Seismic Performance of RC Bridge After Series of Aftershocks Following a Major Earthquake

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Research Article

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Abstract

This study aimed to understand the seismic performance of a typical reinforced concrete (RC) bridge considering number of aftershocks following a major earthquake. The primary objectives of this research are to compare the performance of RC bridge comparing mainshock only fragility and mainshock aftershocks sequence fragility and also evaluate the performance after a major earthquake and its aftershocks. Real suite of mainshock aftershock sequences were applied to perform Incremental dynamic analysis (IDA) in a nonlinear model of typical RC bridge in Nepal using Opensees and damage classification was done using Park-Ang damage index. Fragility curves and fragility surfaces were constructed using the probabilistic seismic demand model (PSDM) form IDA analysis. The results shows that there is underestimation of bridge fragility using only mainshock data for computing fragility curves. Also, the expected seismic performance of the bridge after a major earthquake and its aftershocks were computed using fragility surface with the concept of multi-hazard analysis.

1. Introduction

A major earthquake brings series of aftershocks of significant magnitude around the area of the main shock for a long period of time, especially in the seismically active places such as Nepal. These aftershocks are believed to be the result of increase in stresses around the region of mainshock hypocenter (Shcherbakov, Turcotte, and Rundle 2005) which further causes rupture and resettling of the disturbed rock masses causing many aftershocks. The M 7.8 Gorkha earthquake (2015) was followed by 553 aftershocks of magnitude more than M 4 within the first 45 days and 5 aftershocks with magnitude above M 6 which included earthquakes of M 6.3 around 34 minutes after the main shock and M 7.3 almost 17 days later (Adhikari et al. 2015). Similarly, In the Tohoku earthquake (2011), magnitude M 7.5 and M 7.7 aftershocks occurred less than 4 hours after the magnitude M 9 main shock, with other 3 aftershocks that were M 7 or higher, 82 aftershocks M 6 or higher and 506 aftershocks M 5 or higher within first three months after the mainshock (Kazama and Noda 2012). In 1985 Chile earthquake a foreshock M 5.2 was followed by the mainshock of M 7.8, which was then followed by 4 aftershocks of magnitudes greater than M 6, with the last being a M 7.2 earthquake eleven days later (Earthquake spectra 1986). Similarly, in Chichi (1999) earthquake of M 7.6, more than 296 aftershocks of magnitude M 4.3 or higher were recorded within the first hour of mainshock, including M 6.1 and M 6.8 earthquakes (Chang et al. 2007). Therefore, from the past experiences we can say with certainty that a major earthquake is highly likely to be followed by a number of significant aftershocks.

Critical infrastructure such as bridges are crucial for post hazard recovery and rehabilitation. Also, bridge infrastructure are quite expensive and disruption of services can cause more indirect economic and social burden on the concerned authorities. Recent studies has found out that bridges are the weakest link in the road transportation system and damage to it causes indirect economic loss due to prolonged traffic disruptions and direct loss due to repair or rehabilitation (Kilanitis and Sextos 2019). Therefore, a healthy bridge infrastructure system is of paramount importance to a society. The Lombok earthquake of 2018 had several shocks i.e. M 6.4, M 7, M 6.9 due to which cracking of the retaining structures, damage in

expansion joints, damage in bridge bearing and excessive displacement of superstructure were observed (Putra and Riyono 2020). (Mya Nan Aye, Kasai, and Shigeishi 2018) studied the damage that occurred in five different bridges after the Kumamoto earthquakes (2016), which consisted of seven different high intensity earthquakes ranging from M 5.4 to M 7.3 and reported bearing failure due to excessive displacement, abutment/ pier deformations, excessive lateral displacement of girders, excessive settlements of piers as major failure patterns. Therefore, aftershocks makes the already vulnerable structures that are degraded in terms in its strength and stiffness among others due to the strong main shock prone to failure. Similarly, there will most likely be a situation where one needs to make decision on whether a bridge is able to perform its design objectives after a major earthquake considering the possibility of series of major aftershocks to be induced on a deteriorating structure before any kind of intervention can be done.

Hence, in order to access the current situation and be ready for the future as well as incorporating the consequences of the aftershocks while designing for the structures in the first place we need to study the behavior of structures and degradations in it due to the series of aftershocks. Therefore, this research tries to identify the nonlinear behavior of the RCC bridges subjected to the series of aftershocks after main shock and study the fragility of the structures in these case.

2. Literature Review

(Omranian et al. 2022) studied the effect of multiple earthquakes on the horizontally curved RC bridges and concluded the omission of the effects of aftershocks can significantly underestimate the damage and overestimate the performance by almost 50%. (Lee et al. 2020) studied the cumulative damage of RC piers due to multiple earthquakes and concluded there will be significant decrease in flexural and shear stiffness with each earthquake and suggested to properly incorporate multiple earthquake ground motions in order to identify the demand in the bridge piers during seismic performance evaluation. (Ghosh, Padgett, and Sánchez-Silva 2015) proposed a damage accumulation framework for a single column box girder bridge in California region with Park and Ang Damage Index as a damage indicator and concluded that there is significant increase in probability of exceedance of certain damage index considering multiple shocks in case of both mainshock and aftershocks.

(Fakharifar et al. 2015) studied the vulnerability of RC bridges considering post main shock cascading events and concluded the damage caused by the main shock can significantly jeopardize the post main shock resilience due to damage accumulation and geometric nonlinearity. (Li, Song, and Van De Lindt 2014) concluded the structural capacity may reduce significantly when the building is subjected to a high intensity main shock and it is very likely that the structure will collapse after an aftershock of lesser intensity and magnitude. (Di Sarno 2013) worked on effects of multiple earthquakes on inelastic structural response and showed that there is higher demand for RC framed structure considering multiple earthquakes as well as stressed the need of using adequate hysteretic model for the performance evaluation of the building.

