Int. J. Struct. & Civil Engg. Res. 2013

International Journal of Structural and Civil Engineering Research

ISSN 2319 – 6009 www.ijscer.com
Vol. 2, No. 2, May 2013

Research Paper

EVALUATION OF PROBABILISTIC FAILURE OF BRIDGE PIER

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Bridge is an essential lifeline structure, which provides an emergency link in transportation network system. During post-earthquake bridge should remain functioning without any collapse. But in reality, uncertainty behavior of civil engineering structures involve during service life since, actual and assumed environmental conditions during design are never coincide. The purpose of this paper is to evaluate the probabilistic failure of bridge pier subjected to far field ground motion. A nonlinear static pushover and incremental dynamic time-history analysis have been performed using the SeismoStruct nonlinear analysis program for 3D bridge bent. 20 far field ground motions and their respective PGA are considered to develop fragility curve for conventional and FRP retrofitted bridge pier. The purpose of this study is to probabilistic determination of seismic vulnerability of bridge pier and help to decision making for effective retrofitting technique.

Keywords: Fragility curve, Probabilistic failure, Incremental dynamic analysis, PSDM

INTRODUCTION

Bridges are considered to be lifeline structures, since they provide an emergency link in a surface transportation network during disaster, such as Earthquakes. Hence bridges are required higher seismic performance than others civil engineering structures. The failure of such structures causes claim of lives, economical loss including immeasurable sufferings. The exact performance evaluation of bridge under environmental action like wind, earthquake loading became uncertain since such loadings action is unknown. The use of fragility curves establishes a relationship between ground shaking intensity and the probability of reaching or exceeding a certain response level. For assessment of seismic losses due to earthquake, fragility, fragility curves become valuable tool for pre-earthquake disaster planning and post-earthquake recovery and retrofitting programs.

FINITE ELEMENT MODELING

The analytical model of the bridge bent is approximated as a continuous 2-D finite element frame using the SeismoStruct

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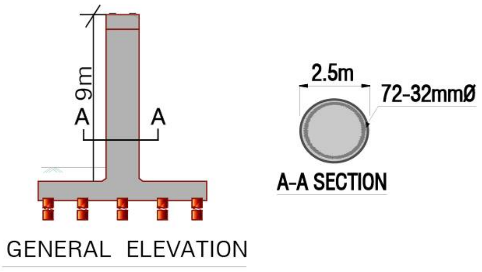
nonlinear analysis program (SeismoStruct 6). Nonlinear static pushover and incremental dynamic time-history analysis have been performed on the bridge bents to determine the performances of the retrofitted bridge bents. The program has the ability to figure out large displacement behavior and the collapse load of structure under either static or dynamic loading, while taking into account both geometric nonlinearities and inelasticity (Baker and Cornell, 2006). 3D inelastic beam elements have been used for modeling the pier. The elevation and section of the pier is shown in Figure 1.

Figure 1: Elevation and Section of Pier Model



The fiber modeling approach has been employed to represent the distribution of the material nonlinearity along the length and cross sectional area of the member. Each fiber has stress-strain relationship, which can be specified to represent unconfined concrete, confined concrete, and longitudinal steel reinforcement. The confinement effect of the concrete section is considered on the basis of reinforcement detailing. The distribution of inelastic deformation and force is sampled by specifying cross-section slices along the length of the element. Twenty far field ground motions and their respective Peak Ground Acceleration (PGA) are considered to develop fragility curve.

To develop the analytical model Menegotto-pinto steel model (Choi *et al.,* 2004) with Flippou *et al.* (1983) isotropic strain hardening property is used for reinforcing steel material. The yield strength, strain hardening parameter and modulus of elasticity of steel are considered as 400 MPa, 0.5% and 2*10^5 Mpa, respectively. For concrete non linear variable confinement model of Madas and Elnashai (1992) with compressive strength of 21 MPa and tensile strength 3.5 MPa has been used. CFRP has been modeled using non linear FRP confined concrete model developed by Ferracuti and Savoia (2005). For compression, this model follows the constitutive relationship and cyclic rules proposed by Mander *et al.* (1999), and those follows the constitutive and Reinhardt (1989), for tension. FRP confined concrete model proposed by Spoestra and Monti (1999) have been employed to model the effect of the confinement introduced by the FRP wrapping.

