). Another is based on demand capacity ratio (DCR) and cumulative inelastic duration concept for linear dynamic analysis procedure that it has been introduced based on FEMA 356 concepts and developed by Yousef Ghanat and USACE (Tekie 2003).

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There are always some uncertainties concerning the behavior of concrete dams under seismic loads. Some uncertainties are related to issues such as the material properties of dam and foundation, the differences between shape and height of dams and so on. Others go back to insufficiency of our knowledge regarding seismic hazards and earthquake characteristic. Because of these factors, the most promising method for analyzing concrete gravity dams seems to be a combination of nonlinear dynamic time history analysis topped by applying our knowledge of probabilities that could be inferred from seismic fragility curves.

Because of complex nature of dam-reservoir-foundation system, we had to illustrate seismic fragility curves of concrete gravity dams only by concentrating on uncertainties of earthquake inputs based on energy error of nonlinear analysis and by using length and areas of damaged elements. Data of Pine Flat dam was used for this purpose, Figure 1. Pine Flat dam was built near Fresno, California in 1954. It was assumed that data for the material properties could be accurate based on in site field tests.

Fragility analysis

Probabilistic Safety Assessment

For the purpose of dam safety, some limit states should be introduced in order to investigate performance levels of a dam. For example in frames, this limit state could be drift of stories, rotation of nodes, etc. For obtaining seismic fragility curves, the probability of exceeding to the structural limit state should be considered. In the equation (1), fragility is the probability of engineering demand parameter (EDP) that exceeds the structural limit state (LS) at the defined PGA.

$$Fragility = P[EDP > LS \mid PGA] \tag{1}$$

This probability could be presented by lognormal distribution or some of the others distributions:

$$Fragility = P[EDP > LS \mid PGA] = 1 - P[EDP < LS \mid PGA] = 1 - \Phi\left[\frac{\ln(LS) - \mu}{\sigma}\right] \tag{2}$$

In the equation above Φ is standard normal probability integral, μ is logarithmic mean of data and σ is logarithmic standard deviation.

Structural modeling of dam behavior

Smeared Crack Model

In this research, smeared crack model was used for the purpose of analysis. The constitutive model for smeared fracture analysis defining (i) the pre-softening material behavior, (ii) the criterion for softening initiation, (iii) the fracture energy conservation, and (iv) the softening, closing and reopening of cracks (Ghaemian 1999).

Energy Balance Error

The energy balance error is computed as equation (3). In this equation, E^K is absolute kinetic energy. E^D is viscous damping energy. E^R is nonlinear resorting work. The work of preseismic applied force is E^P, the absolute seismic input energy is E^Q and the work done by hydrodynamic pressure is E^H.

$$Energy\ \ balance\, error = \frac{(E^P + E^Q + E^H) - (E^K + E^D + E^R)}{(E^Q + E^H)} \times 100 \tag{3}$$

In the analysis, the results of the fracture response are presented for the time before the five percent energy balance error is reached. The error in the energy balance represents an excessive amount of damage when numerical damping is introduced.

Finite element model of dam

Description of Model

With regard to the above-mentioned dam, an educational computer code, NSAG-DRI (Ghaemian 2008) was used to carry out the nonlinear analysis of the tallest monolith of Pine Flat Dam. This code was capable of carrying out coupled equation of dam-reservoir system. For modeling of tensile stress on dam's body the smeared crack model was applied. The 4-node, quadrilateral, isoparametric finite element model of tallest monolith in plane stress has been illustrated in Figure 2. The model had 5664 nodes (3768 nodes at foundation) and 5512 elements.

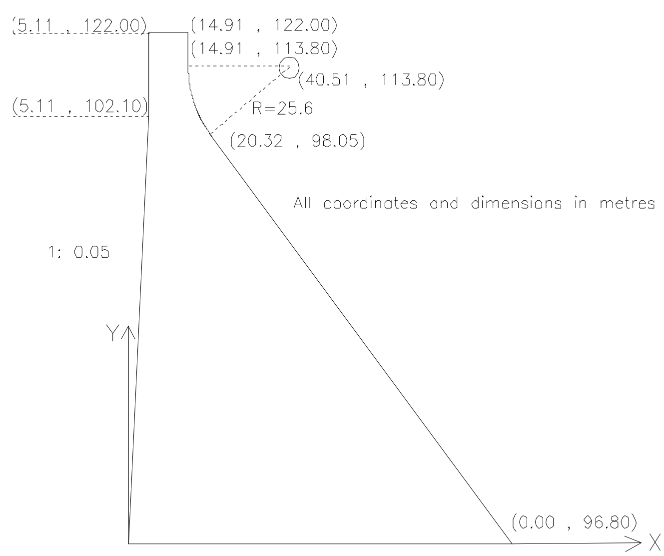


Figure 1. Dimensions of the tallest monolith of Pine Flat dam.