(Akbari 2012) conducted a research on the fragility of bridges with irregular pier height and concluded that the smaller piers are more vulnerable in the irregular bridges in terms of pier height and also suggested the vulnerability of the smaller piers decrease by increasing its steel ratio or decreasing the steel ratio of the taller piers. (Tavares, Padgett, and Paultre 2012) assessed the seismic performance of five different bridge classes in Quebec, Canada and concluded that the continuous bridges are more vulnerable than the simply supported bridges and also suggested that the seismic performance is overestimated when the system fragility is governed by the fragilities of column or elastomeric bearing and underestimated when governed by the fragility of abutment walls. (Howard Hwang, Jing Bo Liu 2001) assessed the seismic fragility of highway bridges and recommended the damage parameters in terms of displacement ductility and intensity measure in terms of PGA. (Nielson 2005) studied the seismic fragility of nine different classes of bridge and concluded the bridges with steel girders are the most vulnerable, followed by the bridges with concrete girders and single span bridges of all types respectively.

(Liu, Lu, and Paolacci 2015) studied the multivariate fragility of steel concrete composite bridges using PGV and spectral accelerations as multiple IMs and found that the seismic performance can be more accurately predicted using multivariate analysis. (Wang, Wu, and Liu 2018) considered multiple seismic demand parameters in a RC bridge to evaluate its seismic performance and concluded the multidimensional fragility to be higher than the fragility governed by a single component performance and suggested including different component contribution in the performance evaluation to avoid overestimation.

(Jamie E. Padgett, Bryant G. Nielson 2008) studied the different intensity measures that can be used for performance evaluation of bridges on the basis of their efficacy, practicality and sufficiency and concluded the peak ground acceleration (PGA) to be the optimum among all IMs. (Park and Ang 1985) developed a damage model called Park Ang Damage Index as a function of maximum deformation and energy dissipation due to the cyclic loading to be applicable to measure damage in concrete quantitatively. Park Ang damage index is the most suitable for measuring the performance of a structure in multiple earthquake excitations (Qian 2012).

The nonlinear time history analysis shall be done by continuously increasing the intensity of the earthquake to an extreme level, commonly called Incremental Dynamic analysis (IDA) (Vamvatsikos and Allin Cornell 2002). (Cornell et al. 2000) developed a probabilistic framework in terms of probability of achieving certain performance for given intensity of input by linking the “demand” and “capacity” through a Probabilistic Seismic Demand Model (PSDM) by performing series of nonlinear dynamic analysis.

3. Selection Of Ground Motions

For the nonlinear time history analysis, real set of mainshock aftershock earthquake ground motions were selected from *strongmotion.org*. For each mainshock, two different real aftershock were selected from the same source. A structure subjected to a set of ground motions responds differently depending

upon the frequency, duration, intensity of that ground motion, therefore, it is important to select a proper suite of ground motion corresponding to the site conditions for the proper analysis of structures (Kramer 1996). Thus, proper suite of ground motion representing the proper site conditions for selected bridge were used and they are shown in Table 1.

Table 1
Selected Ground motions

S.N	Earthquake name/Date	Moment magnitude		
		MS	AFS 1	AFS 2
1	Sumatra ,Indonesia (2007)	8.4	7.9	7
2	Tokachi-oki, Japan(2003)	8	7	6.7
3	Gorkha (2015)	7.8	6.3	7.3
4	Northridge (1994)	6.7	5.1	5.3
5	Chile Valparaiso (1985)	7.4	5.9	7.8
6	Imperial Valley, California (1979)	6.5	5	5
7	Japan Tohokhu (2011)	9	7.9	7.7
*MS: Mainshock; AFS1: Aftershock 1; AFS2: Aftershock 2				

The mainshock only comparison shall be done with the same ground motions in Table 1, but taking only mainshock. The above ground motion time history was matched to NBC: 2020 design spectrum for type C soil conditions and then scaled to different intensity levels for Incremental dynamic analysis.

4. Damage Model

There are various damage models for use during seismic performance of RC bridges. One of the most widely used damage model is column drift ratio ((Tavares et al. 2012), (Akbari 2012), (Billah and Alam 2015) etc.). However, due to the reasons further explained below column drift ratio is not suitable for this research involving mainshock aftershock sequences.

In the Fig. 1, the maximum column drift during mainshock of Tohokhu earthquake is 1.15%. Similarly, if we consider aftershocks too the maximum drift during the mainshock aftershock sequence doesn’t increase beyond 1.15%, due to which it seems the application of aftershocks doesn’t have any significance. However, this is not true as aftershocks do add some level of damage to the structure. Similarly, residual displacement at the end of the sequences also may not give the exact state of damage as it is dependent on the spectral characteristics of the aftershocks itself (Qian 2012). Therefore, it is clear that lateral displacement alone cannot properly justify the true damage state of the structure.

Therefore, it is necessary to include the energy dissipated during each earthquake in conjunction with the displacement ductility terms in order to represent true damage state of the structure and for that Park Ang Damage Index is a good choice (Park and Ang 1985). Park Ang damage scale is frequently used damage index to quantify the damage in concrete where the damage is expressed as the function of maximum deformation and repeated cyclic loading. The expression of Park Ang damage index is given by the following Eq. (1).

DI= $\frac{\delta_m}{\delta_u} + \frac{\beta \int dE}{Q_y \delta_u}$ *(1)*

Where,

DI = Damage Index

δ_m = Maximum deformation under given earthquake

δ_u =Ultimate deformation under monotonic loading. Given by; $\delta_u = \delta_y \mathrm{x} \mu$

δ_y =Deformation at start of yielding

μ =Deformation ductility

β = cyclic loading factor taken as 0.05–0.2.

