IABSE-JSCE Joint Conference on Advances in Bridge Engineering-III, August 21-22, 2015, Dhaka, Bangladesh. ISBN: 978-984-33-9313-5
Amin, Okui, Bhuiyan, Ueda (eds.) www.iabse-bd.org

Reliability based seismic performance analysis of retrofitted bridge bent

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ABSTRACT: This study focuses on reliability-based seismic performance analysis of a bridge bent using analytical fragility curves under far field earthquake ground motion records. The fragility curves are prepared to assess the relative performance of the bridge bent retrofitted with two techniques: concrete jacketing (CJ) and carbon fibre reinforced polymer (CFRP) jacketing. In this regard, the method of probabilistic seismic demand model (PSDM) is used to derive the analytical fragility curves using nonlinear time-history analyses of the bridge bent. The PSDM establishes a correlation between displacement ductility demand of the bridge bent and the peak ground acceleration (PGA) of each ground motion record. A total of 200 earthquake excitations of far field earthquake ground motion records are utilized to evaluate the seismic responses of the bridge bent. The results obtained from this study indicate that the bridge bents retrofitted with CFRP possess less vulnerability at different damage states under the given earthquakes.

1 INTRODUCTION

Bridges are essential components of an overall transportation system as they play important roles in evacuation and emergency routes for rescues, first-aid, firefighting, medical services and transporting disaster commodities. The performance of highway bridge systems observed in past earthquakes—including the 1971 San Fernando earthquake, the 1994 Northridge earthquake, the 1995 Great Hanshin earthquake in Japan, the 1999 Chi-Chi earthquake in Taiwan, the 2010 Chile earthquake, and the 2010 Haiti earthquake—have demonstrated that bridges are highly susceptible to damages during earthquakes (Alim et al., 2014). Bridges give the impression of being rather simple structural systems. Indeed, they have always occupied a special place in the affections of structural designers because their structural form tends to be a simple expression of their functional requirement. Bridges, possibly because of their structural simplicity, have not performed well as might be expected under seismic attack. In recent earthquakes in California in 1989, Japan in 1995, etc. modern bridges designed specifically for seismic resistance have collapsed or have been severely damaged when subjected to ground shaking of an intensity that has frequently been less that corresponding to current code intensities (Alim, 2014).

A fragility curve displays the conditional probability that a structure surpasses some defined limit state at different levels of load or other actions. For seismic fragility, the curves represent the probability of seismic damage at various levels of ground shaking, which is described for the purposes of this research in terms of peak ground acceleration (PGA). Since last decade, several authors have tried to explain this term with different parameters from different seismic eyesight's (Alim, 2014). Most of these assumptions and explanations were mainly focused on different civil engineering structures - particularly on buildings and bridge structures. Yamazaki et al., (2000) developed a set of empirical fragility curves based on the actual damage data acquired from the 1995 Hyogo-ken Nanbu (Kobe) earthquake. Shinozuka et al., (2000) presented both empirical and analytical approaches for fragility curves. Kim and Shinozuka (2004) then developed fragility curves for concrete bridges retrofitted by column steel jacketing. The fragility curves were expressed in the form of a two parameter lognormal distribution function with the estimation of the two parameters performed an optimization algorithm, and it could be achieved through ground motion records and seismic structural response analyses (Alim, 2014).

2 MODELING OF THE BRIDGE

2.1 *Physical Model*

To evaluate the seismic performance of the bridge bent, Bahadarhat flyover considered in this study. A typical 40 *m* span with 7.29 *m* high pier is considered for the study. The bent's geometric configuration is shown in Figure 1. Seven girders and a concrete deck are spanning between the two bents.