The model, as shown in Figure 3, is a flexible massed foundation with Lysmer boundary conditions at the base and sides (USACE 2003,2007). Earthquake input applied at the base of dam's body (dam-foundation interface). It was assumed that foundation has linear behavior. The length and depth of the foundation were 348m and 126m, respectively. It means the foundation has been modeled about one height in sides and depth. At the bottom of foundation all nodes have been restrained to vertical motion for preventing subsiding of foundation under the effect of dam and foundation weight. Even though that modeling would be caused that waves can be back to system but using dashpots in some nodes in the vertical direction at the bottom of foundation will not reduce this effect because the restriction in the motion of the adjacent nodes. So it was assumed all nodes at the bottom of foundation are restricted in the vertical direction. In this case the model can be considered more conservative.

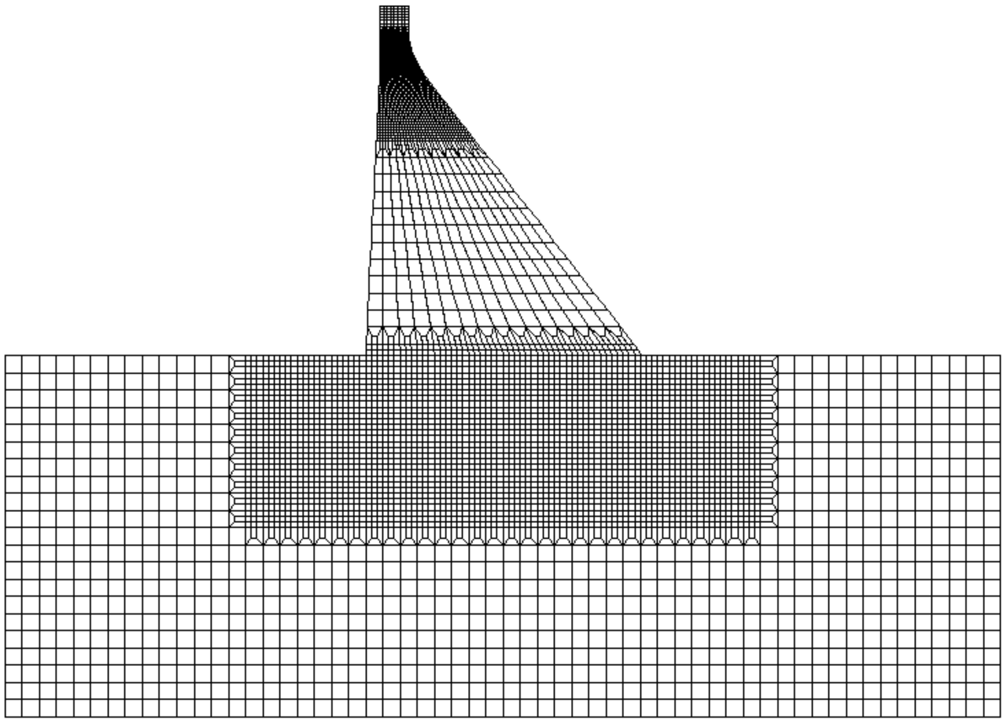


Figure 2. Finite element model of the tallest monolith of Pine Flat Dam.