Q_y =calculated yield strength (if the maximum strengthQ_u, is smaller thanQ_y, Q_y

is replaced by Q_u)

$\int dE =$ Total absorbed hysteretic energy

Here, the expression for the total absorbed hysteretic energy added to the deformation parameter adds the much needed accuracy in the damage state of the structure and it is given by Eq. (2).

$$\int dE = \sum_{i=1}^{n} \frac{P_{i+1} + P_i}{2} (u_{i+1} - u_i)$$

2

Where;

P = Force applied/Reaction and u = deformation.

5. Analytical Fragility Curves

Fragility curves provides a quantitative measure to predict the seismic performance of a given structure. These curves suggest the probability of exceedance of a certain damage state for a given level of

Intensity measure. According to (Cornell et al. 2000) the median demand (S_d) due to a certain intensity measure (IM) can be represented by the following Eq. (3).

$$S_d = a\ IM^b \qquad (3)$$

Taking natural logarithmic in the above equation, we get:

$\ln(S_d) = \ln(a) + b \ln(IM)$

Where IM is the seismic intensity measure and a, b are regression coefficients.

The plot between $\ln(S_d)$ and ln (IM) is called the Probabilistic Seismic Demand Model (PSDM).

Similarly, for multivariate analysis the model can be represented by the higher order regression analysis for greater accuracy as shown in Eq. (4).

$$S_d = \ln(a) + b \ln(IM1) + c \ln(IM2) + d\,(IM1)^2 + e\,(IM2)^2 + f\,(IM1 x IM2) \qquad (4)$$

After this, the fragility curve can be represented by a lognormal cumulative distribution function in Eq. (5):

$$Pf = \left(\frac{\ln\left(\frac{Sd}{Sc}\right)}{\sqrt{\beta c^2 + \beta_d^2}} \right) \quad (5)$$

Where,

S_c is the median value of structural capacity defined for the damage state

β_c is the lognormal standard deviation of the structural capacity

S_d is the seismic demand in terms of chosen ground motion intensity parameter and

β_d is the lognormal standard deviation for the demand.

The damage state and the corresponding values of Sc and βc taken from (Ghobarah, Aly, and El-Attar, 1997) for this research are shown in the Table 2.

Table 2
Damage Limit state median values

Damage state	S_c
Slight Damage (DS1)	0.14
Moderate Damage (DS2)	0.4
Extensive Damage (DS3)	0.6
Complete Damage (DS4)	1.0

6. Analytical Model Of Bridge

The bridge shall be modeled in the open source finite element software Opensees and the demand parameters are estimated due to various intensity of seismic loadings. Opensees is an open-source analysis program developed by the Pacific Earthquake Engineering Research Center at the University of California, Berkeley. Among various types of bridge two spanned RC bridge is chosen as the model for this study as the maximum number of bridges in Nepal are of this type. The components used in the model are shown in Fig. 2.

The bridge pier is modeled using displacement based nonlinear element dispBeamColumn element in Opensees. The distributed plasticity approach is followed in this research with fibers of concrete in conjunction with reinforcing steel. Material model of concrete is taken as concrete02 and for rebar Reinforcing steel form uniaxial material library. The bridge abutment is modeled using rigid beam elements and its interaction with the backfill soil is modeled using nonlinear bilinear element based on (Mosalam et al. 2015). The basic modeling approach in this case is modeling the longitudinal responses of the backfill (passive soil resistance) and expansion joint between the deck and abutment using gap element. The passive longitudinal response of backfill soil is modeled with an idealized bilinear elastic-perfectly-plastic force-deformation relationship. The vertical response of the bearing pad and abutment soil impedance is modeled using soil springs. The bridge deck is modeled as elastic beam column element with the un-cracked sectional properties. The stiffness of linear spring representing the foundation soil is calculated based on the (Gazetas 1992).

7. Results And Discussion

The comparison of Nonlinear Incremental dynamic analysis (IDA) for the mainshock only and mainshock with its aftershocks are shown in Fig. 3. The results shows that DS1 is achieved at the mean IM of 0.43g for mainshock only which is 15% less than mean IM of 0.51g for mainshock aftershock sequence. Similarly, DS4 is achieved at the mean IM of 0.99g for mainshock only which is 28% less than mean IM of 0.1.37g for mainshock aftershock sequence. Table 3 shows the comparison of mean IM values of different damage states for mainshock only and mainshock aftershock sequence.

Table 3
Comparison of mean IMs for different damage states

Damage	MS		MSAS	
	Mean(IM)	SD	Mean(IM)	SD
Slight	0.51g	0.05	0.43g	0.09
Moderate	0.9g	0.09	0.74g	0.17
Extensive	1.12g	0.12	0.89g	0.21
Complete	1.37g	0.2	0.99g	0.27
*MS = Mainshock only; MSAS = Mainshock with its aftershocks; SD = Standard deviation				

Similarly, the probabilistic seismic demand model (PSDM) obtained from the data of IDA is obtained as in Fig. 4.

Now, the results obtained were used to plot the fragility curves for MS and MSAS as shown in Fig. 5.

Table 4 compares the bridge fragility at different IMs for MS and MSAS. It shows that the probability of exceedance of slight damage stage in MSAS case is 4.8 times the same in MS case for DBE. Similarly, the probability of exceedance of extensive in MSAS case is 15 times the same in MS case for MCE. Most of the bridges in Nepal are designed based on (IRC:SP:114-2018) where the design philosophy suggests during DBE which can occur more than once in the structures lifetime, the bridge should only sustain minor damage. However, if we consider aftershocks the probability of exceedance of slight or minor damage increases form 4% to 21 %. Similarly, the design philosophy for MCE suggests the bridge can have structural damage but should be accessible to repair. But the probability of damage to go beyond extensive damage where concrete is unusable, increases from 2% to 31% if we consider aftershocks. This shows that the representative bridge chosen for this study fails to achieve the code based performance objectives if we consider aftershocks during performance evaluation.