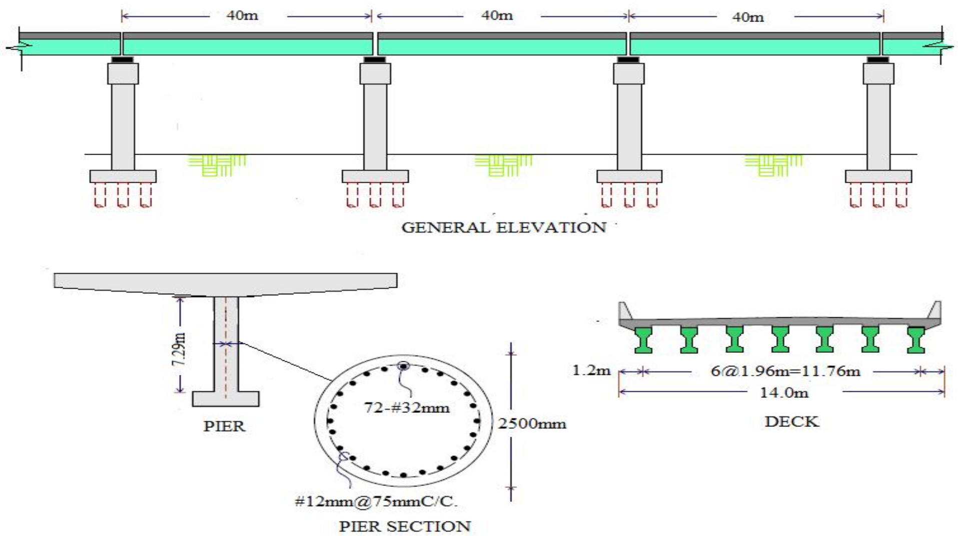

Figure 1. Geometry of span of the typical bridge

2.2 *Analytical Model*

The analytical model of a bridge bent along with a bridge pier is shown in Figure 2. The analytical model of the bridge bent is approximated as a continuous 2-D finite element frame using the SeismoStruct nonlinear analysis program (SeismoStruct, 2010). 2-D inelastic beam elements have been used for modeling the bridge component. This simplification holds true only when the bridge superstructure is assumed to be rigid in its own plane which shows no significant structural effects on the seismic performance of the bridge system when subjected to earthquake ground acceleration in longitudinal direction. Here, fiber modeling approach has been employed to represent the distribution of material nonlinearity along the length and cross-sectional area of the member. The confinement effect of the concrete section is considered on the basis of reinforcement detailing. To develop the analytical model, Menegotto-Pinto steel model (Menegotto and Pinto, 1973) with Filippou (Filippou et al., 1983) isotropic strain hardening property is used for reinforcing steel material. FRP confined concrete model developed by Ferracuti and Savoia (2005) has been implemented. In this model the confinement effect of the FRP wrapping follows the rules proposed by Spoelstra and Monti (1999). The pier is modeled by using beam column element. Fiber Model is used to generate the section of the pier. In the current study, the nonlinear Fiber section is used to model the concrete pier. The pier and pier cap are modeled by using beam element. Pier cap is considered as solid concrete element for simplicity. Fiber section is used to model the concrete pier cap. The following figure shows the modeling of pier cap.

2.3 *Ground Motion for Incremental Dynamic Analysis*

A suite of 20 near fault ground motions are used in this study to develop fragility curves for the as-built and retrofitted bridge bents. The far field ground motions were adopted for this analysis. The characteristics of the earthquake ground motion records are presented in Table 1. All these ground motions have very high PGA ranging from 0.24g to .0728g (Alim, 2014 and Alim et al. 2014).

