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Practical Approach to Fragility Analysis of Bridges

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Abstract: Damages during past earthquakes reveal seismic vulnerability of bridge structures and the necessity of probabilistic approach toward seismic performance evaluation of bridges and its interpretation in terms of decision variables such as repair cost, downtime and life loss. This Procedure involves hazard analysis, structural analysis, damage analysis and loss analysis. The purpose of present study is reviewing different methods developed to derive fragility curves for damage analysis of bridges and demonstrating a simple procedure for fragility analysis using Microsoft Office Excel worksheet to reach probability of occurring predefined level of damage due to different levels of seismic demand parameters. The input of this procedure is the intensity of ground motion and the output is an appropriate estimate of the expected damage. Different observed damages of the bridges are discussed and compared the practical definition of damage states. Different methods of fragility analyses are discussed and a practical step by step example is illustrated.

Keywords: Bridge vulnerability, fragility curves, seismic assessment

INTRODUCTION

Due to the uncertainty and randomness inherited in the ground motions, a probabilistic approach is needed for seismic design and assessment of structures. Among different probabilistic methods seismic fragility analysis is a powerful means for quantifying the likelihood of structural damage as a function of ground motion level. Both ground motion level and structural damage can be defined numerically or qualitatively. Fragility function is the relationship between the severity of ground motion and the probability of reaching or exceeding predefined levels of damage. Fragility analysis output can be expressed through damage probability matrix and fragility curves. The damage probability matrix gives the probability of different damage states at a specific level of ground motion, while each fragility curve gives the probability of a specific damage state at different levels of ground motion.

Fragility of the structure is classified according to analytical or empirical methods used. Analytical fragility curves are derived from numerical simulations of the system under artificial or historical earthquake records. Empirical fragility curves are generated from experimental results or damage data after earthquakes. Experts' judgment is another way for empirical fragility analysis.

One challenge in analytical methods is generating artificial ground motions consistent with the site properties. Lack of sufficient data is the main deficiency in empirical methods. Resulted fragility curves must become integrated with seismic hazard and cost data to provide proper estimates of seismic risk.

There have been recent studies on seismic fragility analysis of bridges. Nielson and DesRoches (2007) considered multiple vulnerable components in steel and concrete girder bridges and Choe *et al*. (2008, 2009) applied nonlinear static analysis to consider possible capacity reduction and fragility increase in a typical single-bent bridge in California with RC columns in marine splash zones. Ghosh and Padgett (2010) studied the effect of aging in time dependent fragility curves. Therefore, seismic fragility analysis of important structures and specifically bridges is going to be more widespread among engineers and a simple way to generate fragility curves is needed.

In present study, a practical method to generate seismic fragility curve of a bridge structure is discussed through an example. The introduced procedure simplifies the complicated steps of generating fragility curves to make them applicable for engineers who intend to design seismically resistant bridges or evaluate seismic behavior of these structures.

MATERIALS AND METHODS

Seismically inducing bridge damages:

Observed seismically induced damages of bridges: Past observations demonstrate that bridges are seismically vulnerable. Deficiency of previous design codes beside

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Table 1: Qualitative description of bridge damages

Damage state	Slight/minor damage	Moderate damage	Extensive damage	Complete damage
Description	Minor cracking and spalling to the abutment, cracks in shear keys at abutments, minor spalling and cracks at hinges, minor spalling at the column (damage requires no more than cosmetic repair) minor cracking to the deck	Any column experiencing moderate cracking and spalling (column structurally still sound), any connection having cracked shear keys or bent bolts, or moderate settlement of the approach	Any column degrading without collapse (column structurally unsafe), any connection losing some structurally unsafe),any connection losing some bearing support, or major settlement of the approach	Any column collapsing and connection losing all bearing support, which may lead to imminent deck collapse

Table 2: Capacity-demand ratio associated to damage states

Damage state	No/minor damage	Repairable damage	Significant damage
Description (Hwang *et al*., 2000)	Although minor inelastic response may occur, post earthquake damage is limited to narrow cracking in concrete. Permanent deformations are not apparent.	Inelastic response may occur, resulting in concrete cracking, reinforcement yield and minor spalling of cover concrete. Extent of damage should be sufficiently limited so that the structure can be restored essentially to its pre-earthquake condition without replacement of reinforcement or replacement of structural members. Repair should not require closure. Permanent offsets should be avoided.	Although there is minimum risk of collapse, permanent offsets may occur and damage consisting of cracking, reinforcement yielding and major spalling of concrete may require closure to repair. Partial or complete replacement may be required in some cases.
C/D ratios	0.5<C/D	0.33<C/D<0.5	C/D<0.33