EARTHQUAKE GROUND MOTION

Twenty Far field ground motions are used in the analysis listed in Table 1. In this study PGA is considered as the Intensity Measure (IM) for it's efficacy, utility and adequacy in vulnerability assessment. The PGA of the ground motions range from PGA 0.22 to PGA 0.728. In Figure 2 spectral acceleration with 5% damping is shown.

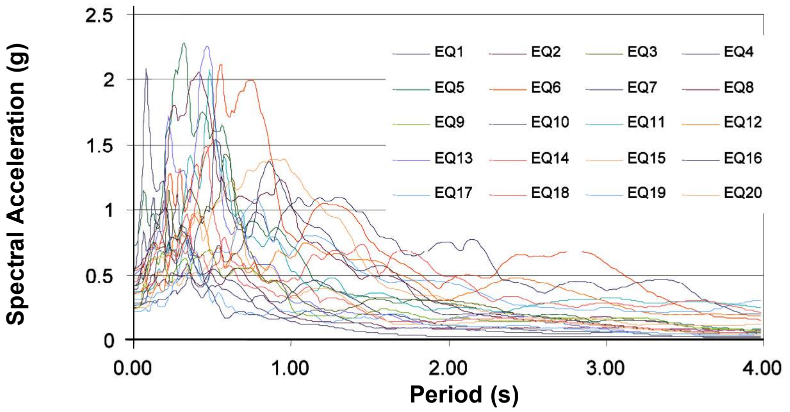
CHARACTERISTICS OF DAMAGE STATE

The probability of entering a damage state an input ground motion intensity parameter is expressed by fragility curves. Different forms of Engineering Demand Parameter (EDPs) are used to measure the DS of the bridge

Table 1: Characteristics of Far Field Ground Motion Histories

Eq. No.	Earthquake Name	Recording Station	Epicentral Distance (km)	PGA$_{max}$ (g) (cm/s^2)	PGV$_{max}$ (cm/s)
1	Northridge	Beverly Hills - Mulhol	13.3	0.416	58.95
2	Landers	Yermo Fire Station	86	0.24	51.5
3	Northridge	Canyon Country-WLC	26.5	0.41	42.97
4	Landers	Coolwater	82.1	0.283	26
5	Duzce, Turkey	Bolu	41.3	0.728	56.44
6	Loma Prieta	Capitola	9.8	0.53	35
7	Hector Mine	Hector	26.5	0.266	28.56
8	Loma Prieta	Gilroy Array #3	31.4	0.56	36
9	Imperial Valley	Delta	33.7	0.238	26
10	Manjil, Iran	Abbar	40.4	0.51	43
11	Imperial Valley	El Centro Array #11	29.4	0.364	34.44
12	Superstition Hills	El Centro Imp. Co.	35.8	0.36	46.4
13	Kobe, Japan	Nishi-Akashi	8.7	0.51	37.28
14	Superstition Hills	Poe Road (temp)	11.2	0.45	35.8
15	Kobe, Japan	Shin-Osaka	46	0.24	38
16	Cape Mendocino	Rio Dell Overpass	22.7	0.385	43.8
17	Kocaeli, Turkey	Duzce	98.2	0.312	59
18	Chi-Chi, Taiwan	CHY101	32	0.353	70.65
19	Kocaeli, Turkey	Arcelik	53.7	0.22	17.69
20	Chi-Chi, Taiwan	TCU045	77.5	0.474	36.7

Figure 2: Spectral Acceleration of Ground Motion



components. Based on energy dissipation capacity and ductility demand of structure, Park and Ang (1985) developed a damage index while Hwang *et al.* (2000) used the capacity/demand ratio of the bridge columns as EDP to develop fragility curves. A capacity model to measure the damage of bridge components based on prespective and descriptive damage states in terms of EDPs Hwang *et al.* (2000). This study on fragility analysis of bridge used the displacement ductility as damage measure. Hwang *et al.* (2000) used the capacity/demand ratio as the bridge pier to develop fragility curves. The damage states are presented in Table 2.