Table 4
Fragility at different IMs for different damage states

Damage	0.35g		0.5g		0.75g		1g	
	MS	MSAS	MS	MSAS	MS	MSAS	MS	MSAS
Slight	0.043	0.21	0.61	0.81	1	1	1	1
Moderate	0	0.004	0.001	0.09	0.2	0.575	0.79	0.90
Extensive	0	0	0	0.035	0.02	0.31	0.31	0.66
Complete	0	0	0	0	0	0.1	0.016	0.34
*MS = Mainshock only; MSAS = Mainshock with its aftershocks								

The study of fragility of the bridge for new mainshock after the previous mainshock aftershock sequences can be done similar to the concept of multi-hazard analysis and the results can be shown quantitatively using a fragility surface. The PSDM obtained for the multi-hazard analysis is shown in Fig. 6.

The fragility surfaces obtained for the different damage states are shown in the Fig. 7 and the same plots in contours are shown in Fig. 8.

The fragility contours in Fig. 8 can help us identify the fragility of the bridge for a new mainshock after the first mainshock aftershock sequence. Now for example, the DBE can occur more than once in the lifetime of the bridge. If we consider the moderate damage fragility contour in Fig. 8(b) the probability of exceedance of the moderate damage state after first mainshock of 0.35g (DBE) and for a new mainshock of 0.35g is 40%. Since our representative bridge is designed based on IRC: SP: 114-2018 code, it should be able to fulfill the design philosophy for DBEs. However, 40% probability of exceedance of moderate damage doesn't seem to fulfill the philosophy. Therefore, rehabilitation or repair should be done based on the requirement after the DBE for these bridge cases. So, contours in Fig. 8 help us conduct the post mainshock aftershock rehabilitation process more quantitative and effective.

8. Conclusion

This study presents the results of considering aftershocks along with mainshock for the performance evaluation of a RC bridge as well as evaluates the expected performance of the bridge after the series of mainshock and aftershocks. With the help of Fig. 5 we can draw the following conclusions regarding the use of mainshock only or mainshock aftershock ground motion data for seismic performance evaluation;

- The fragility of the bridge for moderate and extensive damage states significantly increases for maximum credible earthquake using aftershocks.
- Similarly, the slight damage fragility significantly increases for design basis earthquake.
- For low demand (less than 0.35g); there is insignificant difference in expected bridge performance for all damage states.
- For high demand (more than 0.8g); there is significant difference in expected bridge performance for moderate, extensive and complete damage states.

Therefore, it is extremely important to consider subsequent aftershocks after a major earthquake during performance evaluation of a bridge and change considerations made in code provision for design accordingly in order to achieve the desired performance objectives.

Similarly, Figs. 7–8 help us identify the expected performance of the bridge after a major earthquake and its expected aftershocks. These types of study helps us make post hazard recovery and decision making more quantitative and efficient.

Declarations

Conflict of interest

The authors declare that they have no conflict of interest.

Funding details

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Informed Consent

Not applicable for this research

Authorship Contribution

Pradej Badal and Gokarna Motra contributed equally to carry out the research, to the analysis of the results and Pradej Badal contributed to the writing of the manuscript.

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Figures

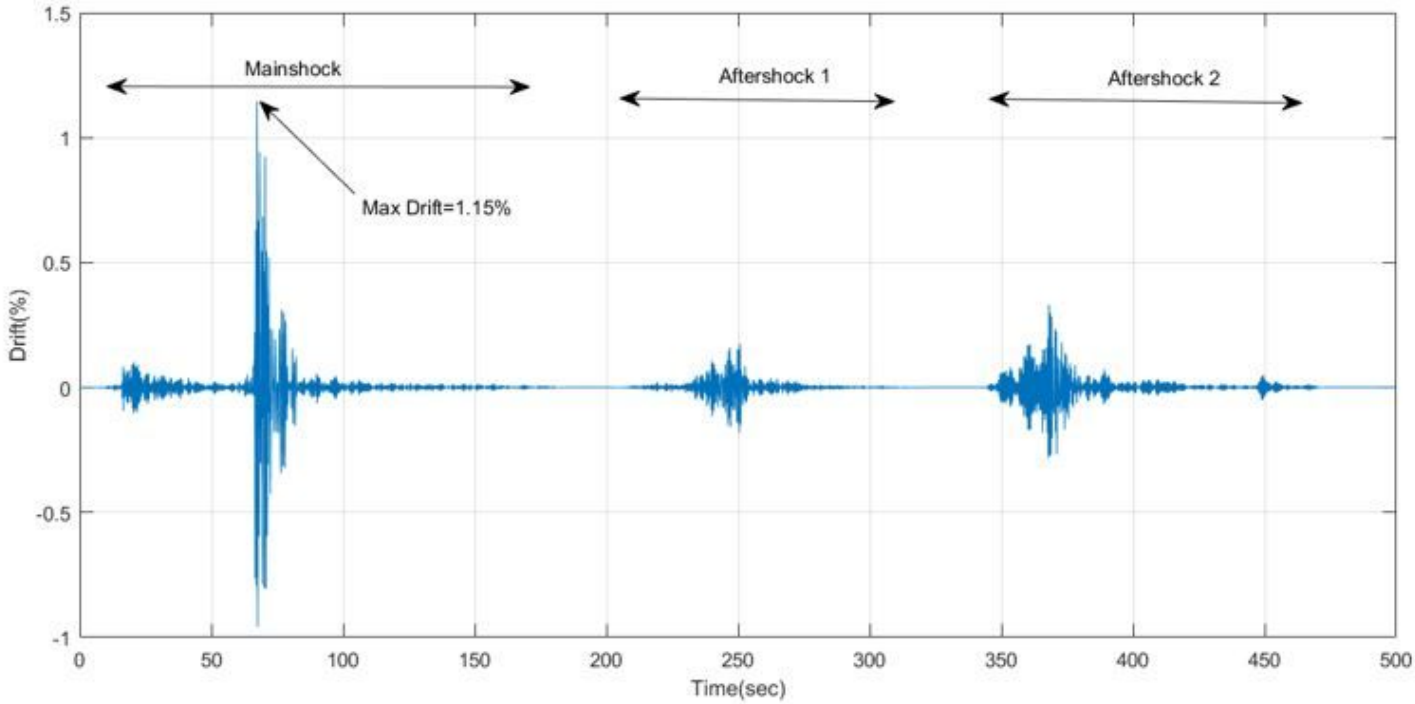


Figure 1

Mainshock Aftershock drift time-history of Tohokhu earthquake.