Table 3: Descriptions of damage observed in the experiment

Damage state	None	Minor damage	Moderate damage	Major damage
Observed damage	no damage in columns	extent of spalling of concrete cover ≤80 mm	extent of spalling of concrete cover >80 mm	buckling of reinforcing bar (s); formation of plastic hinge (s)

inappropriate philosophy of design caused different damages to these structures, which is studied comprehensively. Before 1970, application of elastic assumption in design (which is still in use in some counties), has caused three main consequences (Priestley *et al*., 1996):

- Seismic deflection was extremely underestimated due to using gross section in place of cracked section.
- This philosophy considers seismic force elastic and therefore artificially lower than actually induced forces. So, the ratio of gravity force to seismic force adopted for design was incorrect which led to moment patterns under combination of gravity load and seismic load which were underestimated and in wrong shape with unrealistic location of plastic hinges.
- Inelastic behavior and its dependence to ductility is another important parameter in seismic performance of the bridge which has the prominent role in energy dissipation, but is also neglected in elastic assumption.

In most fallowing observed damages, these deficiencies are obvious:

- Failures according to seismic displacement:
  - Span failure due to unseating at movement joints (Like Span collapse 1971, San Fernando earthquake and unseating of simply supported link span of Nishinomya-ko bridge 1995, Kobe earthquake)
  - Amplification of displacement due to soil effects
  - Abutment slumping (Like abutment slumping and rotation failure of Rio Banano bridge, 1990, Costa Rica earthquake)
- Column failures:
  - Flexural strength and ductility failures (Inadequate flexural strength and undependable column flexural strength and premature termination of reinforcement like confinement failure, 1971, San Fernando Earthquake)
  - Column shear failures
- Joint failures
- Footing failures

Proposed damage states: According to observed damages, different qualitative and quantitative descriptions were proposed. Some of them are discussed in this section. Hazus (1997) description is represented in Table 1.

Hwang *et al*. (2000) related qualitative descriptions to capacity-demand ratio as shown in Table 2.

The quantitative description observed in experiment is illustrated in Table 3 (Johnson *et al*., 2006).

Table 4: Repair action and outage of damage states

Damage states	None	Slight/minor damage	Moderate damage	Major/extensive damage	Complete damage /collapse
Failure mechanism	pre-yielding	minor spalling	bar buckling	bar fracture	collapse
Repair action	none	inspect, patch	repair components	rebuild components	rebuild structure
Outage	none	<3 days	<3 weeks	<3 months	>3 months

Mander *et al.* (2007) related hazus (1999) described damage states to repair action and outage to gain loss assessment, which is shown in Table 4.

Fragility function: Fragility function is the probability that a component, element or system will face a predefined damage level, given a predefined hazard level. Fragility functions can be expressed in the form of lognormal cumulative distribution functions, with a median of α and logarithmic standard deviation, β. The mathematical format of fragility function is as Eq. (1):

$$F_i(D) = \Phi\left(\frac{1n\left(\frac{D}{\alpha_i}\right)}{\beta_i}\right) \quad (1)$$

In which *Fi* (*D*) is the conditional probability that the structural element reaches damage state "i"; Φ is the standard normal cumulative distribution function, α_i represents the median value of the probability distribution and βi stands for the logarithmic standard deviation. Therefore, the probability that an element has damage state i and it does not reach next damage states can be expressed as Eq. (2):

$$P[i|D] = Fi(D) - Fi + 1(D) \quad (2)$$

Lognormal distribution is appropriate due to the fact that it fits failure data of structural and nonstructural component, it has been used in hazard analysis for years and it has zero probability density for below zero variables and therefore, it is a suitable function to match observation with mathematics.

The logarithmic standard deviation, β, expresses uncertainty in the actual value of demand parameter, associated to the damage state, which is a result of diversity in the quality of construction, installation and load pattern experienced before failure.

