SEISMIC FRAGILITY ANALYSES OF A CURVED BRIDGE SYSTEM CONSIDERING AN OPTIMAL INTENSITY MEASURE

University of Tsukuba, Student Member, ○Muhammad Rashid
University of Tsukuba, Regular Member, Mayuko Nishio

1. INTRODUCTION

Seismic hazards pose a significant threat to the infrastructure, particularly to bridges, which are critical components of transportation networks. The damage caused by earthquakes to bridges can lead to severe economic and social consequences. Therefore, it is of paramount importance to assess the seismic vulnerability of bridge structures and develop strategies to mitigate their potential damage. Seismic fragility analysis is a widely accepted methodology for evaluating the probability of a structure's failure during seismic events. In this context, an optimal intensity measure (IM), is a crucial parameter that can significantly affect the results of seismic fragility analysis.

This study aims at presenting the procedure for fragility estimation using an optimal intensity measure. The optimal IM was chosen based on efficiency, practicality, proficiency, and hazard computability. For illustration purposes, an existing testbed structure is considered and subjected to a series of time history loading. The bridge structure is geometrically curved in shape and is located in Japan, which is a high seismic hazard zone. The fragility curves are developed using the probabilistic seismic demand models (PSDM) and the associated capacity limit states. The findings of this study can help understand to prioritize the retrofitting strategies to mitigate the potential damage caused by earthquakes.

2. NUMERICAL MODELING OF THE STUDY BRIDGE SYSTEM

The target bridge is a composite type of system, consisting of steel and reinforced concrete (RC) sections. The geometric complexities are due to the presence of the on-off ramps and horizontal curvature in the mainline. The bridge is supported by five steel and one RC pier and abutments at the end. The analytical modeling was carried out in a finite element environment as shown in Fig. 1, where the bridge superstructure, the bent-top beams, and the abutments were modeled as a linear beam element. For the non-linear response of the piers, the piers are modeled as fiber elements. Using the design standards, the damping coefficient of 3% and 5% are used for the steel and concrete elements, respectively. The multiple shear springs (MSS) bi-linear elements are used to simulate the mechanical characteristics of the lead rubber bearings (LRB). To account for the soil-structure interaction, equivalent linear translational and rotational springs were defined to model the piers and abutment foundations.

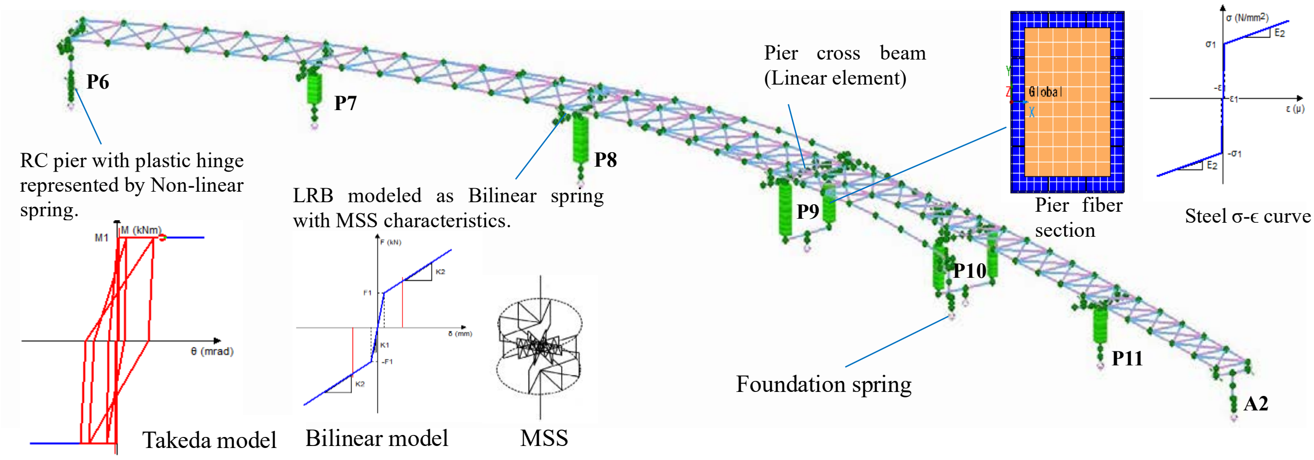

Fig. 1 Analytical model of the target bridge structure

3. SELECTION OF AN OPTIMAL INTENSITY MEASURE

Seismic IMs are of great significance in deriving the PSDM. The PSDMs are developed using the power law in Eq. (1) through regression analysis of structural non-linear time history analysis results.

$$EDP = a(IM)^b \tag{1}$$

Where EDP is the component's response, IM is the intensity measure, and a, and b are the regression coefficients. The parameters of the PSDM are then used to select the optimal IM.