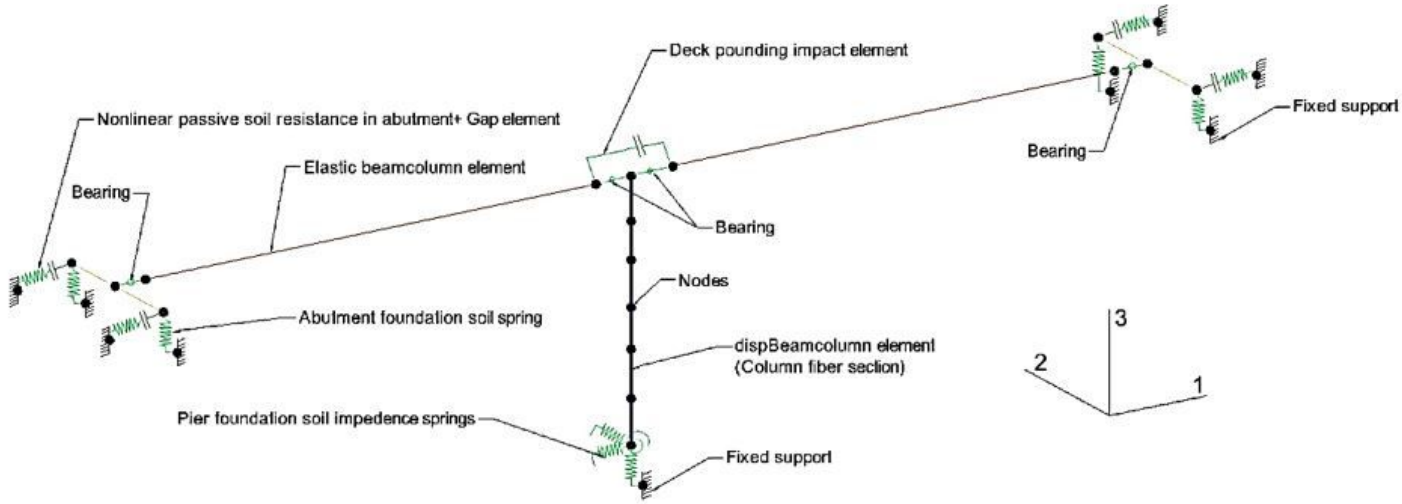
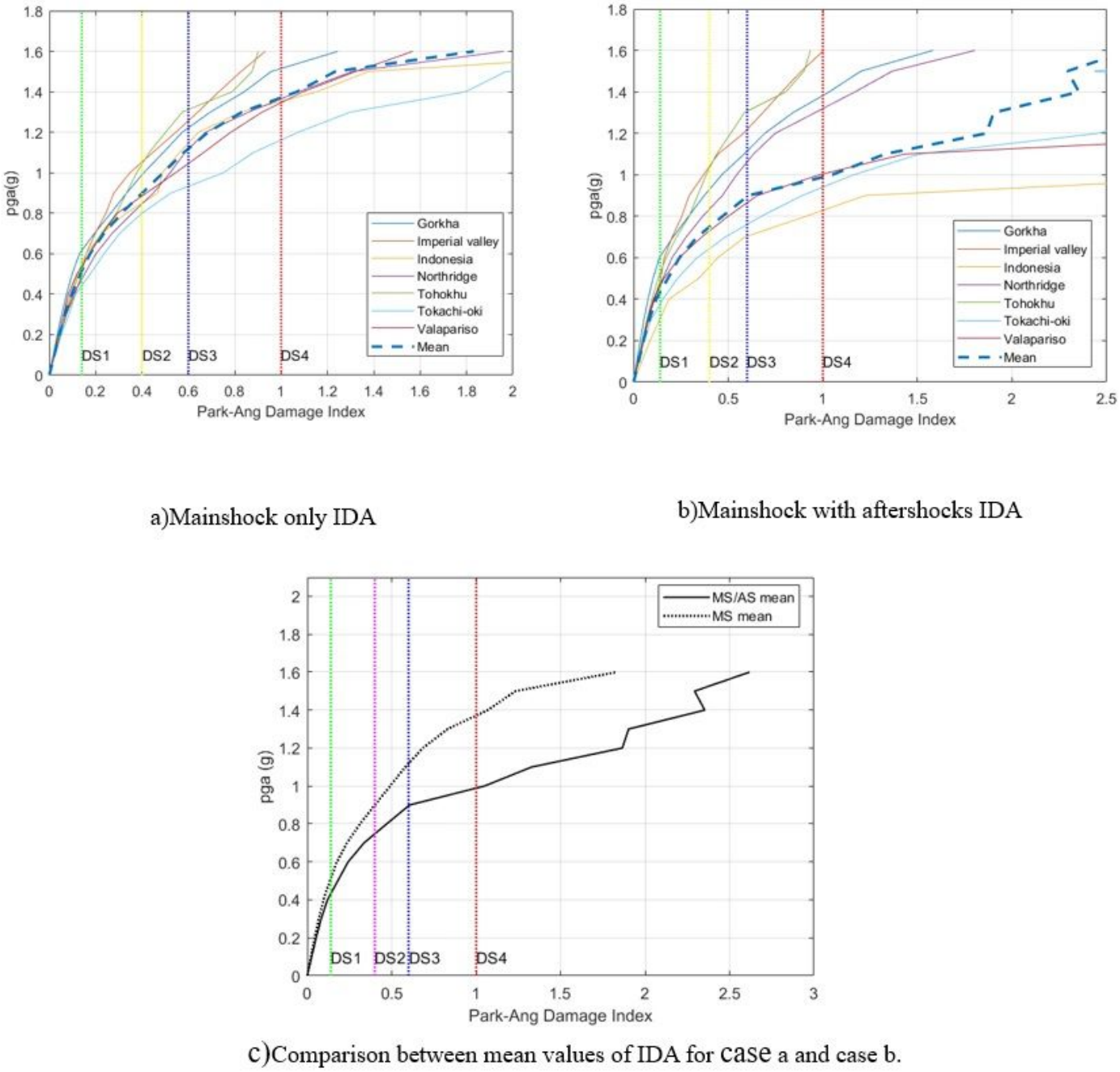


