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Probabilistic seismic demand of isolated straight concrete girder highway bridges using fragility functions

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Abstract. In this study, it has been tried to prepare an analytical fragility curves for isolated straight continues highway bridges by considering different spectral intensity measures. A three-span concrete isolated bridge has been selected and the seismic performance of the bridge has been improved by Lead Rubber Bearing (LRB). Incremental Dynamic Analysis (IDA) is applied to the bridge in longitudinal direction. A suite of 14 earthquake ground motions from medium to sever motions are scaled and used for nonlinear time history analysis. Fragility function considers the relationship of earthquake intensity measures (IM) and probability of exceeding certain Damage State (DS). A full three dimensional finite element model of the isolated bridge has been developed and analyzed. A wide range of different intensity measures are selected and the optimal intensity measure which has the less dispersion is proposed.

Keywords: straight highway bridge; fragility curve; Incremental Dynamic Analysis (IDA); finite element modeling

1. Introduction

Fragility curves are coming into use to provide information about the effect of earthquakes on highway bridges. Fragility curves can estimate the amount of damage a bridge will suffer with various severities of seismic activity above a certain level of damage. This information is useful for planning purposes, assessing the need for retrofitting and estimating losses.

There are two common ways to develop the fragility curves the first one is based on empirical results from the data base of the past earthquakes and the second one is analytical approach using the results of nonlinear time history analysis of structure under large number of earthquake ground motions.

The advantages of analytical approaches are the fragility curves can be generated for the structures which has not experienced any earthquakes yet. Analytical approaches are also divided in two categories: 1) cloud approach 2) Incremental Dynamic Analysis (IDA).

In this study, incremental dynamic approach has been used to considering the behavior of the structural response and consideration of different damage states.

Padgett *et al.* (2007) proposed and applied a new selection criterion such as efficiency, practicality, sufficiency, and hazard computability in their selections choose an optimal intensity measure.

Zhange *et al.* (2009), has presented the methodology to assess the optimum design parameters for isolation devices for highway bridges using the fragility function method. Shafieezadeh *et al.* (2012) presented the concept of using fractional order intensity measures (IMs) in probabilistic seismic demand analysis. They had studied the results on the multi-span continuous steel girder bridge class as case studies. Billah *et al.* (2015) presented a review of the different methodologies developed for seismic fragility assessment of highway bridges along with their features, limitations and applications. Bayat *et al.* (2015) has done a great study on the proposing a new intensity measure for skewed highway bridges average spectral acceleration (ASA) in their studies. In recent years, there are many scientists have been working on the probabilistic seismic demand analysis of structures and nonlinear analysis of bridges such as: (Akcay *et al.* 2016, Deepu *et al.* 2014, Fang *et al.* 2013, Nielson 2005, Padgett *et al.* 2010, Pan *et al.* 2010, Parghi *et al.* 2017, Alam *et al.* 2018, Muto *et al.* 2008, Baker 2006, Vamvatsikos *et al.* 2002, Dolsek 2009, Aslani *et al.* 2005, Baker 2015, Wu *et al.* 2018, Bayat *et al.* 2017, Ansari *et al.* 2019, Lazzari *et al.* 2019, Jeon *et al.* 2018, Ye *et al.* 2017, Alam *et al.* 2019, Ozkaynak*et al.* 2017, Kia *et al.* 2016, 2017).

In this paper, three dimensional of straight continues isolated highway bridge has been created and studied. A full incremental dynamic analysis has been done on the bridge and the results of nonlinear time history has been used to presentation of an optimal intensity measure and generation of fragility curves.

