AN APPLICATION OF SEISMIC FINANCIAL RISK ASSESSMENT OF THREE-SPAN CONTINUOUS RIGID FRAME BRIDGE

Kai Zhang[1] Xi Zhu[2] and Cheng Jiang[3]

[1] *Ph.D. candidate, School. of Civil Engineering, Beijing Jiaotong University, Beijing. China*
[2] *Professor, School. of Civil Engineering, Beijing Jiaotong University, Beijing. China*
[3] *Graduate student, School of Civil Engineering, Beijing Jiaotong University, Beijing, China*
Email: kengawk@hotmail.com, xzhu@bjtu.edu.cn

ABSTRACT :
Incremental dynamic analysis (IDA) is an effective and precise procedure to carry out seismic risk assessment to investigate expected structural responses, seismic vulnerability and financial loss to various types of structures. The illustrated example is a rigid frame reinforced concrete bridge as a key railway pivot designed to the draft code for seismic design of railway engineering. A quantitative risk analysis procedure is conducted, including selecting a suitable suite of site-dependent ground motions, performing IDA on a nonlinear model of the prototype structure, organizing and parameterizing the IDA results into different damage states in a probabilistic format. The financial risk assessment can be estimated to predict direct financial loss in dollars and help to select a reasonable method of repair that can restore the specific functionality and easily be comprehended by either engineers or facilities owners. This paper extends a probabilistic risk assessment methodology to quantify expected annual financial loss for the rigid frame reinforced concrete bridge, correlating the systematic seismic capacity and demand to financial risk. The result shows that the railway bridge designed to design basic earthquake may face up to more financial loss statistically than the loss caused by maximum considered earthquake events for the occurrence probability of the latter is too lower in the local region. The result suggests that facility owners and managers may reduce the seismic financial risk with selecting proper retrofit strategies against the minor and moderate earthquakes with a relative low collapse probability.

KEYWORDS:
rigid frame railway bridge, incremental dynamic analysis, seismic financial risk, damage state, seismic vulnerability

1. INTRODUCTION

The exact extent of damages is extremely difficult to predict for the uncertainties invariably exist in forecasting the likelihood of the earthquake damages. Therefore, a more rational approach is required to take into account all uncertainties from seismic demand and structural capacity, giving assurance to the users regarding the level of confidence or reliability. In this paper, the seismic financial risk to a rigid frame bridge in Southwest China is quantified, which designed to the latest seismic code to railway is engineering. And a proper approach to perform the seismic financial risk assessment is explored under the background of performance–based earthquake engineering (PBEE).

Cornell et al (2002) proposed a power-law equation for the median curve generated from a series of nonlinear time-history analysis, which gives the linear relationship between the rate of exceeding an engineering demand parameter (EDP) and recurrence or annual frequency in log-log scale, which leads to more rigorous researches world widely focus on the limitations and applications of this formula. Vamvatsikos and Cornell (2002) developed incremental dynamic analysis (IDA) that gives a clear indication of the relationship between the seismic capacity and seismic demand. Mander and Dhakal et al (2006) integrate the scenario losses over the entire range of occurrence probability and quantify the seismic risk in term of an expected annual loss (EAL)， incorporating a range of seismic scenarios, return rate, and expected damage into a single mean dollar loss.

Pagni and Lowes (2006) identified five methods of repair to restore a component to its pre-earthquake condition and Brown et al (2007) developed fragility function to predict the method of repair (MOR) required for modern reinforced-concrete beam-column joints subject to earthquake loading. This paper attempts to establish the links among these researches to have a comprehensive understanding of the seismic financial risk assessment.