Figure 2

Analytical Model of Bridge



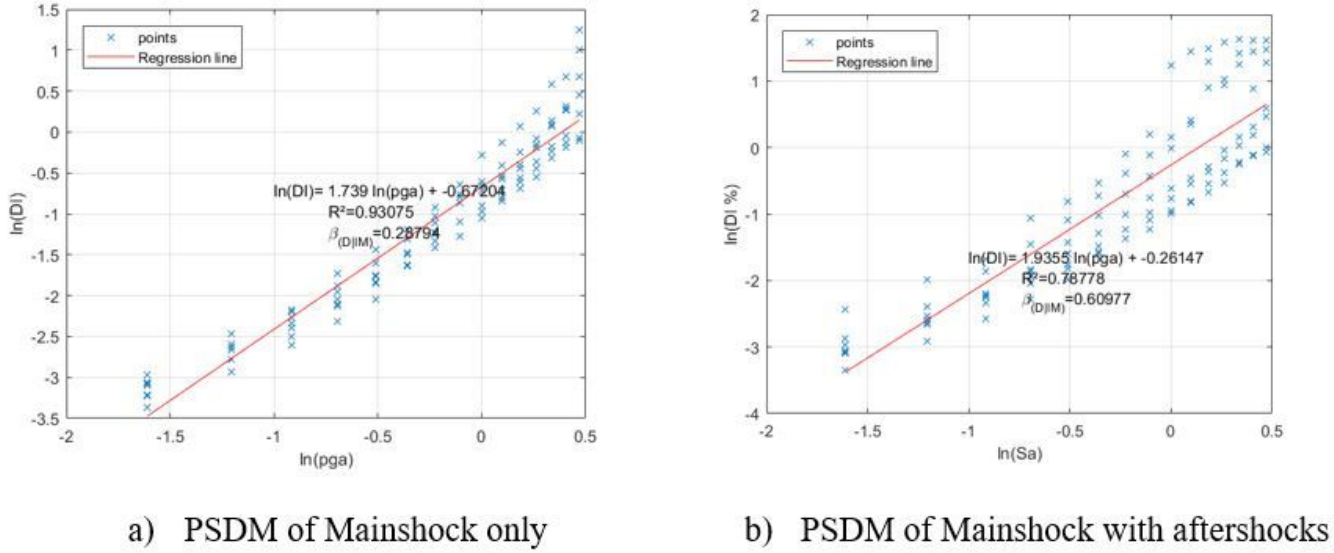
a)Mainshock only IDA

b)Mainshock with aftershocks IDA

c)Comparison between mean values of IDA for case a and case b.

Figure 3

IDA curves of the bridge *DS1=Slight damage; DS2=Moderate damage; DS3=Extensive damage; DS4=Complete damage.