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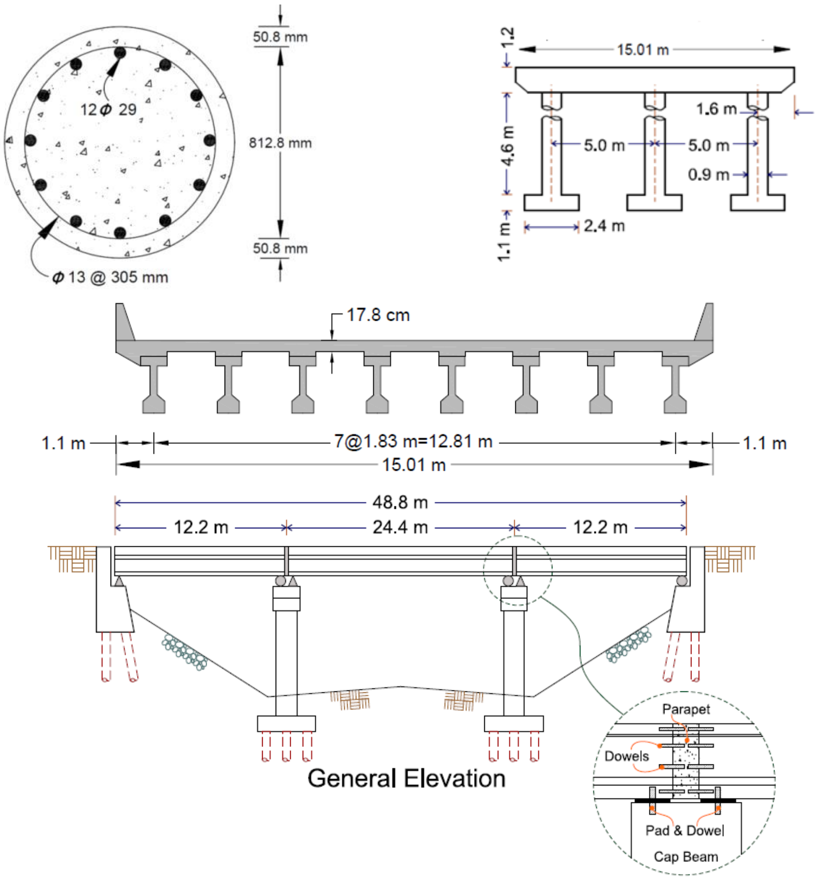

Fig. 1 General elevation and concrete member reinforcing layout; deck detail; column (Nielson 2005)

2. Bridge modeling and verification

2.1 Modeling description

The selected model in this paper has been derived from Nielson (2005) and selected and reference bridge. A three span straight continues concrete girder highway bridge is shown in Fig. 1. The spans are made continuous by casting a concrete parapet between the deck and girders as is illustrated in the detail of Fig. 1. The total length of the bridge is 48.8 m and the side span are 12.2 m and the middle span has the 25.4 m length. The width of the bridge is 15.01 m and supported with eight AASHTO type prestressed concrete girders. The isolation devices have been applied between the deck and the column. The first stiffness ratio to the second stiffness of the LRB is 25 and the design period of the bridge is 3s. More detailed information of the bridge modelling properties can be found previous studies (Nielson 2005, Bayat *et al.* 2015). Finite element modelling of the bridge was developed by using SAP2000 V.14.2.4.. The LRBs are molded as a bilinear behavior using 2 joint link elements.

3. Seismic ground motion records

A great record selection and suggestion has been done by FEMA-P695 (FEMA 2009) from a great various records data base. The selected records are presented in Table 1.

Table 1 Characteristics of the earthquake ground motion histories (FEMA 2009)

ID No	Earthquake				Recording station	
	M	PGA (g)	Year	Name	Name	owner
1	7.0	0.48	1992	Cape Mendocino	Rio Dell Overpass	USGS
2	7.6	0.21	1999	Chi-Chi, Taiwan	CHY101	CWB
3	7.1	0.82	1999	Duzce,Turkey	Bolu	ERD
4	6.5	0.45	1976	Friuli, Italy	Tolmezzo	------------
5	7.1	0.35	1999	Hector Mine	Hector	SCSN
6	6.5	0.34	1979	Imperial Valley	Delt	UNAMUCSD
7	6.9	0.38	1995	Kobe, Japan	Nishi-Akashi	CUE
8	7.5	0.24	1999	Kokaeli,Turkey	Duzce	ERD
9	7.3	0.36	1992	Landers	Yemo Fire Station	CDMG
10	6.9	0.42	1989	Loma Prieta	Capitola	CDMG
11	7.4	0.56	1990	Manjil	Abbar	BHRC
12	6.7	0.55	1994	Northridge	Beverly Hills-Mulhol	USC
13	6.6	0.36	1971	San Ferando	LA-Hollywood Stor	CDMG
14	6.5	0.51	1987	Superstition Hills	El Centro Imp.Co	CDMG

The records scaled to PGA=1 g and applied to the bridge with the increment of 0.1 g.