ANALYTICAL FRAGILITY CURVES

Fragility is modelled by a lognormal cumulative distribution function where the structural demand and capacity are assumed to be lognormally or normally distributed. In this study, probabilistic seismic demand models 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*:

$$EDP = a(IM)^b \text{ or, } \ln(EDP) = \ln(a) + b\ln(IM) \qquad \text{...(1)}$$

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 Equation (2) (Karthik *et al.,* 2012).

$$\beta_{EDP|IM} = \sqrt{\frac{\sum_{i=1}^{N}\left(\ln(EDP) - \ln\left(aIM^b\right)\right)^2}{N-2}} \qquad \text{... (2)}$$

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

$$P[LS \mid IM] = \varphi\left[\frac{\ln(IM) - \ln(IM_n)}{\beta_{\text{comp}}}\right] \qquad \text{... (3)}$$

$$\ln(IM_n) = \frac{\ln(S_c) - \ln(a)}{b} \qquad \text{... (4)}$$

Table 2: Damage State

Damage State		**Slight (DS=1)**	**Moderate (DS=2)**	**Extensive (DS=3)**	**Collapse (DS=3)**	**Reference**
Bridge Component	Physical Phenomenon	Cracking and Spalling	Moderate Cracking and Spalling	Degradation without Collapse	Failing Leading to Collapse	
Bridge Pier	Displacement Ductility	$\mu d>1.0$	$\mu d>1.2$	$\mu d>1.76$	$\mu d>4.76$	Hwang *et al.* (2001)

ln(IM_n) is defined as the median value of the intensity measure for the chosen damage state (slight, moderate, extensive, collapse), a and b are the regression coefficients of the PSDMs and the dispersion component is presented in Equation (5) (Madas and Elnashai , 1992 and Mander, 1999).

$$\beta_{comp} = \sqrt{\frac{\beta_{EDP}\,|\,IM + \beta_c^2}{b}} \qquad ...(5)$$

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 Karthik Ramanathan *et al.* (2012).

RESULT EVALUATION

PSDM of two type of bridge pier are shown in Figures 3 and 4. The impact of two different measures under far field earthquake ground motions on the demand models is compared in Table 3.

Plots of the fragility curves for two cases are shown in Figures 5 to 8, which illustrated relative vulnerability of the retrofitted bridge bents over a range of far field Earthquake intensities and damage states. From figures