a) PSDM of Mainshock only

b) PSDM of Mainshock with aftershocks

Figure 4

Figure PSDM comparison

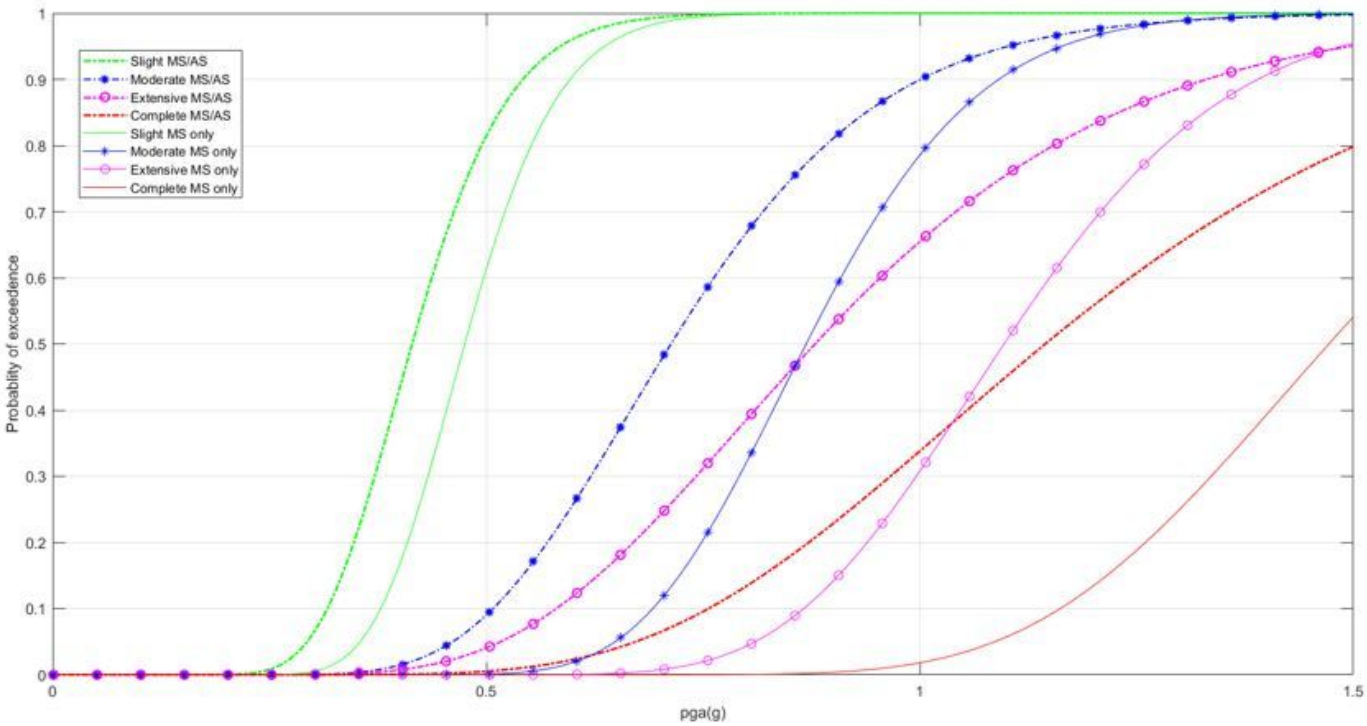


Figure 5

Comparison of fragility curves for 4 damage states.

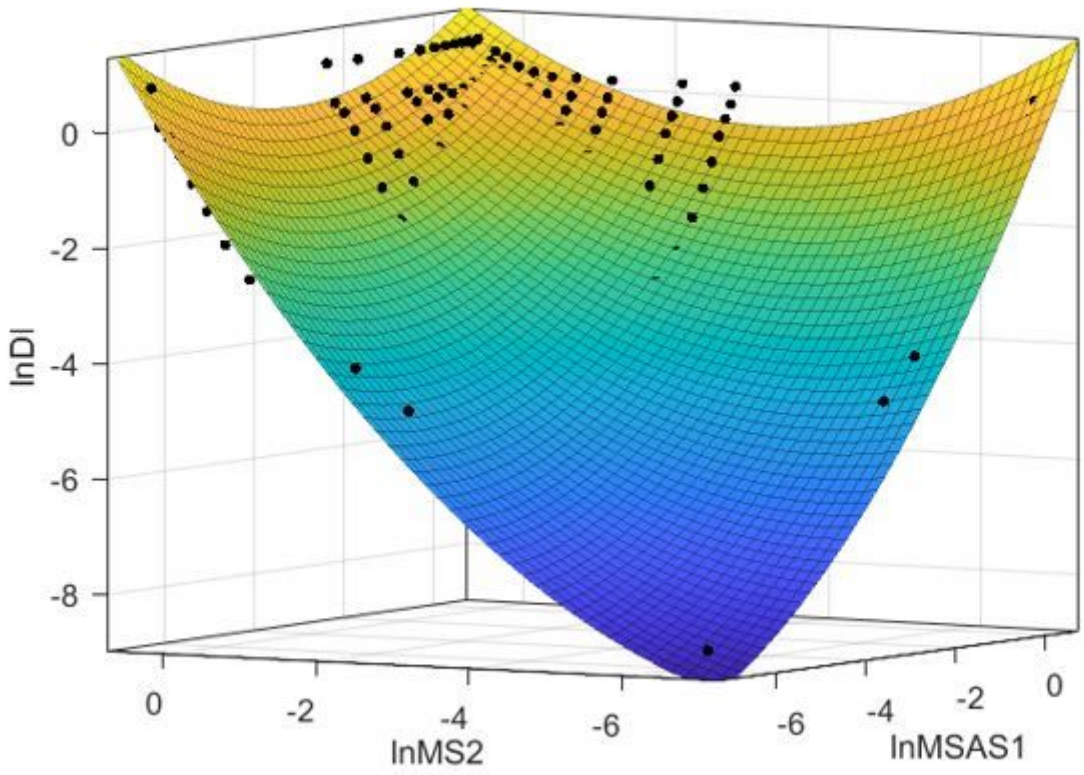


Figure 6

PSDM for multi-hazard analysis *MSAS1= Mainshock aftershock sequence; MS2= New Mainshock