4. Incremental Dynamic Analysis (IDA)

Incremental dynamic analysis (IDA) is a novel approach which was first proposed by Vamvatsikos and Cornell (2002) to capture the behavior for the structures from linear to nonlinear range and collapse state. In incremental dynamic analysis, each earthquake records are scaled to 1 g and applied to the structure with the increment of 0.1 g. In each step a full nonlinear time history analysis has been applied to the bridge and response of the bridge captures in each step.

5. Intensity measures and fragility curves

5.1 Intensity measure (IM)

The IDA results are generally presented in terms of Engineering Demand Parameter (EDP) and Intensity Measure (IM). Relation between SD and IM estimated as (Padgett *et al*. 2010)

$$S_D = a\, IM^{\,b} \tag{1}$$

With a linear regression we can obtain the coefficient of *a* and *b* and re-written the Eq. (5) as

$$\ln(SD) = b.\ln(IM) + \ln(a) \tag{2}$$

Different Intensity Measures can be classified by following criteria to select the optimal intensity measure

5.1.1 Efficient intensity measure

The efficiency of an Intensity measure is related to the less dispersion of about the median of the results of nonlinear time history analysis. The lower values of $\beta_{D|IM}$ leads to a more efficient intensity measure Padgett *et al*. (2010).

5.1.2 Practical intensity measure

The practicality of an intensity measure is evaluated by the parameter” in in Eq. (2). The higher value of “*b*” leads to a more practical intensity measure in comparison together Padgett *et al*. (2010).

5.1.3 Proficient intensity measure

Padgett *et al*. (2010) composite the measure of efficiency and practically as a new criteria of selecting an optimal intensity measure as follow formulation

$$P[D \geq d \,|\, IM] = 1 - \phi\left(\frac{\ln(IM) - \dfrac{\ln(d) - \ln(a)}{b}}{\dfrac{\beta_{D|IM}}{b}} \right) \tag{3}$$

A lower values of modified dispersion is a more proficient *IM*

$$\zeta = \frac{\beta_{D|IM}}{b} \tag{4}$$

5.2 Fragility function

Fragility function had formulated by Cornell *et al.* (2000) and the fragility curves are presented as lognormal distribution

$$P[D \geq d \,|\, IM] = 1 - \phi\left(\frac{\ln(d) - \ln(S_D)}{\beta_{D|IM}} \right) \tag{5}$$

$\phi(\bullet)$=Standard normal cumulative distribution function

S_D=Median value of the structural demand in terms of a seismic intensity

$\beta_{D|IM}$=Logarithmic standard deviation, or dispersion, of the demand conditioned on the IM. The The dispersion of the mean demand conditioned on the IM is

$$\beta_{D|IM} \cong \sqrt{\frac{\sum\left(\ln(d_i) - \ln\left(b.\ln(IM) + \ln(a)\right)\right)^2}{N-2}} \tag{6}$$

N=number of ground motions

d_i=Peak demands

Table 2 Damage State (DS) for concrete columns and bearings (Zhang *et al*. 2009)

Component/ Damage	Slight (DS=1)	Moderate (DS=2)	Extensive (DS=3)	Collapse (DS=4)
Column	Cracking and spalling	Moderate cracking and spalling	Degradation w/o collapse	Failure leading to collapse
	μ>1	μ>2	μ>4	μ>7
	θ>0.007	θ>0.015	θ>0.007	θ>0.025
Bearing	γ>100%	γ>150%	γ>100%	γ>200%