Classification of fragility data: Correlating damage data with the severity of the corresponding ground motions is the core part of Fragility analysis. Data derived from damages observed in past earthquakes, experimental data resulted from test results, data gained by analytical modeling and data based on experts' opinion.

Past earthquake damage data: Damage data can be derived from past earthquakes. To gain this purpose desired earthquake intensity should be determined. Lack of accurate data can be a deficiency of this method preventing comprehensive fragility analysis based on this approach.

Developing fragility curves based on experimental data: Different tests should be performed to generate necessary damage for fragility analysis. This approach is advantageous in comparison with the first approach as controlling over the range of ground motions and accurate measuring of structural damage is possible. But it is a more expensive and time consuming way due to scarcity of test equipments such as test tables.

Data derived by analytical modeling: In this approach a model of the structure is generated and numerical analysis is performed to gain the required damage. Structural analysis can be performed under different levels of ground motion intensities. Seismic excitation can be generated by scaling past earthquake acceleration time history and generating artificial seismic excitation based on stochastic models. Due to its advantages, this approach is more practical in fragility analysis.

Data based on experts' judgment: In this approach, experts are asked to give their estimate of the damage level due to each level of ground motion intensity. This is the least expensive approach. But it is not as reliable of previous approaches as it is subject to personal opinion.

Characterization of ground motion intensity: Ground motion intensity should be quantified to generate fragility function of the structure. It is essential for fragility analysis to express the ground motion intensity in a quantitative way. One or more ground motion parameters are considered for fragility information to be generated. Appropriate ground motion should well represent the characteristics such as frequency content and duration of strong motion. Three possible choices are: peak values, spectral values and energy parameters.

Peak values of the Ground motion, Displacement (PGD), Velocity (PGV) and Acceleration (PGA), can be used as ground motion representative. But they only represent amplitude and does not show the frequency content of ground motions, themselves.

Spectral values represent peak values of response of the structural system when subjected to the ground

motion. They are maximum acceleration, maximum relative velocity, or maximum relative displacement of a linear elastic Single-degree-of Freedom (SOOF) system subjected to the considered ground motion.

Energy parameters represent ground motions due to energy dissipated from the motion or energy observed by a standard system subjected to the motion.

How to generate fragility curves: Different methods have been developed to generate fragility curves. Access to damage data, structural system and ground motion model are criteria which dictate use of each of these methods. Some of them are discussed in the following sections.

Monte carlo method: The simplest method of generating fragility curves is Mont Carlo method. However it needs more computations than other methods. This method can be used with different ground motion models without limitation considering uncertainty and randomness of ground motion and structural characteristics. To gain the fragility by using Monte Carlo simulation, for each level of ground motion level, n sample time histories should be generated. The bridge should be analyzed under each ground motion and the number of ground motions which causes each damage state should be recorded. Each fragility level is calculated by division of number of observed damage by total number of sample ground motions with that intensity level, n. This procedure should be repeated for different levels of ground motion and different damage states to finally gain the total fragility.

Hirata method: Seismic capacity and seismic response demand can be assumed random variables with lognormal distribution. The median and standard deviation of the seismic capacity is gained from experience or observed events. For the seismic response demand at a defined seismic level, these parameters are derived from the structural analysis under different seismic excitations with same intensity level.

Assuming R and S (e) have log-normal distribution, the probability of failure is calculated through Eq. (3):

$$P_f = (e) = \Phi\left(\frac{\ln S_m(e) - \ln R_m}{\sqrt{\beta_r^2 + \beta_s^2}}\right) \quad (3)$$

In which R_m and S_m (e) are medians of R and S (e), respectively and β_r and β_s are their log-normal standard deviations. Φ is the cumulative function of standard normal distribution.

Methods based on random vibration theory: For linear structures, random vibration theory is another mean to gain fragility curves. In this method ground motion is modeled randomly and applied to the linear analytical model of the structure. F_{ds} (i), at intensity i for damage state ds is calculated as Eq. (4):

$$\begin{aligned} F_{ds}(i) &= P\,[max\{(i, t) > R_{ds}] \\ &= P\,[\textit{at least one crossing of} \\ &\quad R_{ds}\ \textit{during motion duration}, T\,] \\ &= 1 - \exp(-\upsilon(i, R_{ds}).\,T) \end{aligned} \quad (4)$$

In which R_{ds} is the response of the system according to damage state ds, T is ground motion duration and υ (i, R_{ds}) is the mean up-crossing rate of the response at ground motion intensity i.e., Hwang *et al*. (1997) has used this method to generate fragility of a plane reinforced concrete frame.