2. IDA-BASED SEISMIC RISK ASSESSMENT

The Pacific Earthquake Engineering Research (PEER) Center's probabilistic framework expressed in triple integral equation disaggregates the whole process of seismic assessment into four stage analyses: seismic hazard analysis, structural analysis, damage analysis and decision making analysis according to the total probability theorem (Cornell and Krawinkler 2000), which successfully describes sources of randomness and uncertainty from structures and earthquake events in several interim probabilistic models. And Mander et al (2006) propose the EAL framework to quantify the seismic financial risk, using IDA procedure and taking into account the probability of exceeding a loss ratio under a damage measure G(Lr|dm) in the financial risk assessment as below:

$$EAL = \iiint L_r dG(L_r \mid dm) \mid dG(dm \mid edp) \| dG(edp \mid im) \| df_a(im) \| \tag{2.1}$$

Where $f_a(\cdot)$ is the annual rate of exceeding $(\cdot)$, im is the intensity measure (IM) (e.g. peak ground motion (PGA), edp is the engineering demand parameter (EDP) (e.g. the maximum section rotation), dm is the damage measure (e.g. spalling, bar buckling and collapse), dv is the decision variable (e.g. MOR, downtime); G(x|y)=P(x>X|y=Y) the conditional complimentary cumulative distribution (CCDF), L_r is the loss ratio defined as the cost to repair a structure divided by the total replacement cost.

2.1 The Selection of Ground Motion Records and the Estimation of the Hazard-recurrence Parameters

Earthquake events are the source of an aleotory uncertainty, which almost cannot be changed and usually can be considered following lognormal distribution. It is necessary to place emphasis on the determination of the inherent record-to-record randomness of earthquake event in term of the coefficient of variation β_D to an IM, such as PGA or S_a in Fig. 1.

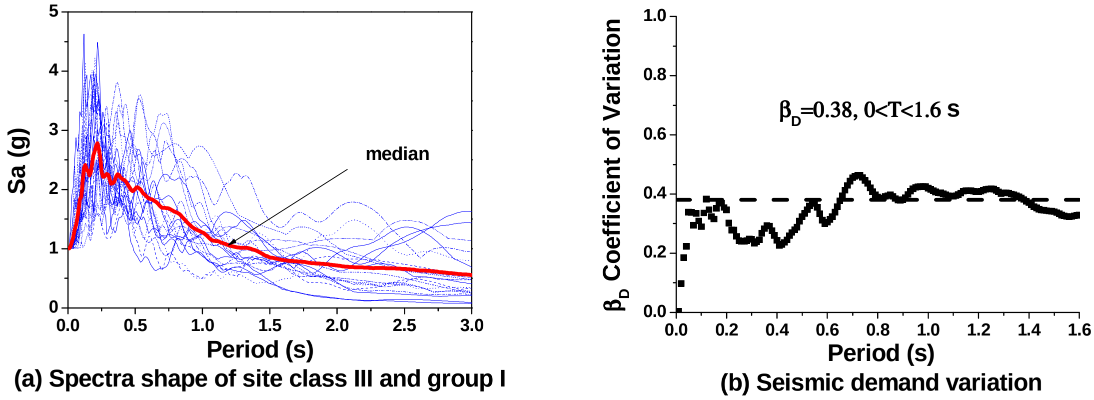


Figure 1 Ground motion records normalized to dynamic amplification factor

FEMA350 (2000) and Cornell et al (2002) suggested an approximate seismic hazard-recurrence relationship:

$$f_a(im) = k_1(im)^{-k_2} \tag{2.2}$$

Where k_1 and k_2 are constants determined by two level acceleration values of design basis earthquake (DBE) and maximum considered earthquake (MCE). f_a=1/T_r, T_r is return period. The f_a-T_r relation ignores the error when f_a is relative big (Ang and Tong 1975).

2.2. Perform Incremental Dynamic Analysis

Incremental dynamic analysis (IDA) is a parametric analysis method to estimate structural performance under seismic loads more precisely. It involves performing lots of dynamic analyses with more ground motion records, each scaled to multiple levels of IM. Usually the nonlinear time history analysis is conducted on a nonlinear computational model of the prototype bridge until the result curve turns out dynamic unstable, which indicates the structural collapse. And a suite of records in maximum response parameter versus intensity level generated shows the seismic capacities of the bridge under different seismic demands.