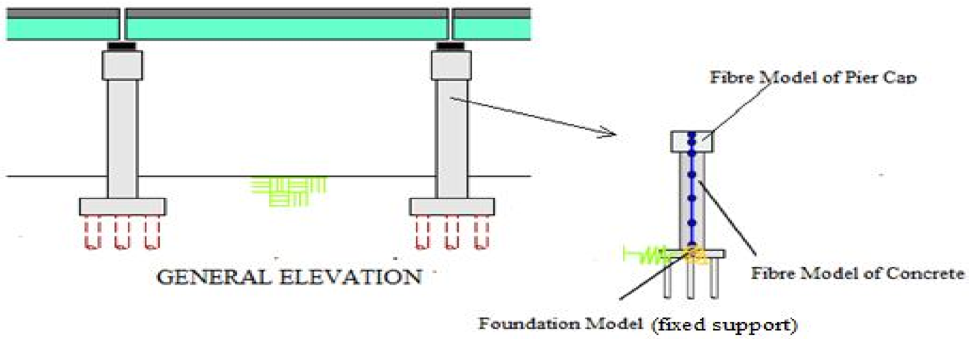

Figure 2. Analytical model of the bridge bent

Table 1. Characteristics of the earthquake ground motion histories

Earthquake No.	Name	Recording Station	PGAmax (g)	PGVmax (cm/s.)
EQ-1	Northridge	Beverly Hills - Mulhol	0.416	58.95
EQ-2	Landers	Yermo Fire Station	0.24	51.5
EQ-3	Northridge	Canyon Country-WLC	0.4	43.0
EQ-4	Landers	Coolwater	0.283	26
EQ-5	Duzce, Turkey	Bolu	0.7	56.4
EQ-6	Loma Prieta	Capitola	0.53	35
EQ-7	Hector Mine	Hector	0.3	28.6
EQ-8	Loma Prieta	Gilroy Array #3	0.56	36
EQ-9	Imperial Valley	Delta	0.2	26.0
EQ-10	Manjil, Iran	Abbar	0.51	43
EQ-11	Imperial Valley	El Centro Array #11	0.4	34.4
EQ-12	Superstition Hills	El Centro Imp. Co.	0.36	46.4
EQ-13	Kobe, Japan	Nishi-Akashi	0.5	37.3
EQ-14	Superstition Hills	Poe Road (temp)	0.45	35.8
EQ-15	Kobe, Japan	Shin-Osaka	0.2	38.0
EQ-16	Cape Mendocino	Rio Dell Overpass	0.385	43.8
EQ-17	Kocaeli, Turkey	Duzce	0.3	59.0
EQ-18	Chi-Chi, Taiwan	CHY101	0.353	70.65
EQ-19	Kocaeli, Turkey	Arcelik	0.2	17.7
EQ-20	Chi-Chi, Taiwan	TCU045	0.474	36.7

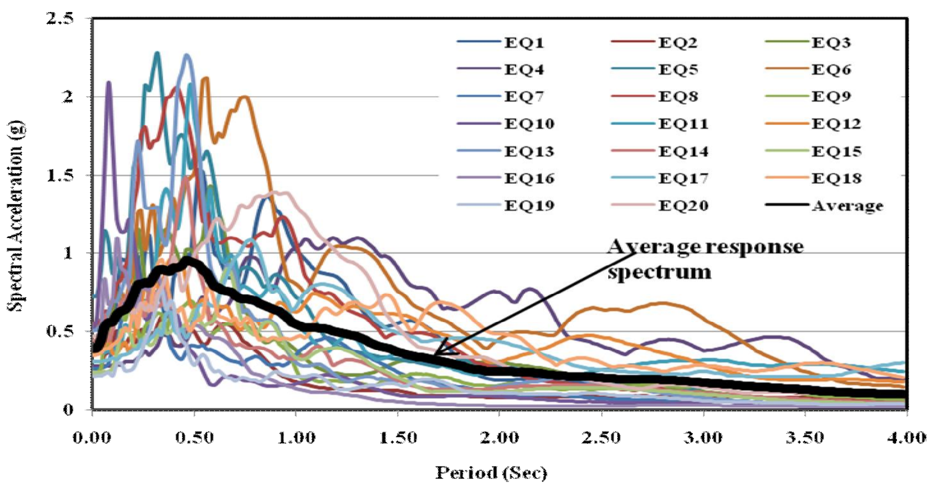

Figure 3. Spectral acceleration of earthquake ground motion records

Figure 3 shows the acceleration response spectra with 5% damping ratio of the recorded far field ground motions. Figure 4 shows the different percentiles of acceleration response spectra with 5% damping ratio illustrating that the selected earthquake ground motion records are well describing the medium to strong intensity earthquake motion histories.