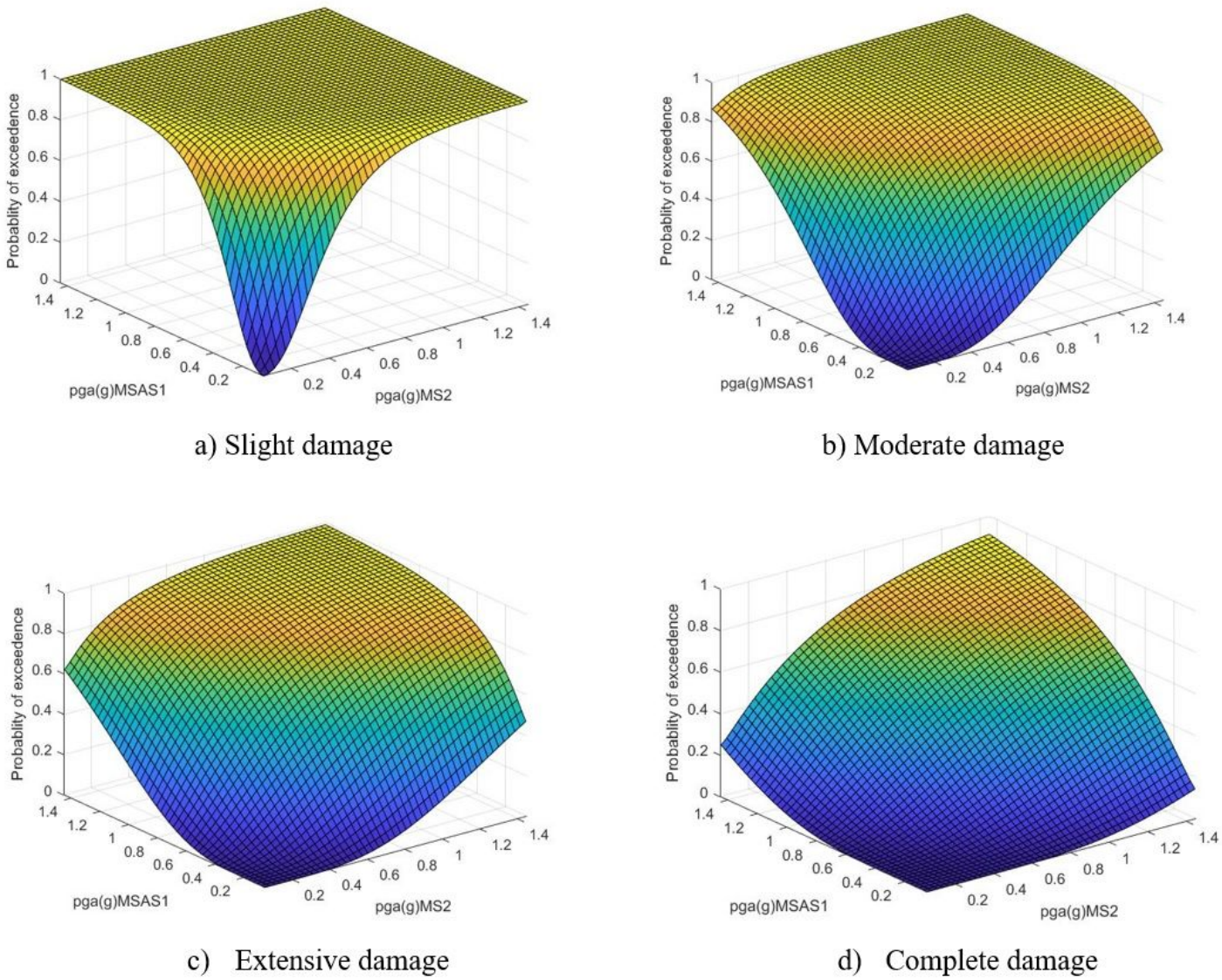


Figure 7

Fragility surfaces for 4 damage states. *MSAS1= Mainshock aftershock sequence; MS2= New Mainshock

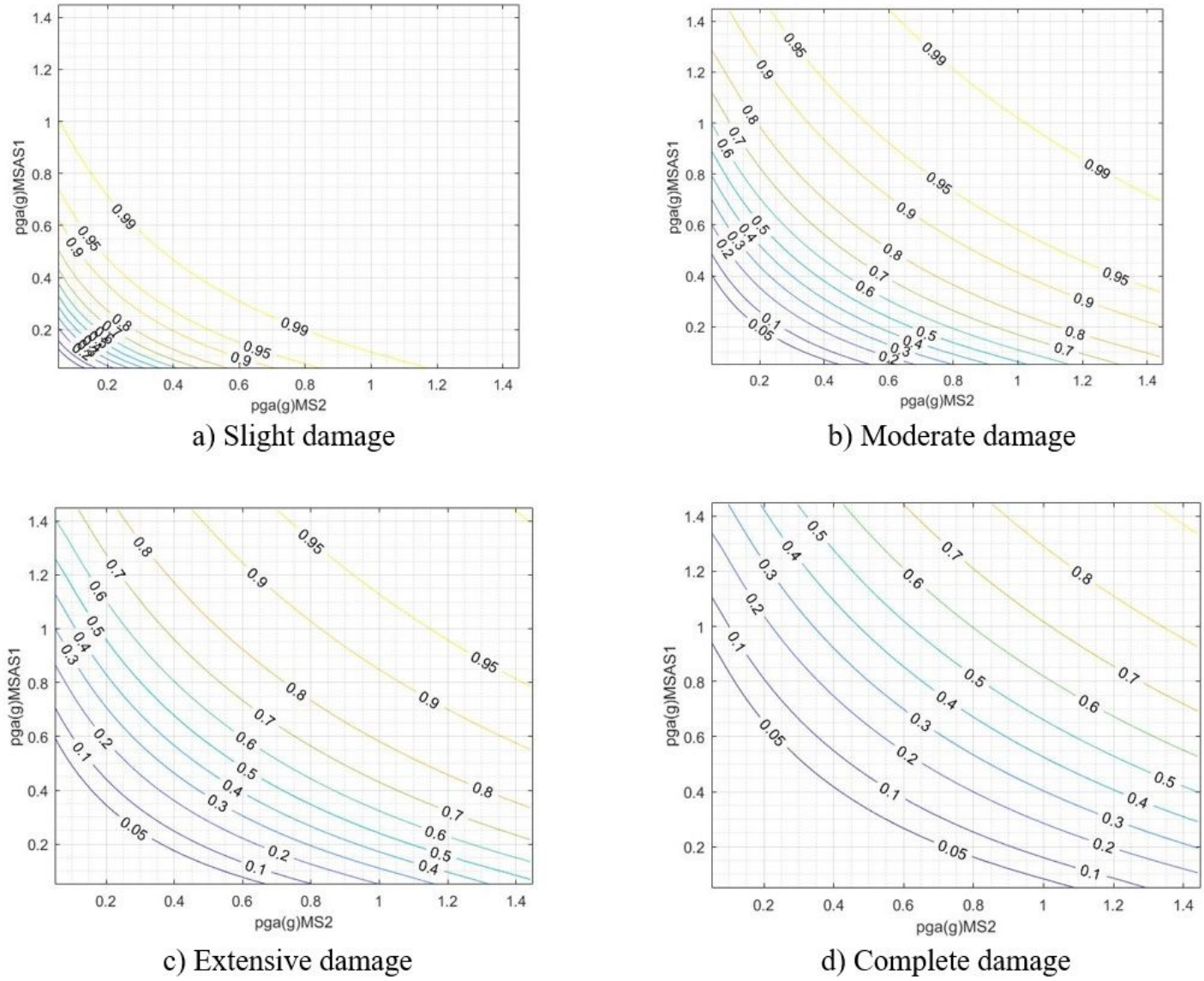


Figure 8

Fragility contours for 4 damage states. *MSAS1= Mainshock aftershock sequence; MS2= New Mainshock