2.3. Model the IDA Percentile Curves for Deformation Hazard Curve

Epistemic uncertainty mainly exists in the modeling procedure. Based on the results of IDA curves, the IDA 50% percentile curve could be used to form the deformation hazard curve with nonlinear least square technique, instead of using R-O equation (Mander et al 2006), if there exists significant correlation between IM (e.g. PGA) and deformation (e.g. the maximum section rotation θ_{max}). And to encompass the randomness of seismic demand along with the structural capacity, together with the uncertainty to the model error, it is reasonable to use the composite value of the lognormal coefficient of variation suggested by Kennedy et al (1980):

$$\beta_{com} = \sqrt{\beta_C^2 + \beta_D^2 + \beta_U^2} \tag{2.3}$$

Where β_C is the lognormal standard deviation for the structural capacity; β_D is the lognormal standard deviation for the seismic demand; β_U is the lognormal standard deviation for modeling uncertainty. β_C and β_U are suggested to be 0.2 and 0.25 in FEMA350 (2000). In this study the composite lognormal coefficient of variation is equal to 0.5, for β_D is 0.38 from the 20 ground motion records.

2.4. Determine the Damage States and Corresponding Financial Loss

This study links the five damage states defined comprehensively in repair and downtime by Mander and Basoz (1999) to the MOR strategies provided by Brown et al (2007) in table 2.1. The damage states are described in MORs: in DS1 structure represents elastic behaviors mainly and no further repair needed (MOR0); the DS2 means the damage come out with minor crack of the cover concrete and can be inspected, adjusted or patched with cracks injection with epoxy (MOR1); in DS3 the damage can be repaired with patching spalled concrete, injecting crack with epoxy or removing post-spalling bar (MOR2). With the damage developed further, the damaged concrete have to be replaced (MOR3) until arriving at DS5, which means replacement entirely (MOR4) for the function losses due to excessive permanent drift or excessive damages to critical components. The cost of MOR3 is usually almost equal to that of MOR4, sometimes even more expensive in the loss ratio L_r shown in table 2.1 (Dhakal et al 2006). The damage states can imply the cost of a MOR which depends on corresponding criteria that result in the sensitivity of L_r. And the confidence intervals for the damage states can be calculated in Eqn. 2.4 (Dhakal 2006) to describe the financial risk in a probabilistic format, assuming that the parameters of IM, EDP and DM all follow the lognormal distribution. Linking the damage state to financial loss based on the consequence MOR may help engineers and facility owners select a proper retrofit strategy according to the financial loss. The bound value of DS_i in rotation is obtained from moment-curvature analysis.

$$CI_{\%} = \frac{100}{1 + \left(\frac{\theta_{CI}}{\theta_c}\right)^{1.8/\beta_{com}}} \tag{2.4}$$

Table 2.1 Damage state adopted and loss ratio

	Damage state	Failure mechanism	Repair required	MOR	Outage	DM /10^{-3}rad	Loss ratio
DS1	None	Pre-yielding	None	MOR0	None		0
DS2	Minor/slight	Minor spalling	Inspect, patch	MOR1`	<3 days	0.244	10%
DS3	Moderate	Bar buckling	Repair components	MOR2	<3 weeks	4.36	30%
DS4	Major/extensive	Bar fracture	Rebuild components	MOR3	<3months	7.1	100%
DS5	Complete Collapse	Collapse	Rebuild structure	MOR4	>3 months	11.6	100%

2.5. Risk Modeling and EAL Calculating

Based on the loss ratio for the corresponding DS in table 2.1, the conditional probability of loss ratio $P[L_r|DS_i]$ can be calculated in Eqn. 2.5 and EAL can be represented by Eqn. 2.6 suggested by Dhakal (2006):

$$P[L_r \mid DS_i] = P[DS_i] \times L_r[DS_i] \tag{2.5}$$

Where $P[L_r|DS_i]$ is the conditional probability of loss ratio when arriving at DS_i; $P[DS_i]$ is the probability of being in a given DS_i; $L_r[DS_i]$ is the loss ratio for DS_i.

$$EAL = \sum_{i=1}^{n}\sum_{j=1}^{m}(f_{a_i} - f_{a_{i-1}})\left(\frac{P[L_{r,j}] + P[L_{r,j-1}]}{2}\right) \tag{2.6}$$

Where $f_{a,i}$ is the annual frequency of the i th earthquake records; $P[L_{r,j}]$ is the sum of $P[L_r|DS_j]$.