3 FINITE ELEMENT ANALYSIS OF THE BRDIGE

3.1 *Characteristics of Damage State*

In the seismic fragility analysis, different forms of engineering demand parameters (EDPs) are used to monitor the structural responses under earthquake ground motion and measure the damage state (DS) of the bridge components. Damage states for bridges should be defined in such a way that each damage state indicates a particular level of bridge functionality. A capacity model is needed to measure the damage of bridge component based on prescriptive and descriptive damage states in terms of EDPs (Choi et al., 2004; Neilson, 2005). Four damage states as defined by Federal Emergency Management Authority through HAZUS (FEMA, 2000) are commonly adopted in the seismic vulnerability assessment of engineering structures, namely slight, moderate, with extensive and collapse damages. Bridge piers are one of the most critical components, which are often forced to enter into nonlinear range of deformations under strong earthquakes. In this study, the displacement ductility of the bridge pier is adopted as damage index (DI). Hwang et al., 2001 recommended four different damage states for bridge pier (Table 2) based on ductility limit. But retrofit affects the seismic response and demand of the bridge pier and the capacity as well. For the retrofitted bridge pier new limit states need to be defined. Limit states capacities for all the two retrofitted bridge bent are obtained by transforming the ductility limit state proposed by Hwang et al., (2001) shown in Table 2. The use of ductility limit for retrofitted RC columns is well documented in literature (Ramanathan et al., 2012 and Billah and Alam, 2012).

Table 2. Damage/limit state of bridge components (Hwang et al., 2001; Ramanathan et al., 2012; Billah and Alam, 2012)

Damage State	⟶	Slight (DS=1)	Moderate (DS=2)	Extensive (DS=3)	Damage ((DS=4)
Bridge Component	Physical Phenomenon	Cracking and spalling	Moderate cracking and spalling	Degradation without collapse	Failure leading to collapse
As Built Bridge Pier	Displacement Ductility,	$\mu_d > 1$	$\mu_d > 1.2$	$\mu_d > 1.76$	$\mu_d > 4.76$
CFRP Retrofitted Pier	Displacement Ductility,	$\mu_d > 1.81$	$\mu_d > 3.89$	$\mu_d > 6.48$	$\mu_d > 12.96$
RCC Jacketing Retrofitted Pier	Displacement Ductility,	$\mu_d > 1.72$	$\mu_d > 3.7$	$\mu_d > 6.19$	$\mu_d > 12.32$

3.2 *Incremental Dynamic Analysis*

In Incremental Dynamic Analysis (Vamvatsikos and Cornell, 2002; Alim, 2014), the structure is subjected to a series of non-linear time-history analysis of the increasing intensity (e.g. Peak ground motion acceleration is incrementally scaled from a low elastic response value up to the attainment of a pre-defined post-yield target limit state). Incremental Dynamic Analysis (IDA) is a new methodology which can give a clear indication of the relationship between the seismic capacity and the demand. The analysis was carried out for the as-built and retrofitted concrete bridge bent. The peak values of base shear are then plotted against their top displacement counterparts, for each of the dynamic runs, giving rise to the so-called dynamic pushover or IDA envelop curves.

3.3 *Fragility Curve Development*

Fragility curve allows the evaluation of potential seismic risk assessment of any structure. Fragility function describes the conditional probability i.e. the likelihood of a structure being damaged beyond a specific damage level for a given ground motion intensity measure. The fragility or conditional probability can be expressed as,

$$\text{Fragility} = P\,[LS|IM=y]; \qquad (1)$$

where, LS is the limit state or damage state of the structure or structural component, IM is the ground motion intensity measure and y is the realized condition of the ground motion intensity measure. In order to develop fragility curves different methods and approaches have been developed. Depending on the available data and resources, fragility functions can be generated empirically based on post-earthquake surveys and observed damage data from past earthquakes (Basoz and Kiremidjian, 1999; Yamazaki et al., 1999). However, limited damage data and subjectivity in defining damage states limit the application of empirical fragility curves (Padgett and DesRoches, 2008). In absence of adequate damage data, fragility functions can be developed using a variety of analytical methods such as elastic spectral analyses (Hwang et al., 2001), nonlinear static analyses (Shinozuka et al., 2000) and nonlinear time-history analyses (Alam et al. 2012; Bhuiyan and Alam 2012; Hwang et al., 2001; Choi et al., 2004).

Two approaches are used to develop the PSDM): the scaling approach and the cloud approach (Alam et al. 2012; Bhuiyan and Alam 2012. In the scaling approach, all the ground motions are scaled to selective intensity levels and an incremental dynamic analysis (IDA) is conducted at each level of intensity; however, in the cloud approach, un-scaled earthquake ground motions are used in the nonlinear time-history analysis and then a probabilistic seismic demand model is developed based on the nonlinear time history analyses results. In the current study, the cloud method was utilized in evaluating the seismic fragility functions of the retrofitted bridge bents. In the cloud approach, a regression analysis is carried out to obtain the mean and standard deviation for each limit state by assuming the power law function (Cornell et al., 2002), which gives a logarithmic correlation between median EDP and selected IM.