An optimal IM helps to mirror the degree of uncertainty in the demand model and hence a reliable fragility estimation. In the past, for different structural types, many IMs were adopted for seismic demand estimation. Peak ground acceleration (PGA), peak ground velocity (PGV), and first mode spectral acceleration (S_{aT1}) were some of the most commonly used IMs (Mackie & Stojadinović, 2001). To identify an optimal IM for the study bridge, a total of six IMs are investigated as shown in Table 1. An optimal IM is advantageous in terms of efficiency, proficiency, practicality, and hazard computability, which are characterized by a smaller value of dispersion (β_{IM}), modified dispersion (ξ), larger value of the slope term (b)

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Contact address: Deptt of Engg Mechanics and Energy, University of Tsukuba, 1-1-1 Tennodai, Tsukuba 305-8573, Japan

in the PSDM and less effort in conducting the probabilistic seismic hazard analysis, respectively (Padgett & DesRoches, 2008). In this study, the optimal IM is derived based on the aforementioned properties for the chosen IMs from the components PSDMs. For each component, the bold values in the given table represent the most suitable parameter. Spectral acceleration at 1.0 sec (Sa_{10}) has the best metric of all the measures due to a stronger correlation with the structural response and thus reduces the uncertainty in the demand models. By contrast, the displacement-related IM exhibits the poorer metric, which means that the ground displacement cannot predict well the demand for the bridge system.

Table 1. Selection of an optimal *IM* based on the components' PSDM parameters

Component	**Bearing**				**Concrete pier**				**Steel pier**			
PSDM parameters	b	β_{IM}	$\lvert\xi\rvert$	R^2	b	β_{IM}	$\lvert\xi\rvert$	R^2	b	β_{IM}	$\lvert\xi\rvert$	R^2
Peak acceleration (*PGA*)	1.37	0.92	0.67	0.41	0.29	0.67	2.3	0.45	-0.62	0.56	0.9	0.4
Peak velocity (*PGV*)	2.16	0.6	0.28	0.75	0.87	0.45	0.52	0.76	-0.15	0.38	2.57	0.72
Peak displacement (*PGD*)	0.49	0.84	1.71	0.5	-0.42	0.65	1.55	0.49	-1.15	0.51	**0.45**	0.5
Arias intensity (I_a)	0.4	0.88	2.21	0.46	-0.48	0.65	1.35	0.49	-1.2	0.54	**0.45**	0.45
Spectral acceleration at 0.2 sec (Sa_{02})	0.66	0.96	1.46	0.35	-0.27	0.71	2.66	0.39	-1.05	0.59	0.56	0.33
Spectral acceleration at 1.0 sec (Sa_{10})	**2.35**	**0.44**	**0.19**	**0.87**	**0.9**	**0.44**	**0.49**	**0.77**	**0.3**	**0.24**	0.8	**0.89**

4. FRAGILITY CURVES FOR THE BRIDGE SYSTEM

The seismic vulnerability of the bridge structure is described in terms of fragility functions for different limit states. The components PSDM are developed using the peak response of the components obtained from the non-linear time history analysis and the associated engineering demand parameter (EDP) value. Ductility demand, shear strain, and distortion strain are considered the EDPs for fragility development for the RC pier, bearings, and steel pier respectively. Using the power law in Eq. (1), the mean and standard deviation for each limit state is calculated through regression analysis. Finally, the fragility curves are derived based on Eq. (2).

$$P[LS \mid IM = x] = \Phi\left[\ln\left(x/\lambda_c\right) \Big/ \sqrt{\beta_d^{\,2} + \beta_c^{\,2}}\right] \tag{2}$$

where Φ [*] represents the standard normal cumulative distribution function, λ_c is the median value of the intensity measure *IM* and the term in the denominator represents the cumulative dispersion for a particular damage state. In addition to the fragility curves for each of the bridge components, the bridge system fragility curves are also plotted in Fig. 2. The bearings are more vulnerable in terms of reaching the limit state. Whereas the steel piers are hardly reaching the limit states at lower demand values. It can be observed that the system fragility is dominated by the LRB response in the slight and moderate damage states, while the piers are contributing to the higher damage states.

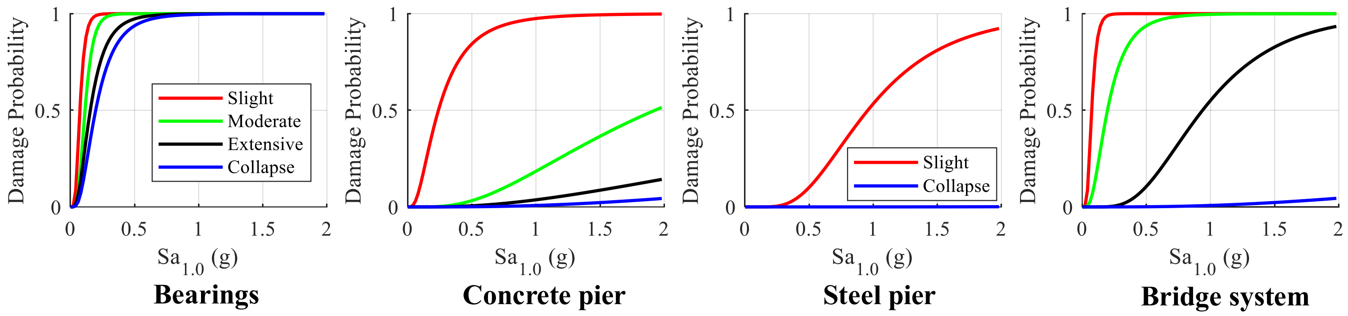

Fig. 2 Fragility curves for bridge components and system

5. CONCLUSIONS

The fragility functions for an existing bridge structure were developed using the optimal IM, based on the lower dispersion and high practicality values from the components PSDM. The fragility curves are developed using Sa_{10} as an optimal IM. Bearings were found the most fragile among the components, which eventually affect the system fragility in the lower damage states. However, at higher damage states, the piers are dominating the system's fragility. These findings may help in accurate vulnerability estimation for the decision-making and structural retrofit prioritization in extreme seismic events.

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