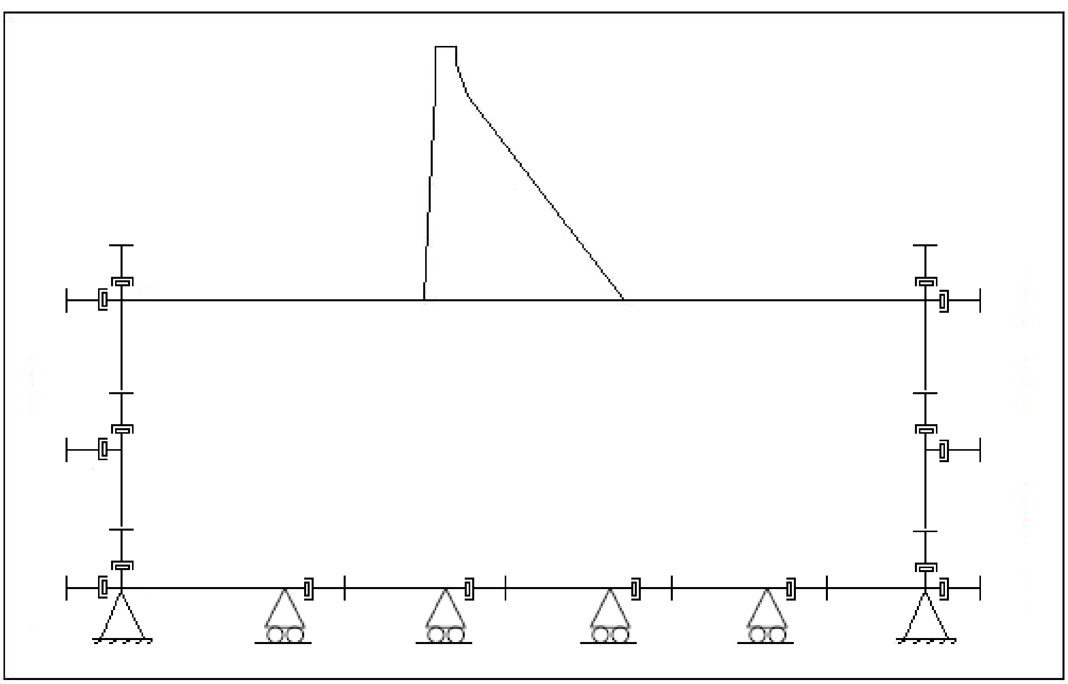


Figure 3. Boundary conditions in massed foundation model.

It was assumed normal water level is 116.88 m. Sharan boundary conditions were used to truncate the model at the far end of the reservoir (Sharan 1985). Material properties have been shown in Table 1 (Ghaemian 1999).

Table 1. Summary of selected parameters.

Concrete Material Properties	
Unit Weight	2483 Kg/m^3
Modulus of Elasticity	27.58 GPa
Static Tensile Strength	2.4 MPa
Poisson’s Ratio	0.20
Fracture Energy (G_f)	300 N/m
Rock Material Properties	
Unit Weight	2643 Kg/m^3
Modulus of Elasticity	22.4 GPa
Poisson’s Ratio	0.33
Wave Reflection Coefficient	α=0.82

A five percent Rayleigh damping ratio was elected for the first mode. Table 2 shows natural periods of the structure with massed foundation model.

Table 2. Natural Periods (Sec).

Mode	Massed Foundation
1	0.8076
2	0.3636
3	0.2453
4	0.2024
5	0.1988
6	0.1713
7	0.1702
8	0.1478
9	0.1386
10	0.1215