6. Damage States (DS) of the nonlinear time history analysis

Damages in isolated bridges are located in isolation system and rest of the damages are concentrated as column drift ratio (Zhang *et al*. 2009)

$$DS_{system} = \max\left(DS_{pier}, DS_{bearing}\right) \tag{7}$$

7. Results and discussions

As a result of incremental dynamic Analysis (IDA) and developing fragility curves, the optimal intensity measure has been selected and presented with figures. The most important parameter in selection of optimal intensity measure is to have an efficiency. Efficiency of an IM is showing its less dispersion in lognormal space. The results of spectral intensity measure are presented in log-normal spaces in Figs. 2 to 12. The linear regression analysis has two parameters “*a*” and “*b*” are the linear regression analysis. The efficiency of the intensity measure is studied with the parameters $\beta_{D|IM}$. The “*b*” parameter is the practicality of the intensity measure. The composite measure of the practicality and the efficiency are in parameter $\zeta = \frac{\beta_{D|IM}}{b}$. The lower values of $\zeta = \frac{\beta_{D|IM}}{b}$ show the more proficient intensity measure. Table 3 is the full

Table 3 Comparisons of regression values of PGA and Spectral Intensity measures and dispersion values

IM	*Ln* (a)	*b*	$\beta_{D\|IM}$	$\zeta = \frac{\beta_{EDP\|IM}}{b}$
PGA(g)	0.791	0.77	2.3	0.75
Sa(0.1Ts,5%)	0.77	0.757	1.83	0.78
Sa(0.2Ts,5%)	0.85	0.872	1.9	0.68
Sa(0.3Ts,5%)	**0.853**	**0.852**	**1.99**	**0.62**
Sa(0.4Ts,5%)	0.785	0.779	2.11	0.82
Sa(0.5Ts,5%)	0.713	0.699	2.23	0.82
Sa(0.6Ts,5%)	0.609	0.607	2.31	1.24
Sa(0.7Ts,5%)	0.637	0.637	2.46	1.43
Sa(0.8Ts,5%)	0.653	0.652	2.52	1.55
Sa(0.9Ts,5%)	0.678	0.676	2.59	1.62
Sa(Ts,5%)	0.61	0.609	2.36	1.84

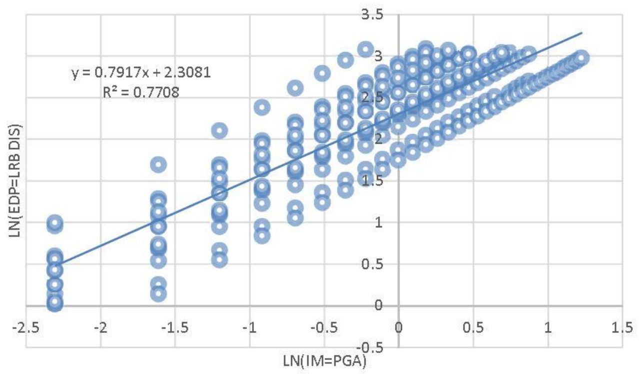

Fig. 2 Simulated maximum LRB displacement (as EDP) of bridge as a function of PGA (as IM) of earthquake motions

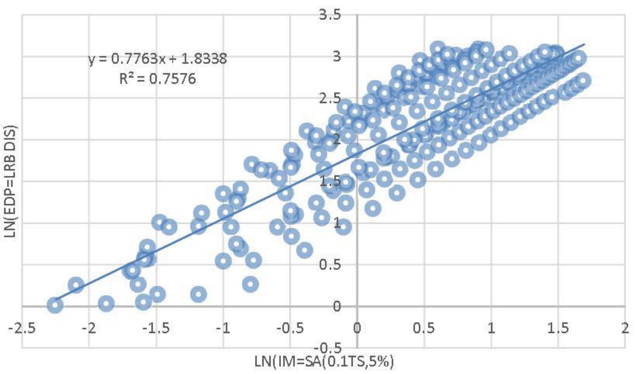

Fig. 3 Simulated maximum LRB displacement (as EDP) of bridge as a function of Sa(0.1Ts,5%) (as IM) of earthquake motions

comparison of the selected spectral intensity measures and comparing their efficiency, practicality and proficiency of them. From the results, it has been indicated that the Sa(0.3Ts,5%) is the proficient intensity measure which improves the results comparing to PGA more that 9%.