In this study probabilistic seismic demand model (PSDM) was used to derive the analytical fragility curves using nonlinear time-history analyses of the retrofitted bridge bents. Although this is the most rigorous method, yet this is the most reliable analytical method (Shinozuka et al., 2000). The PSDM establishes a correlation between the engineering demand parameters (EDP) and the ground intensity measures (IM). In the current study, displacement ductility demand of retrofitted bridge bent was considered as the EDP, and the peak ground acceleration (PGA) was utilized as intensity measure (IM) of each ground motion record. In this study, probabilistic seismic demand models (PSDM) are used to derive the fragility curves. The ground motions are scaled to selective intensity levels and an incremental dynamic analysis (IDA) is conducted at each level of the intensity. A regression analysis is carried out to obtain the mean and standard deviation for each limit state by assuming the power law function (Cornell et al., 2002) , which gives a logarithmic correlation between median EDP and selected IM:

$$EDP = a\,(IM)^b\,; \qquad (2a)$$

$$\text{or,} \quad \ln(EDP) = \ln(a) + b\,\mathrm{Ln}\,(IM); \qquad (2b)$$

where, a and b are unknown coefficients which can be estimated from a regression analysis of the response data collected from the nonlinear time history analysis. In order to create sufficient data for the cloud approach incremental dynamic analysis is carried out instead of nonlinear time history analysis. The dispersion of the demand, $\beta_{EDP|\,IM}$, conditional upon the IM can be estimated from Eq. (3),

$$\beta_{EDP|IM} = \sqrt{\frac{\sum_{i=1}^{N}(\ln(EDP) - \ln(aIM^b))^2}{N-2}}\,; \qquad (3)$$

With the probability seismic demand models and limit states corresponding to various damage states, it is now possible to generate the fragilities using Eq. (3),

$$P[LS|IM] = \varphi\left[\frac{\ln(IM) - \ln(IM_n)}{\beta_{comp}}\right]; \qquad (4)$$

$$\ln(IM_n) = \frac{\ln(S_c) - \ln(a)}{b}\,; \qquad (5)$$

ln(IMn) is defined as the median value of the intensity measure for the chosen damage state (i.e., slight, moderate, extensive and collapse) a and b are the regression coefficients of the PSDMs and the dispersion component is presented in Eq. (6),