3. APPLICATION CASE

3.1. Model Details

A continuous rigid frame railway bridge designed to the draft code for seismic design of railway engineering (China Railway First Survey and Design Institute 2005) with three-span of 100m-192m-100m and 11.2m transverse width on firm soil is developed into a two dimension model shown in Fig. 2. Two main bridge columns are of 98m and 69m high respectively. The PGA of the DBE is 0.1g with the probability of 10% exceeding peak ground motion acceleration in 50 years which return period is 475 years and MCE is of 0.16g with the probability of 2% exceedance in 50 years and its return period of 2475 years (Hu, 2001). The plastic hinge zones are located under the beam-column joint with the range of 1.2m on the assumption of the flexural damage only during earthquake events. And the first modal participating mass rate is 48% to the fundamental period of 1.4s. The plastic hinge zone is modeled with nonlinear computational elements, which adopts Mander confined concrete model (Mander et al 1988) and Takeda bilinear hysteretic model (Takeda et al 1970) shown in Fig. 2, Fig. 3 and Fig. 4 (a). The bounds of DS in table 2.1 are based on moment-curvature analysis in Fig. 3 (b). The nonlinear time history analysis indicates that bridge column #2 usually yields first in plastic hinge zone.

3.2. Seismic Hazard Assessment

The study adopts 20 ground motion records used by Dhakal and Mander (2006), it is evident that the dispersion close to β=0.38 within 1.6 second shown in figure 1 (b), which fully covers the natural period of the structure. These earthquake records have magnitude between 6.5-6.9 with moderate epicenter distances of 16km-32km, recorded on firm soil (PEER Strong Motion Database). Through Eqn. 2.2, the empirical parameters are obtained, k_1=0.002 and k_2=3.5121 used to represent the hazard recurrence relationship. The unknown PGA with return period T_r, a_g^{Tr} can be expressed in DBE hazard parameters in Eqn. 2.7:

$$a_g^{T_r} = \frac{0.98}{(475 f_a)^{1/3.5121}} \tag{2.7}$$

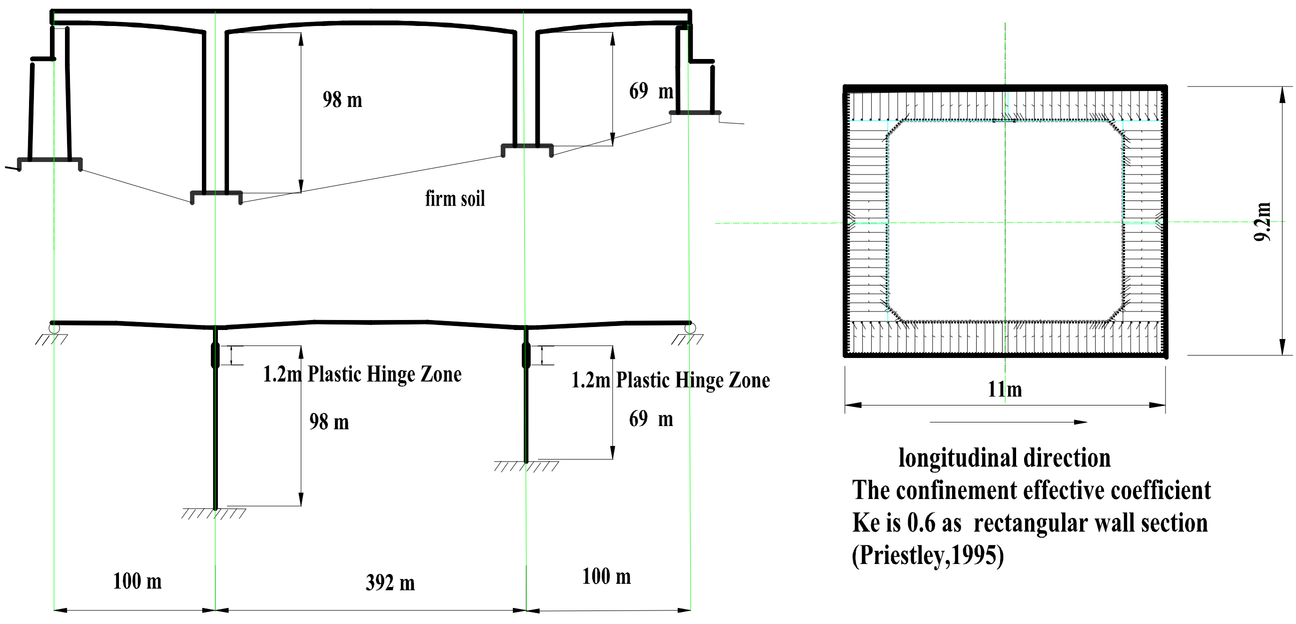


Figure 2 Bridge model and critical sections

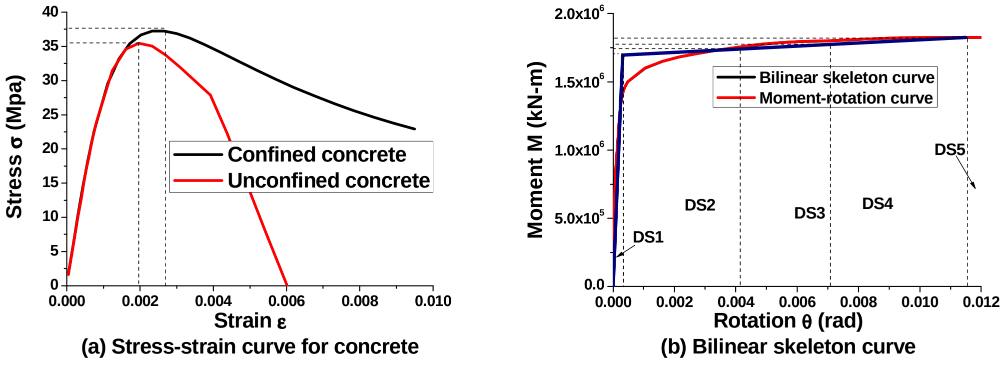
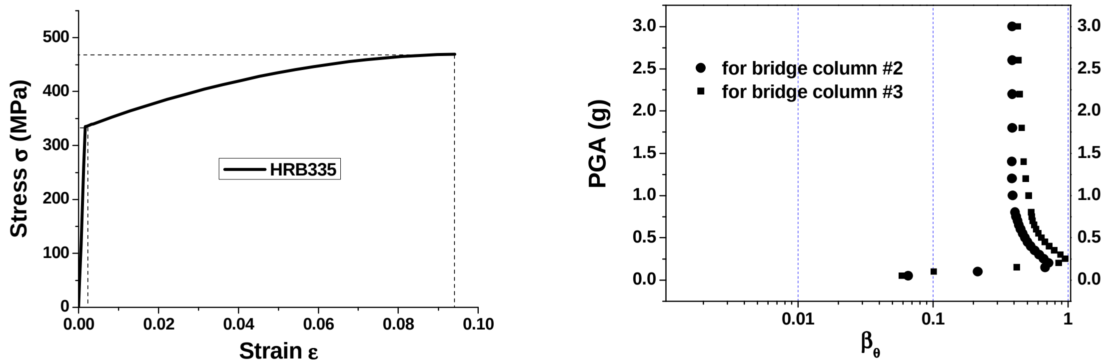