Earthquake Ground Motion

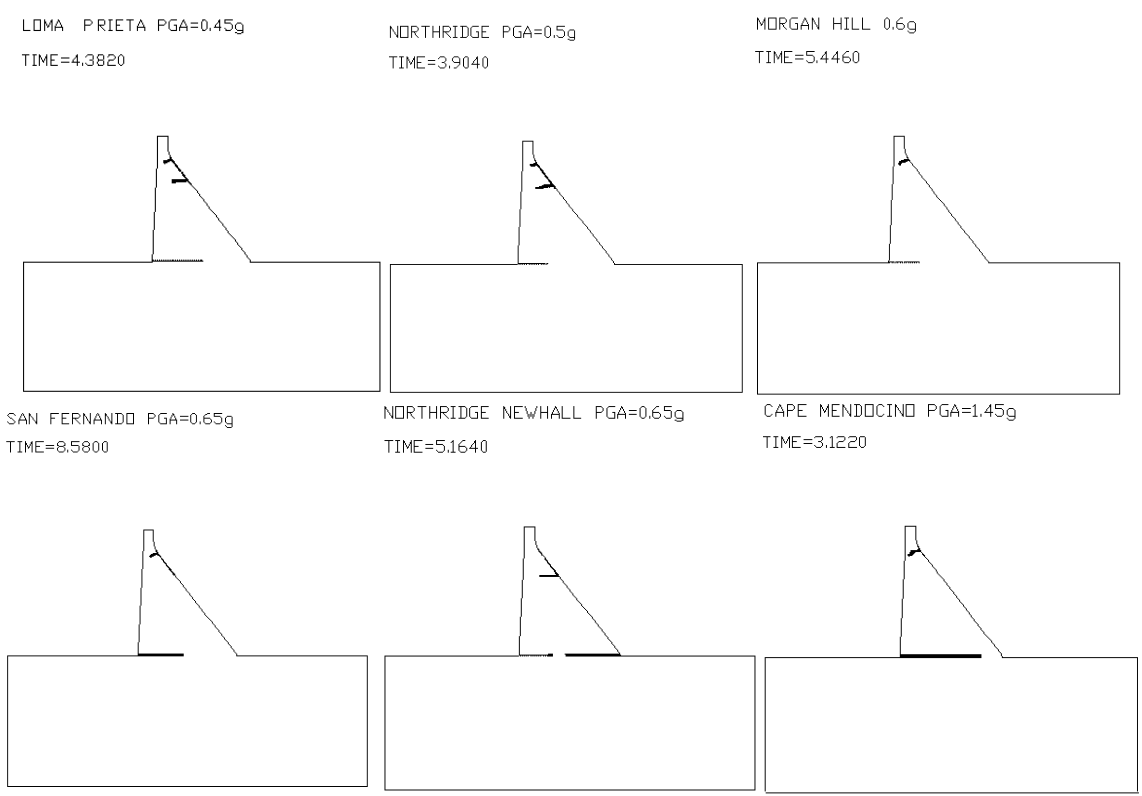
For evaluation of the earthquake damage, the dam was assumed to be located in the near field of the earthquake event. Twelve selected natural acceleration time histories have been shown in Table 3. All these records have been scaled from 0.1g to 0.7g.

Table 3. Ground Motion Records Used for Analysis (Peer Berkeley Database 2009).

Year	Earthquake	Record	Magnitude
1966	PARKFIELD	Cholame	6.19
1971	SAN FERNANDO	Pacoima Dam, DS record	6.6
1974	HOLLISTER	Gilory Array	5.14
1980	LIVERMORE	Morgan Terr Park	5.42
1980	MAMMOTH LAKES	Convict Creek	5.69
1984	MORGAN HILL	Coyote Lake Dam	6.2
1986	N. PALM SPRING	Whitewater Trout Farm	6.06
1987	WHITTIER	San Gabriel	5.99
1989	LOMA PRIETA	Gilroy - Gavilan Coll	6.9
1992	CAPE MENDOCINO	Cape Mendocino	7.1
1994	NORTHRIDGE	Pacoima Dam, DS record	6.7
1994	NORTHRIDGE	Newhall, West Pico Canyon	6.7

Result from nonlinear dynamic analysis and introducing criteria

Different horizontal earthquakes were applied to dam's models to find out when the dam-reservoir-foundation system reaches to 5% energy balance error based on nonlinear dynamic time history analysis. The PGA that causes 5% energy balance error was determined to the accuracy of 0.05g (Figure 4).