Therefore, we have developed the fragility curve of the bridge for this intensity measure as its shown in Figs. 13 to 14 for PGA and Sa(0.3Ts,5%). Based on the proficiency of the IMs, the fragility curve presented with Sa(0.3Ts,5%) is more accurate than the one with PGA.

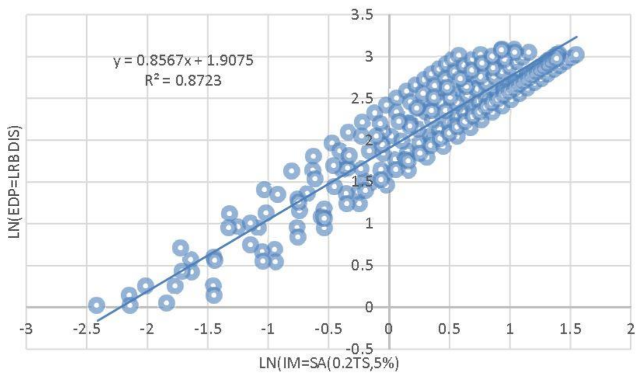

Fig. 4 Simulated maximum LRB displacement (as EDP) of bridge as a function of Sa(0.2Ts,5%) (as IM) of earthquake motions

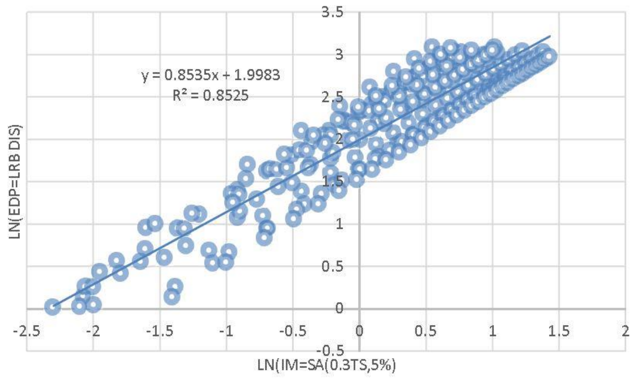

Fig. 5 Simulated maximum LRB displacement (as EDP) of bridge as a function of Sa(0.3Ts,5%) (as IM) of earthquake motions

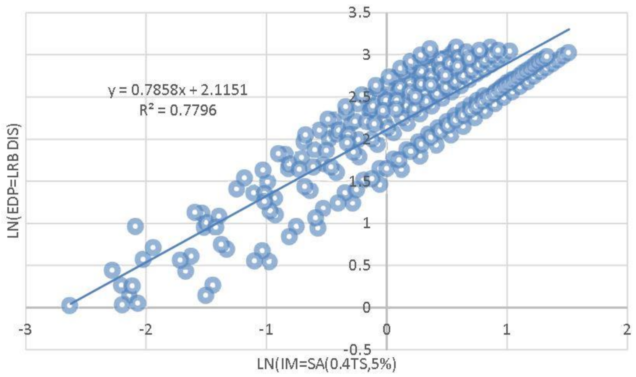

Fig. 6 Simulated maximum LRB displacement (as EDP) of bridge as a function of Sa(0.4Ts,5%) (as IM) of earthquake motions

The presented fragility curves for Sa(0.3Ts,5%) is the most trustable fragility curve for the selected isolated bridge.