Figure 3 Concrete model and skeleton curve for Takeda hysteretic model

3.3. *Perform IDA and Model the Rotation Hazard Curve*

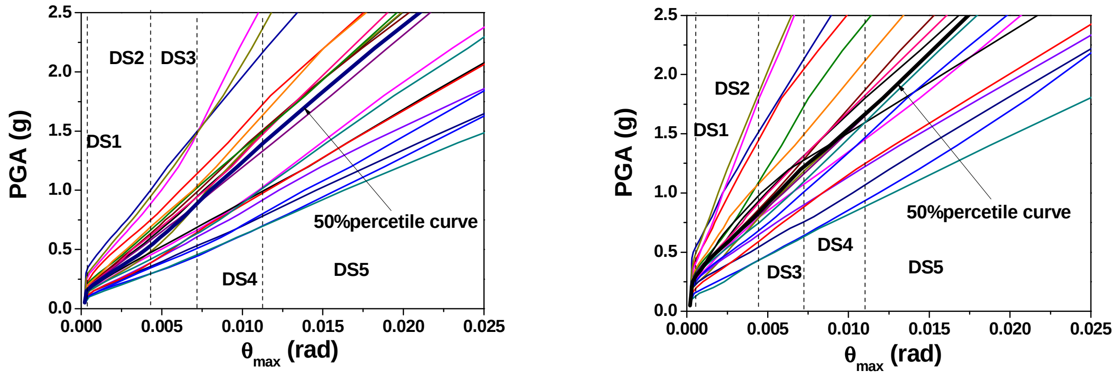
Fig. 4 (b) shows that the lognormal coefficient of variation of the PGA and the maximum rotation θ_{max} to the critical sections of the bridge columns is low based on IDA results in Fig. 5. P-value turns out far less than 0.05

for testing the hypothesis of no correlation. And the correlation factors are all more than 90%, which indicates that the correlation between PGA and maximum rotation θ_{max} is significant. So θ_{max} may have the strong correlation with annual frequency f_a, based on Eqn. 2.7, which is verified in Fig. 6 by nonlinear least square fitting and correlation analysis. And 50% percentile rotation hazard curve (f_a-θ_{max} curve) for bridge column #2 and #3 are developed shown in Fig. 5. The quantitative risk assessment can be carried out based on the nonlinear relationship between θ_{max} and f_a.



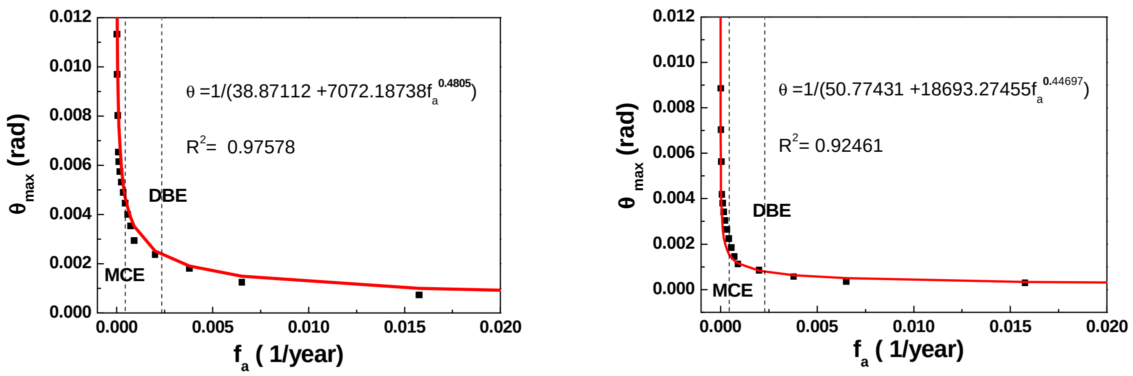
(a) Stress-strain relation of reinforcement (b) Lognormal coefficient of variation