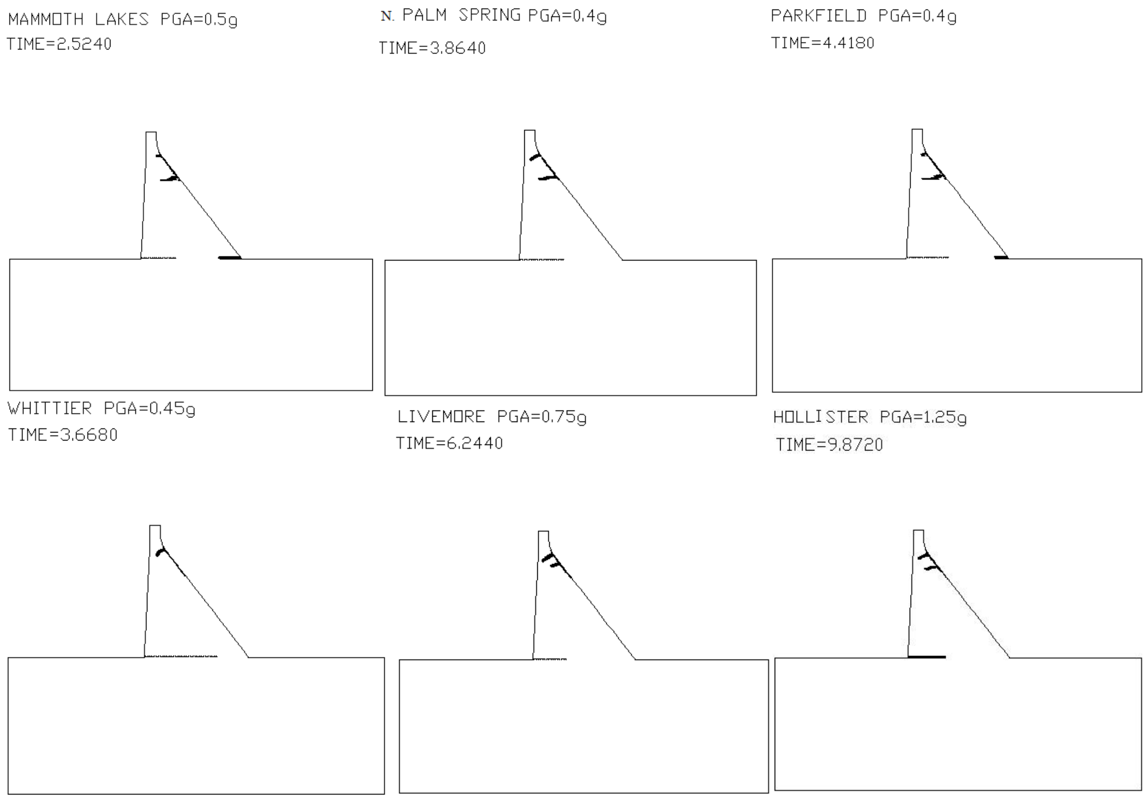


Figure 4. Propagation of cracks at the body of dam.

After determining the maximum PGA that the dam-reservoir-foundation can endure in every horizontal earthquakes, now we have to define limit states. For adopting a factor to determine limit state (LS), the factor should increase as PGA increases. As a result, tensile stress on dams' body cannot be a factor due to the fact that based on nonlinear dynamic analysis, tensile stress in cracked elements is equal to zero. Other factors such as deformation of crest relative to heel are somewhat inaccurate because this rotation is due to crack at the neck of dam. In this research, two factors are considered for requiring criteria that enable us to develop fragility curves for concrete gravity dams.

First limit state (LS), is based on length of crack at the base. Outputs have been organized in Table 4. It is considered that, when the dam-reservoir-foundation system reaches to 5% energy balance error, the lowest length of crack at the toe and heel of the dam is 30m. So, for a performance level which can guarantee that the dam's structure is safe and can continue its operation, the length of crack is chosen at 75% of the lowest length of crack or 22.5 m (*SF=1.33*). It means that it would be about 0.23 of dam's base length.

Table 4. Length of crack at the toe and heel of dam for massed foundation.

Earthquake record(s)	PGA	Length of crack at the toe and heel (m)	Areas of crack elements at the toe and heel of dam (m^2)
LOMA PRIETA	0.45g	50	100
MAMMOTH LAKES	0.50g	34	68
N. PALM SPRING	0.40g	42	84
WHITTIER	0.45g	68	136
NORTHRIDGE NEWHAL	0.65g	84	168
PARKFIELD	0.40g	40	80
MORGAN HILL	0.60g	30	60
SAN FERNANDO	0.65g	44	88
CAPE MENDOCINO	1.45g	76	152
NORTHRIDGE	0.50g	30	60
LIVEMORE	0.75g	64	128
HOLLISTER	1.25g	36	72

Second structural limit state is introduced based on total areas of cracked elements in the body of dam. Outputs are organized in Table 5. The lowest areas of cracked elements for massed foundation happened for Morgan Hill earthquake when the PGA is 0.6g. For a safe performance level 75% of this area is chosen (*SF=1.33*). It is about 75 m^2 or 0.0130 of the tallest monolith section.