$$\beta_{comp} = \frac{\sqrt{\beta_{EDP|IM} + \beta_c^2}}{b}\,; \qquad (6)$$

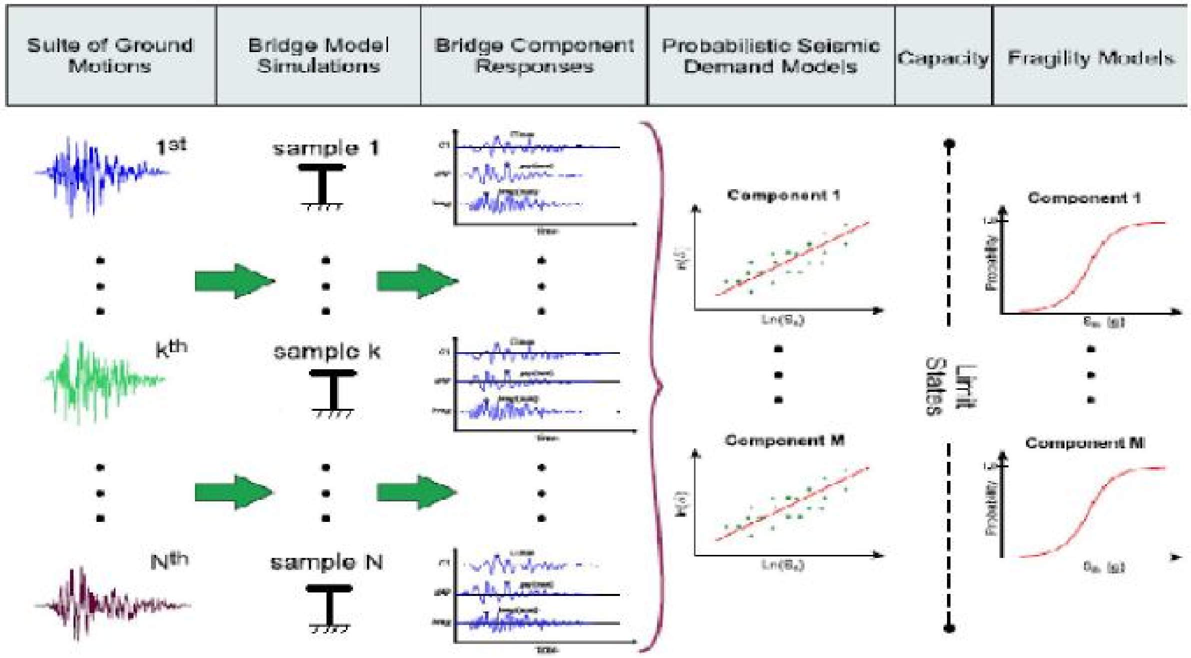

Figure 4. Fragility curve development

where, S_c is the median and β_c is the dispersion value for the damage states of the bridge pier. The dispersion coefficient β_c is used as describe by Ramanathan et al., (2012). The steps of fragility curve development shown in Figure 4 (Alim, 2014).

4 ANALYTICAL RESULTS

PSDM of the as-built bridge pier is shown in Figure 5. The parameters of the PSDM for the as-built and retrofitted bridge are given in Table 3.

Table 3. PSDM parameter for two type of bridge pier

PSDM parameters			
Pier Condition	ln (a)	b	$\beta_{EDP\|IM}$
As-built	1.50	1.19	0.47
FRP Retrofitted	0.98	0.954	0.43
Concrete Jacketed Retrofitted	1.36	1.19	0.45

Plots of the fragility curves for four damage states are shown in Figures 6 to 9 illustrating the relative vulnerability of the retrofitted bridge bents over the as-built bridge bent at each PGA level of ground motion records. In each figure, three PGA levels, such as 0.15g, 0.28g and 0.36g, have been highlighted to state the probability of exceeding a certain damage state before and after retrofitting of the bridge bent. The PGA of 0.15g corresponds to a design earthquake (DE) in and around Chittagong city, the location of the subject bridge bent, having 20% probability of exceedance in 50 years whereas the PGA of 0.28g corresponds to a maximum credible earthquake (MCE) in and around Chittagong city having 2% probability of exceedance in 50 years for which the return period is approximately 2475 years (BNBC 2006, 2015). The PGA of 0.36g corresponds to a maximum credible earthquake (MCE) in and around Sylhet city (the most seismically active zone in Bangladesh) having 2% probability of exceedance in 50 years for which the return period is approximately 2475 years (BNBC 2006, 2015). From the fragility curves presented in Figures 6 to 9 it is revealed that the as-built bridge bent experiences a higher damage than the retrofitted ones in each of the three PGA levels. Moreover, the bridge bent retrofitted with FRP has shown better seismic performance than the concrete jacketed bridge bent in each PGA level.