Figure 4 Reinforcement model and the variation of θ_{max} in IDA



(a) Bridge column #2 (b) Bridge column #3

Figure 5 IDA curve in bridge columns for 20earthquakes



(a) Bridge column #2 (b) Bridge column #3

Figure 6 Fitted rotation hazard curves for 50% percentile IDA curve

3.4. Financial Risk Assessment

The confidence intervals can be calculated through nonlinear fitting relationship in Fig. 6 (a) and Eqn. 2.4 for different damage states of bridge columns and earthquake inputs with various annual frequencies, and then the total probable loss ratio $P[L_r]$ curve can be formed using results shown in Fig. 7 (a) . The financial risk assessment of the column #2 is demonstrated as an example. The conditional probability of loss ratio $P[L_r|DS_i]$ is calculated using Eqn. 2.5 and plot the financial loss curve shown in Fig. 7 (b). More details of the calculating course can be found in reference of Dhakal et al (2006). The EAL of the bridge column #2 calculated with Eqn. 2.6 and the data from table 3.1 shows that the minor and moderate earthquake events lead to more dollar loss statistically.

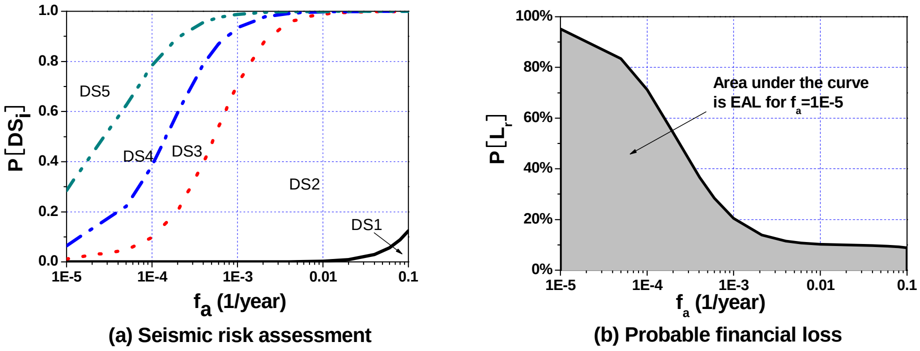


Figure 7 Seismic financial risk assessment curves for column #2

Table 3.1 Annual expected loss calculation for column #2

f_a	$P[L_r]$	Failure mechanism	MOR	Outage	ΔEAL per $1million	EAL per $1million
0.1	0.087	Pre-yielding	MOR0	None	$0	$0
0.01	0.1	Minor spalling	MOR1	<3 days	$8425.494	$8425.494
0.001	0.204	Bar buckling	MOR2	<3 weeks	$997.2752	$9422.769
0.0001	0.71	Bar fracture	MOR3	<3months	$265.9334	$9688.703
0.00001	0.953	Collapse	MOR4	>3 months	$69.00847	$9757.711

4. CONCLUSIONS AND PROSPECTS

This study demonstrates that an IDA procedure can be applied to investigate seismic financial risk exposure to seismic hazards for a three-span continuous rigid frame bridge. The financial risk of an irregular bridge is calculated in terms of loss ratio L_r, using the conventional IM PGA, EDP and DM in terms of θ_{max} with low lognormal coefficient of variation. And the significant correlation between IM and EDP ensures the model fitting in the seismic financial risk analysis. The IDA-based seismic financial risk assessment approach is feasible to consider seismic vulnerabilities of new-built or old bridges comprehensively and balance between reasonable MOR strategies and loss ratio. In addition, the result of the assessment indicates that minor and moderate earthquake may raise more seismic financial loss statistically instead of MCE. And the judgment simply from structural deformation, MOR and dollar loss is direct and rapid to make a well-informed decision , which makes full use of the engineering data and experience.