Figure 3: PSDM of As-Build Concrete Pier

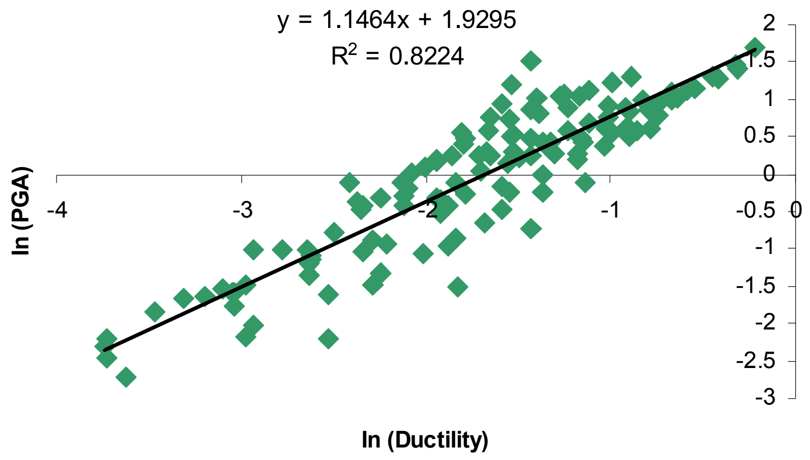


Figure 4: PSDM of Concrete Pier Retrofitted by FRP

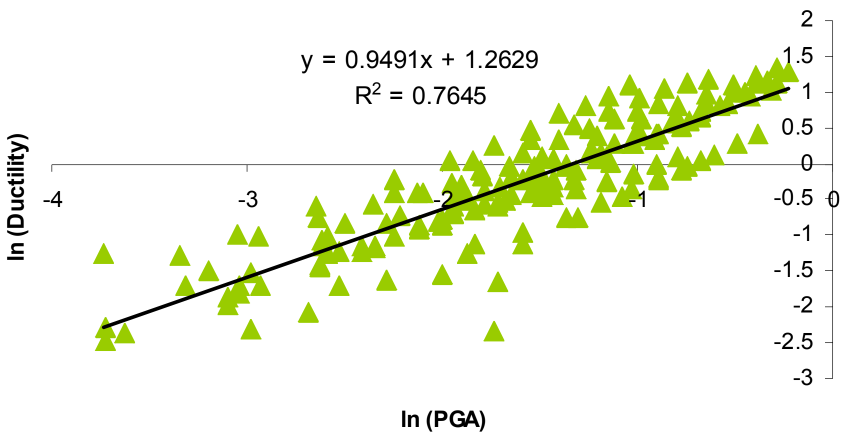


Table 3: PSDM for two types of Bridge Pier

Pier Condition	Column Ductility		
	ln (*a*)	*b*	$\beta_{EDP\|IM}$
As-built	1.15	1.92	0.43
Retrofit by FRP	0.95	1.26	0.43

Figure 5: Fragility Curve for Slight Damage

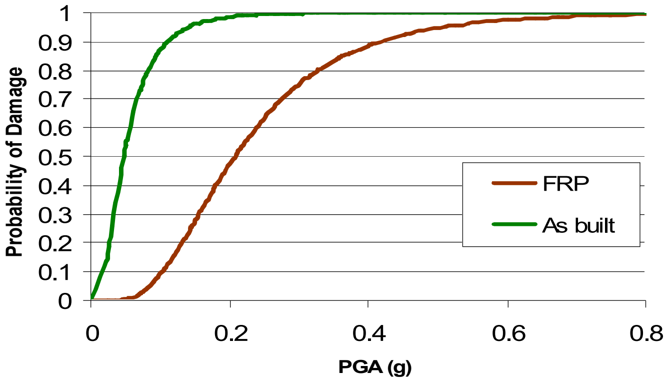


Figure 6: Fragility Curve for Moderate Damage

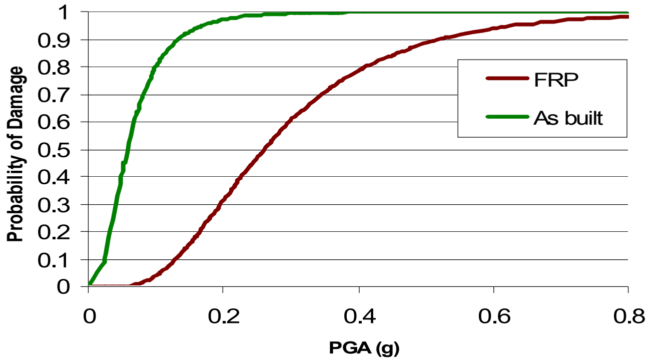


Figure 7: Fragility Curve for Extensive Damage

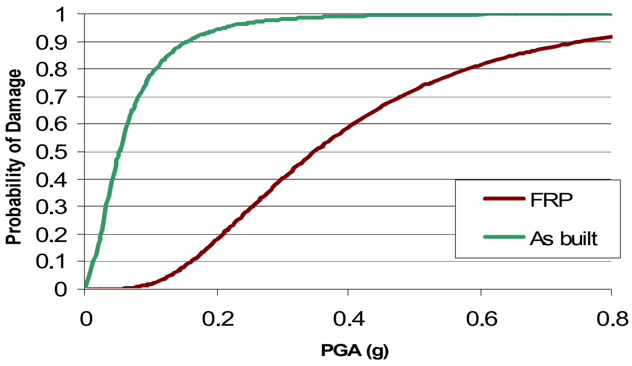
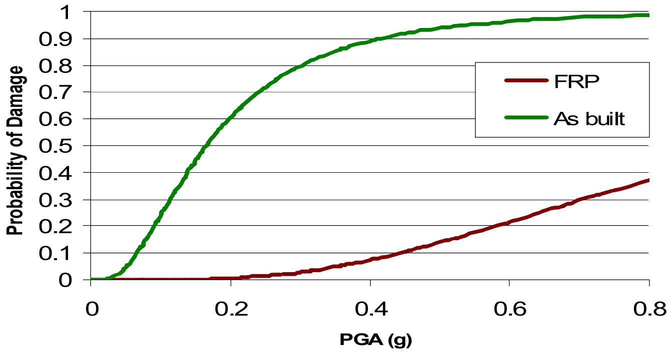


Figure 8: Fragility Curve for Damaged



it is evident that retrofitted measure by FRP is effective for different damage states in terms of reducing the probability of the damage for a given PGA.