Illustrative example:

Seismic response of the structure: Having the response matrix of a sample bridge, this section purposes a simple procedure to derive fragility curves using Microsoft Office Excel worksheet. According to qualitative definition of damage states in Table 1, different parameters can be used as seismic performance indicator of a bridge structure such as column moment-curvature ductility factor of its columns with limit states of $1<\mu<2$ for slight damage, $2<\mu<4$ for moderate damage, $4<\mu<7$ for extensive damage and $\mu>7$ for complete damage. Eight time history records of past events obtained from PEER strong motion database are scaled from PGA = 0.20 to 10.50 m/sec^2 and incremental dynamic analysis (Vamvatsikos and Cornell, 2002) is performed to numerical model of the bridge structure to estimate seismic response values (herein, column-curvature ductility factor).

Table 5 demonstrates the response matrix of an example concrete two span bridge. Collecting all the column curvature ductility factor of the all selected events produces a 15× 8 response matrix.

Simplified method for fragility curve derivation: For each line of the matrix (i.e., each PGA level), the curvature ductility can be assumed to follow a lognormal distribution with Probability Density Function (PDF) as Eq. (5):

$$F_x(X) = \frac{1}{\sqrt{2\pi}\zeta x}\exp\left[-\frac{1}{2}\left(\frac{\ln x - \lambda}{\zeta}\right)^2\right] 0 \le X < \infty \quad (5)$$

Table 5: Example response matrix resulted from Incremental Dynamic Analysis (IDA)

	Column curvature ductility							
PGA (m/sec2)	Event # 1	Event # 2	Event # 3	Event # 4	Event # 5	Event # 6	Event # 7	Event # 8
0.20	0.028	0.031	0.023	0.026	0.031	0.031	0.041	0.026
0.35	0.048	0.053	0.039	0.046	0.053	0.054	0.070	0.046
0.50	0.068	0.075	0.056	0.066	0.075	0.076	0.094	0.066
1.00	0.126	0.155	0.107	0.124	0.137	0.142	0.267	0.137
1.50	0.197	0.268	0.150	0.266	0.188	0.199	0.652	0.231
2.00	0.383	0.441	0.237	0.517	0.228	0.313	1.465	0.586
2.50	0.565	0.713	0.345	0.929	0.310	0.521	2.390	1.042
3.00	0.828	1.041	0.472	1.529	0.477	0.757	3.505	1.534
3.50	1.144	1.401	0.705	2.064	0.652	1.189	4.487	2.092
4.00	1.552	1.687	0.952	2.656	0.772	1.485	5.319	2.692
5.00	2.059	2.299	1.659	3.401	1.336	2.529	6.697	3.116
6.00	2.533	2.748	2.320	4.021	2.028	3.616	7.866	3.404
7.50	3.407	3.412	3.213	5.415	3.016	5.3591	0.444	4.766
9.00	4.070	3.543	4.019	6.573	4.020	6.1181	1.681	5.980
10.50	4.709	3.981	4.765	7.601	4.514	6.675	13.153	7.147

Table 6: Steps of calculation of λ

PGA (m/sec2)	Average (R[i,:]) μ	Stdev (R[i.:]) σ	σ/μ δ	Sqrt (ln[1+δ^2]) ζ	lnμ-1/2ζ^2 λ
0.20	0.03	0.01	0.18	0.18	-3.54
0.35	0.05	0.01	0.18	0.18	-2.99
0.50	0.07	0.01	0.16	0.16	-2.65
1.00	0.15	0.05	0.33	0.32	-1.95
1.50	0.27	0.16	0.59	0.55	-1.46
2.00	0.52	0.40	0.77	0.68	-0.89
2.50	0.85	0.67	0.79	0.70	-0.40
3.00	1.27	0.99	0.78	0.69	0.00
3.50	1.72	1.24	0.72	0.65	0.33
4.00	2.14	1.46	0.68	0.62	0.57
5.00	2.89	1.69	0.58	0.54	0.91
6.00	3.57	1.87	0.52	0.49	1.15
7.50	4.88	2.45	0.50	0.47	1.47
9.00	5.75	2.67	0.46	0.44	1.65
10.50	6.57	2.99	0.45	0.43	1.79