And there are four main limitations to extend the methodology of seismic financial risk assessment to the engineering practice, which required research further: 1) the financial loss data of seismic design and retrofit for bridge is rare in China. There is the necessity to establish engineering database to trace the cost and functionality to seismic retrofit for all types of bridge so that seismic vulnerability rating, seismic financial risk

management and maintenance decision can be conducted. 2) New IM parameters following lognormal distribution, such as advanced vector-valued or scalar IM, may be developed to concerning the uncertainties from the structural-site-specific characteristics so that the design, manage and maintenance against the risk can be more efficiency. 3) The nonlinear behavior of the element should be investigated to better the description of the mechanism of the whole bridge. Proper damage measure could help to lessen the subjectivity and ambiguity in determination of damage states and MOR so that the direct loss could be a relative stable amount. 4) Indirect losses should be taken in account for the bridge as a pivot in transportation network based on cost-interests rate or regional GDP level, which helps represent the functionality of a bridge more comprehensively and precisely.
.

ACKNOWLEGMENT

This work was supported primarily by National Science Foundation of China under award number 50578007. Any opinion, findings and conclusions or recommendations expressed in this material are those of the author and do not necessarily reflect those of National Science Foundation of China.

REFERENCES

Ang, AHS. and Tang, WH. (1975). Probability Concepts in Engineering Planning and Design: Volume I—Basic Principles, Wiley, New York, USA.
Brown, P.C. and Lowes, L.L. (2007). Fragility Functions for Modern Reinforced-Concrete Beam-column Joints. *Earthquake Spectra* **23:1,** 263-289.
Cornell, A. and Krawinkler, H. (2000). Progress and challenges in seismic performance assessment, *PEER Center News* **3:1,** 1-4.
Cornell, A., Jalayer, F., Hamburger, R.O. and Foutch, D.A., (2002). Probabilistic Basis for 2000 SAC Federal Emergency Management Agency Steel Moment Frame Guidelines. *ASCE Journal of Structural Engineering*, **128:4,** 526-533.
China Railway First Survey and Design Institute. (2005). the draft code of seismic design for railway engineering (in Chinese).
Dhakal, RP. and Mander, JB. (2006). Financial risk assessment methodology for natural hazards. *Bulletin of the New Zealand Society of Earthquake Engineering* **39: 2,** 91-105.
Federal Emergency Management Agency. (2000). Recommended seismic design criteria for new steel moment-frame buildings, Report No.FEMA-350, SAC Joint Venture, Washington, DC, USA.
Hu, Yu-Xian. (2001). Seismic Zonation Maps of Ground Motion Parameters (GB18306-2001), China Planning Press, Beijing, China (in Chinese).
Kennedy, R.P., Cornell, C.A., Campbell, R.D., Kalan, S. and Perla, H.F. (1980). Probabilistic Seismic Safety Study of an Existing Nuclear Power Plant, Nuclear Engineering and Design No.59, 315-338.
Mander, JB., Priestley, M.J.N. and Park, R. (1988). Observed stress-strain behavior of confined concrete. *J. Struct. Engrg* ASCE **114:8,** 1827-1849.
Takeda, T., Sozen, MA. and Nielson NN, Reinforced Concrete Response to Simulate Earthquakes, *ASCE J. Struct. Engrg. Div.*, **96:12,** 2257-2573.
Mander, J. B. and Basoz, N. (1999). Enhancement of Highway Transportation lifeline modules in HAZUS, Federal Emergency Management Agency.
Mander, J.B., Dhakal, RP., Mashko, N. and Solberg, KM. (2007). Incremental dynamic analysis applied to seismic financial risk assessment of bridges. *Engineering Structures* **29:10,** 2662–2672.
Pagni, C.A., and Lowes, L.N. (2006). Fragility functions for older reinforced concrete beam-column joints. *Earthquake Spectra* **22,** 214-238.
PEER Strong Motion Database, http://peer.berkeley.edu/smcat/
Priestley, M.J.N., Seible, F. and Calvi, G.M. (1995). Seismic Design and Retrofit of Bridges, John Wiley & Sons, New York, USA.
Vamvatsikos, D. and Cornell, CA. (2002). Incremental dynamic analysis. *Earthquake Engineering and structural Dynamics* **31,** 491-514.