8. Conclusions

In this study, it has been tried to developed the accuracy of the fragility curves for isolated straight highway bridges. The first step to have more accurate fragility curves is to

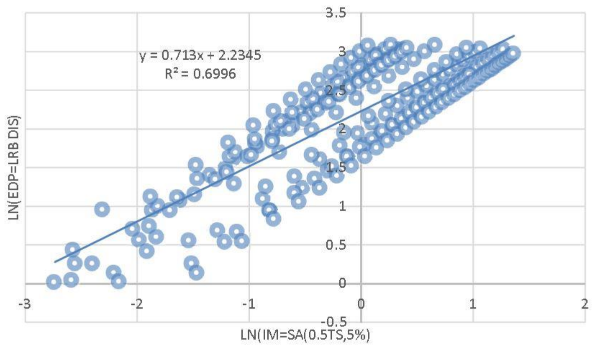

Fig. 7 Simulated maximum LRB displacement (as EDP) of bridge as a function of Sa(0.5Ts,5%) (as IM) of earthquake motions

2.4

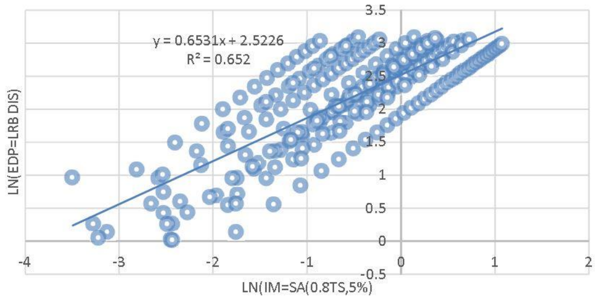

Fig. 10 Simulated maximum LRB displacement (as EDP) of bridge as a function of Sa(0.8Ts,5%) (as IM) of earthquake motions

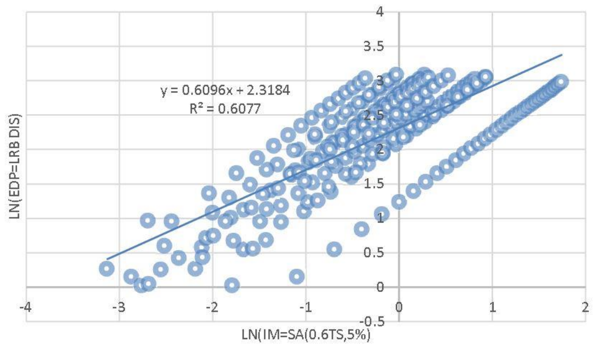

Fig. 8 Simulated maximum LRB displacement (as EDP) of bridge as a function of Sa(0.6Ts,5%) (as IM) of earthquake motions

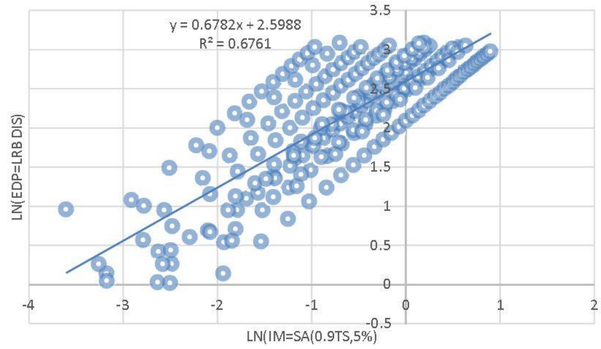

Fig. 11 Simulated maximum LRB displacement (as EDP) of bridge as a function of Sa(0.9Ts,5%) (as IM) of earthquake motions

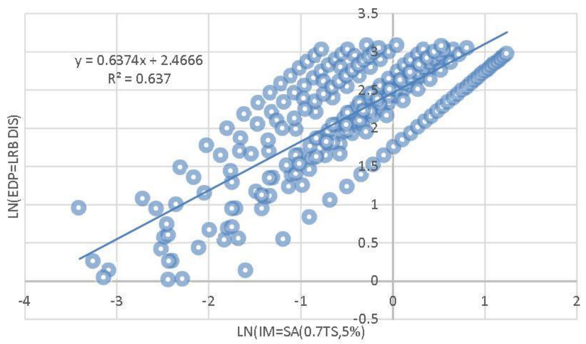

Fig. 9 Simulated maximum LRB displacement (as EDP) of bridge as a function of Sa(0.7Ts,5%) (as IM) of earthquake motions