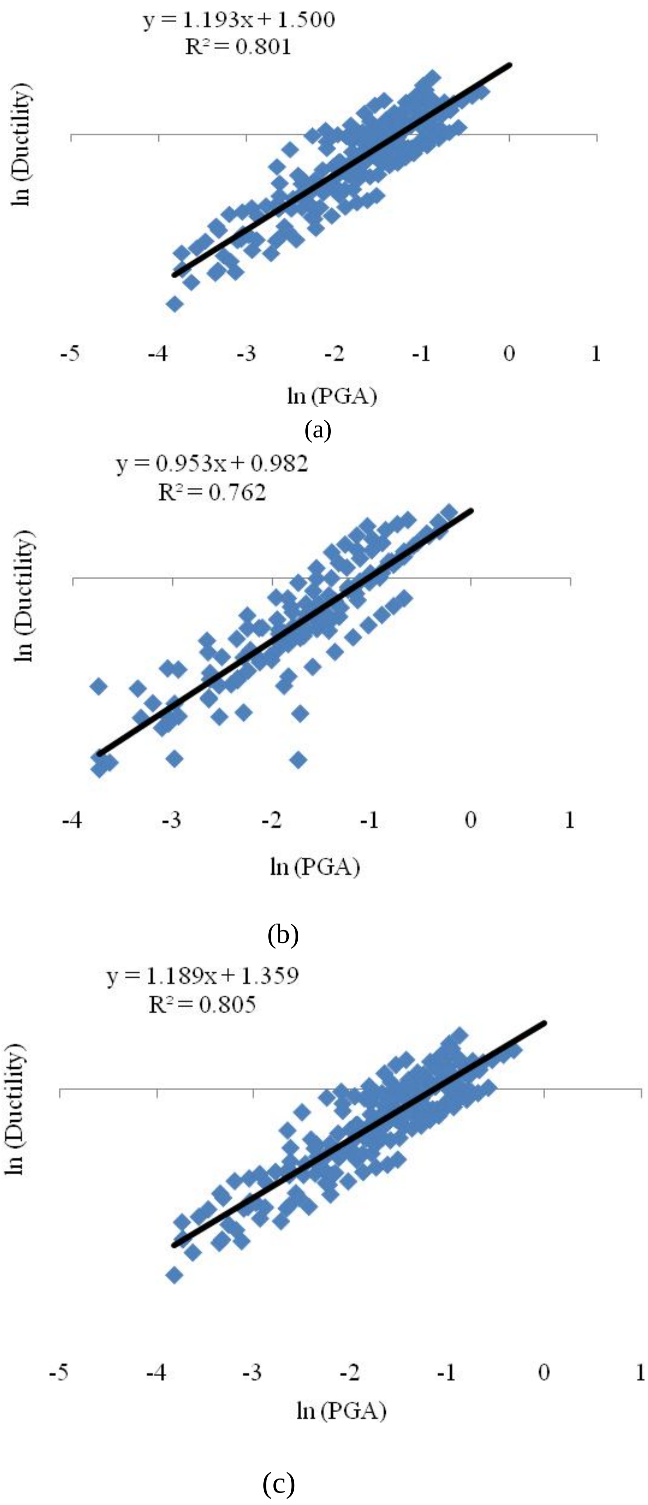

Figure 5. PSDM of concrete pier (a) As-built (b) FRP Retrofitted (c) Concrete Jacketed