CONCLUSION

This study evaluates the seismic fragility of single concrete pier both in as built condition and with retrofitting stage. To investigate seismic vulnerability of the bridge pier 20 near field earthquake ground motion are utilized to evaluate likelihood of exceeding the seismic capacity of the bridge pier. The result indicates that the retrofitted with FRP posses less vulnerability at all damage states under far field earthquake.

REFERENCES

1. Baker J and Cornell A C (2006), "Which Spectral Acceleration Are You Using?", *Earthquake Spectra*, Vol. 22, No. 2.
2. Choi E, DesRoches R and Nielson B (2004), *Seismic Fragility of Typical Bridges in Highway.*
3. Ferracuti B and Savoia M (2005), "Cyclic Behaviour of Frp-wrapped Columns Under Axial and Flexural Loadings", *Proceedings of the International Conference on Fracture*, Turin, Italy.
4. Filippou F C, Popov E P and Bertero V V (1983), "Effects of Bond Deterioration on Hysteretic Behaviour of Reinforced Concrete Joints," Report EERC 83-19, Earthquake Engineering Research Center, University of California, Berkeley.
5. Hwang H, Liu J B and Chiu Y H (2000), Seismic Fragility Analysis of Highway.
6. Karthik Ramanathan, Reginald DesRoches and Jamie E Padgett (2012), "A Comparison of Pre-and Post-seismic Design Considerations in Moderate Seismic Zones Through the Fragility Assessment of Multispan Bridge Classes".
7. Madas P and Elnashai A S (1992), "A New Passive Confinement Model for Transient Analysis of Reinforced Concrete Structures," *Earthquake Engineering and Structural Dynamics,* Vol. 21, pp. 409-431.
8. Mander J B (1999), "Fragility Curve Development for Assessing the Seismic

Vulnerability of Highway Bridges", Report No. 99-SP01, MCEER.

9. Menegotto M and Pinto P E (1973), "Method of Analysis for Cyclically Loaded R C Plane Frame Including in Geometryand Non-Elastic Behavior of Elements Under Combined Normal Force and Bending, Symposium on the Resistance and Ultimate Deformability of Structures Acted on by Well Defined Repeated Loads", *International Association for Bridge and Structural Engineering*, pp. 15-22, Zurich, Switzerkabd.
10. Nielson B (2005), "Analytical Fragility Curves for Highway Bridges in Moderate Seismic Zones", Ph.D. Thesis, Georgia Institute of Technology.
11. Nielson B (2005). "Personal Communication: Analytical Fragility Curves for Highway Bridges in Moderate Seismic Zones".
12. Park Y J and Ang A H S (1985), "Mechanistic Seismic Damage Model for Reinforced Concrete", *ASCE Journal of Structural Engineering,* Vol. 111, No. 4, pp. 722-39.
13. Pinho R, Casarotti C and Antoniou S (2007), "A Comparison of Single-run Push Over Analysis Technique for Seismic Assessment of Bridges", *Earthquake Engineering Structural. Dynamics*, Vol. 36, pp. 1347-1362.
14. Spoelstra M and Monti G (1999), "FRP-Confined Concrete Model," *Journal of Composites for Construction, ASCE*, Vol. 3, pp. 143-150.
15. Vamvatsikos D and Cornell C A (2002), "Incremental Dynamic Analysis", *Earthquake Engineering and Dynamics*, Vol. 31, No. 3, pp. 491-514.
16. Yankelevsky D Z and Reinhardt H W (1989), "Uniaxial Behaviour of Concrete in Cyclic Tension," *Journal of Structural Engineering, ASCE*, Vol. 115, No. 1, pp. 166-182.