Table 7: Calculation of probabilities

Bounds of damage states in term of column curvature ductility			
I	II	III	IV
1	2	4	7
1-Normsdist	1-Normsdist	1-Normsdist	1-Normsdist
1-Normsdist ([(ln1-λ)/ζ]	1-Normsdist ([(ln2-λ)/ζ]	1-Normsdist ([(ln4-λ)/ζ]	1-Normsdist ([(ln7-λ)/ζ]
Prb-I	Prb-II	Prb-III	Pb-IV
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.10	0.01	0.00	0.00
0.28	0.06	0.01	0.00
0.50	0.16	0.02	0.00
0.69	0.29	0.05	0.01
0.82	0.42	0.09	0.01
0.95	0.66	0.19	0.03
0.99	0.82	0.32	0.05
1.00	0.95	0.57	0.16
1.00	0.99	0.73	0.25
1.00	0.99	0.82	0.36

in which, λ and ζ are the parameters of the lognormal distribution, which can be calculated by using mean (μ) and the standard deviation (σ) of the derived response quantities, as shown in Eq. (6) and (7):

$$\zeta^2 = \ln\left[1+\left(\frac{\sigma}{\mu}\right)^2\right] \tag{6}$$

$$\lambda = \ln\mu - \frac{1}{2}\zeta^2 \tag{7}$$

Required calculations is illustrated, step by step, through Microsoft office excel functions in Table 6:

And the probability that the damage of the structural element exceeds damage state Si at a specific PGA level, derived from Eq. (8):

$$P[S > s|PGA] = 1 - \Phi\left[\frac{\ln(x_i) - \lambda}{\zeta}\right] \tag{8}$$

In which Φ (.) is the standard normal cumulative distribution function, Xi is the upper bound for Si (i = I, II, III, IV) and λ and ζ areas defined above and are dependent on the PGA level. The resulted probability is shown in the Table 7:

RESULTS AND DISCUSSION

In previous section, probabilities of occurring different damage levels are calculated. Results shown in Table 7 are used to generate fragility curve of the bridge which is illustrated in Fig. 1.

According to growing use of fragility curves in bridge engineering, a practical way of generating them is proposed. As fragility curve is a visual tool, it is easy to interpret it to evaluate seismic vulnerability of bridges and as an input for loss assessment of them to decide about the necessity of rehabilitation.

Recent studies of vulnerability assessment of bridge structures need numerous fragility analysis, in terms of

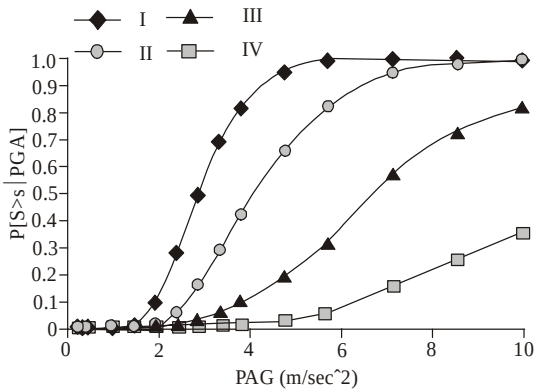

Fig. 1: Generated fragility curves

fragility analysis of bridge components, fragility analysis considering aging during service time and fragility analysis after different retrofit methods, therefore a simplified method is presented in this study to make the process less time consuming. This study intends to make theoretical approach of developing fragility curves more practical in design and evaluation of the bridge structures according to performance-based earthquake engineering philosophy.

A simplified method for generating fragility curves of bridge structures is presented. Having a response matrix consisting of levels of ground motion intensity versus seismic demand parameter resulted from incremental dynamic analysis and defining bounds of damage states, fragility curve of a bridge, using simplified method, is generated as shown in Fig. 1. Derived fragility curve gives the probability of exceeding each damage state, while the structure is subjected to different levels of ground motion intensity. The necessary steps are described in detail, as a guide for future application of this procedure.

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