Table 5. Areas of crack elements at the body of dam for massed foundation

Earthquake record(s)	PGA	Areas of cracked elements at the toe and heel of dam (m^2)	Areas of cracked elements at the neck of dam(m^2)	Total areas of crack elements(m^2)
LOMA PRIETA	0.45g	100	82.09	182.09
MAMMOTH LAKES	0.50g	68	75.93	143.93
N. PALM SPRING	0.40g	84	95.77	179.77
WHITTIER	0.45g	136	58.15	194.14
NORTHRIDGE NEWHALL	0.65g	168	30.1	198.1
PARKFIELD	0.40g	80	83.46	160.46
MORGAN HILL	0.60g	60	39.68	99.68
SAN FERNANDO	0.65g	88	49.94	137.94
CAPE MENDOCINO	1.45g	76	49.94	125.94
NORTHRIDGE	0.50g	60	78.67	138.67
LIVEMORE	0.75g	64	95.09	159.09
HOLLISTER	1.25g	72	93.03	165.03

Seismic fragility curves

Performing nonlinear dynamic analysis, the crack length and the areas of cracked elements are determined for each 0.1g increased in PGA for the model. By using these data, lognormal distribution and defined limit states described in the previous section, seismic fragility curves have been illustrated in Figure 5.

Based on a study in 2003 for raising the height of Pine flat dam, seismic hazard potential at the site is low. Probabilistic seismic hazard analysis shows that the peak horizontal accelerations to be expected at the site is 0.13g with a 2,500-year return period, 0.17g with a 5,000-year return period, and 0.23g with a 10,000-year return period (California Department of Water Resources, 2003). Because two defined limit states are chosen by determining lowest amount of damage which can cause structural unreliability under effect of some powerful near field earthquakes, the safety factor that is prepared by these two limit states should be considered to be quite satisfactory.

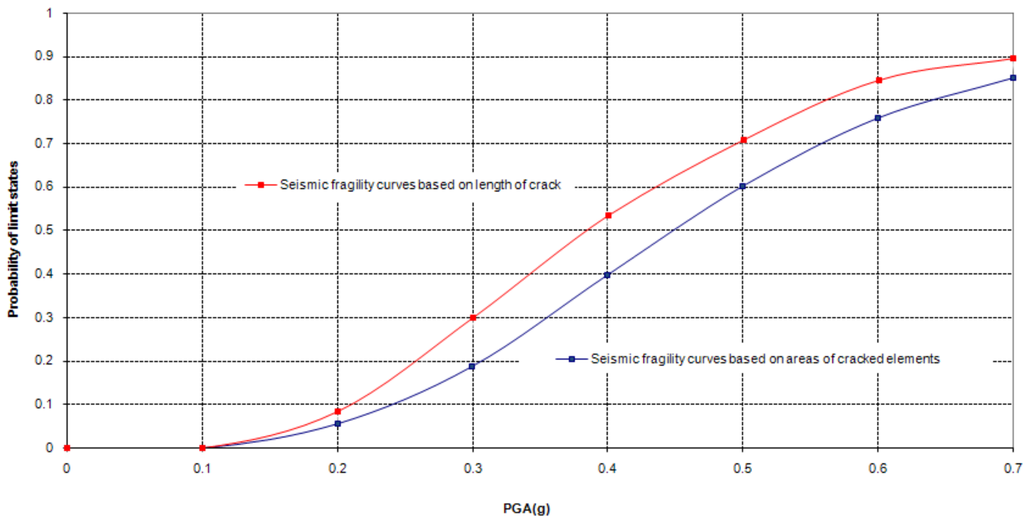


Figure 5. Seismic fragility curves

Conclusions

Seismic fragility curve based on length of crack at the base shows higher probability when it compares with seismic fragility curve based on areas of cracked elements. In this respect, seismic fragility curves based on the areas of cracked elements should be a more realistic approach, especially when it accounts areas of damaged elements at the neck, too.

By considering that 5 percent probability of exceeding more than limit states can guarantee the safety of structure, the structure is safe under strikes of an earthquake with the PGA equals to 0.19g, based on the areas of cracked elements. Such earthquake has a return period more than 5000 years. This dam can perform adequately against an earthquake with the PGA about 0.16g, based on length of cracks criterion. Certainly, applying more earthquake inputs to the model can provide more accurate fragility curves.

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