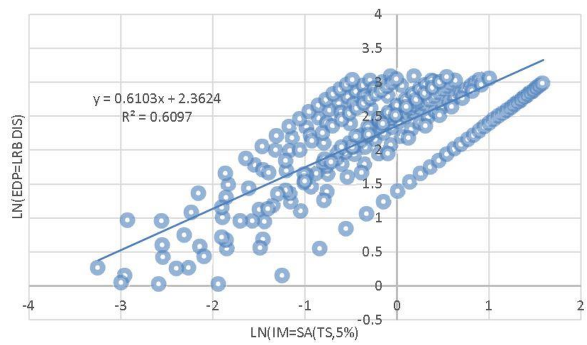

Fig. 12 Simulated maximum LRB displacement (as EDP) of bridge as a function of Sa(Ts,5%) (as IM) of earthquake motions

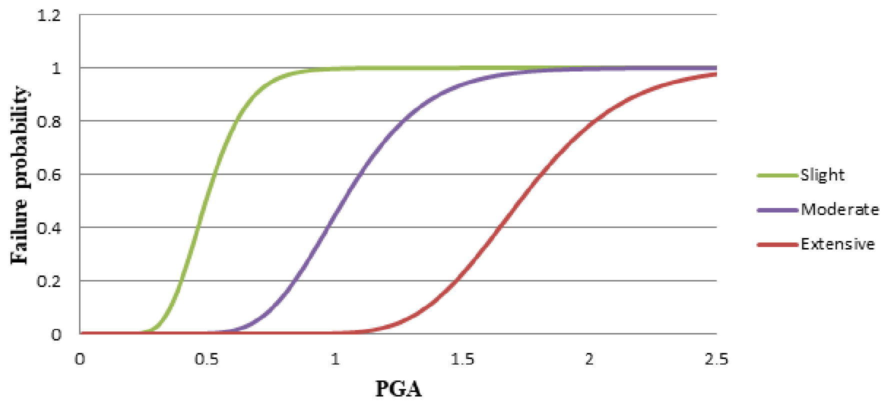

Fig. 13 Fragility curves for the isolated bridge for PGA

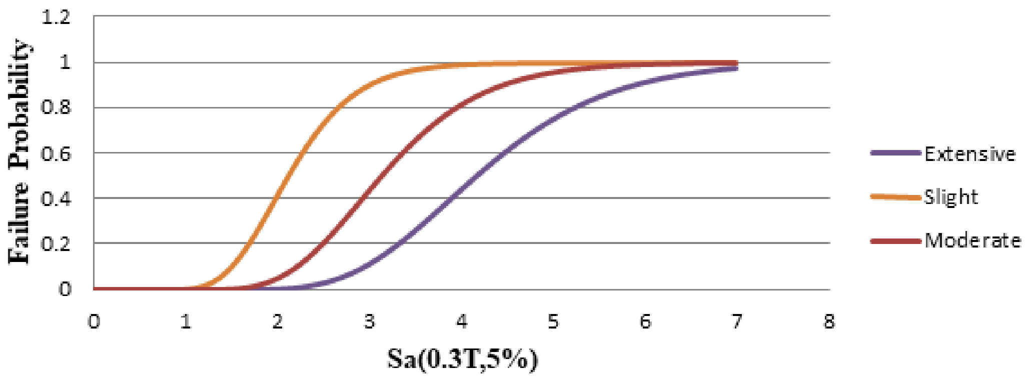

Fig. 14 Fragility curves for the isolated bridge for Sa(0.3Ts,5%)

have an optimal intensity measure which is more proficient intensity measure. A sensitive study has been carried out on the spectral intensity measure based on the first period of the bridges. The presented intensity measure which is Sa(0.3Ts, 5%) is more proficient intensity measure for isolated bridges. The analytical fragility curves are also developed based on the mentioned intensity measures and also have presented for the common one which is PGA.

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