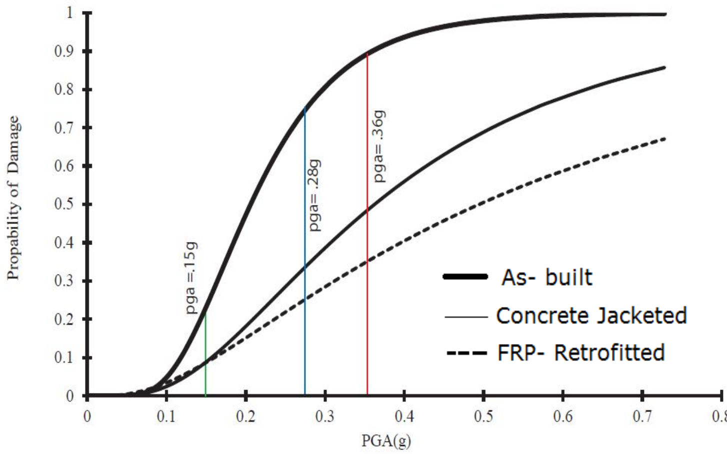

Figure 6. Fragility curves of the bridge bent for slight damage

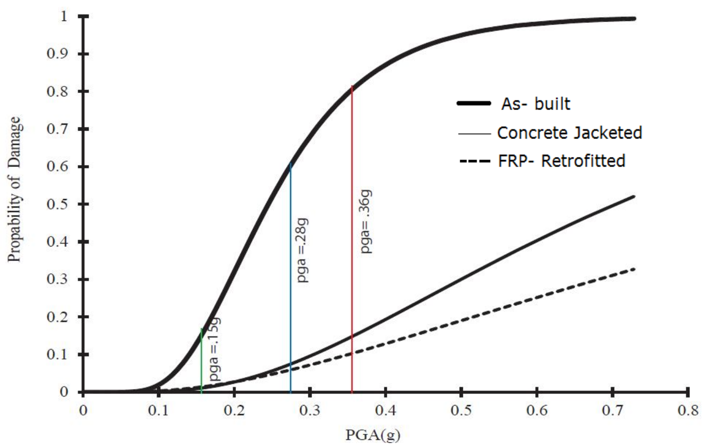

Figure 7. Fragility curves of the bridge bent for moderate damage state

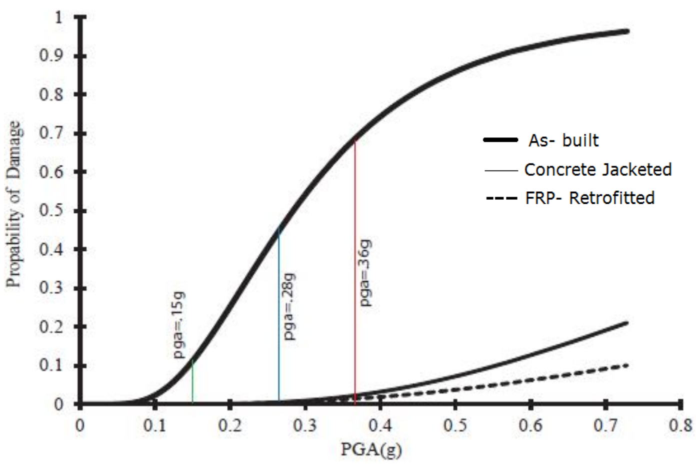

Figure 8. Fragility curves of the bridge bent for extensive damage state

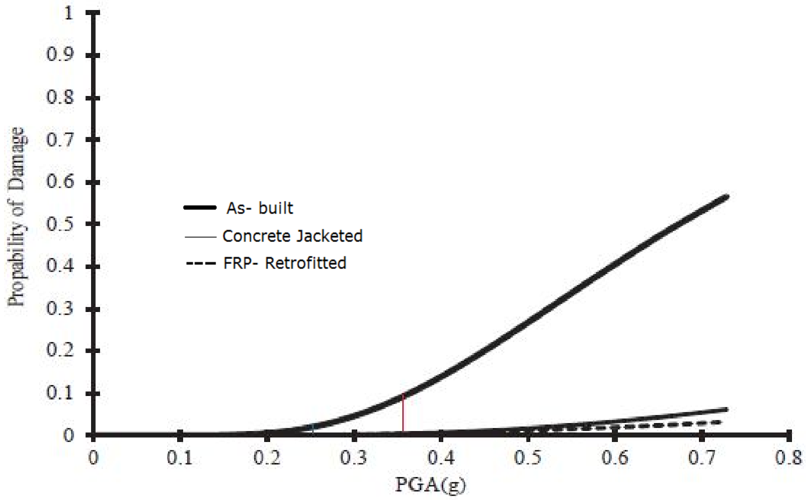

Figure 9. Fragility curves of the bridge bent for collapse damage state

It can be more specifically stated that the retrofitted bridge bent carries no major damage (i.e the extensive and collapse damage states) under the MCE earthquakes (i.e.the earthquakes having PGA of 0.28g and 0.36g); however, the as-built bridge bent shows significant damage states under these two earthquakes.

5 CONCLUSIONS

This study focuses on reliability based seismic performance analysis of an as-built and retrofitted bridge bent using the analytical fragility curves under far field earthquake ground motion records. In this regard, the method of probabilistic seismic demand model (PSDM) is used to derive the analytical fragility curves using nonlinear time-history analyses of the bridge bent. The analytical results, in general, show that the as-built bridge bent is more susceptible to seismic damage under design and maximum credible earthquakes than the retrofitted bridge bents. The FRP jacketed bridge bent shows a better seismic performance than the concrete jacketed bridge bent. More specifically the as-built bridge bent experiences at least 10% and 40% extensive damage, respectively, under the design and maximum credible earthquakes whereas the retrofitted bridge bent experiences almost no or very insignificant damage. The seismic vulnerability of the as-built bridge bent can be significantly reduced by applying a proper retrofitting technique. The fragility curves as obtained for the bridge bent can be used to estimate the potential losses incurred from earthquakes and it will help post-earthquake rehabilitation decision making, and hence selection of suitable retrofits techniques.

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