교량의 해석적 손상도 곡선

Analytical Fragility Curves for Bridge

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Abstract

This paper presents a generation of analytical fragility curves for bridge. The analytical fragility curves are constructed on the basis of nonlinear dynamic analysis. Two-parameter lognormal distribution functions are used to represent the fragility curves with the parameters estimated by the maximum likelihood method.

To demonstrate the development of analytical fragility curves, two of representative bridges with a precast prestressed continuous deck in the Memphis, Tennessee area are used.

Key words : Fragility curves, maximum likelihood method, lognormal distribution.

1. Introduction

In performing a seismic risk analysis of a structural system, it is imperative to identify seismic vulnerability of component structures associated with various states of damage. The development of vulnerability information in the form of fragility curves is a widely practiced approach when the information is to be developed accounting for a multitude of uncertain sources involved, for example, in estimation of seismic hazard, structural characteristics, soil-structure interaction, and site conditions. In principle, the development of fragility curves will require synergistic use of the following methods: (1) professional judgement, (2) quasi-static and design code consistent analysis, (3) utilization of damage data associated with past earthquakes, and (4) numerical simulation of seismic response of structures based on dynamic analysis.

This paper concentrates on the development of analytical fragility curves for bridges as described in (4) above, by numerically simulating seismic response

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with the aid of structural dynamic analysis. At the same time, this paper introduces the development procedure of fragility curves under the assumption that they can be represented by two-parameter lognormal distribution functions. Analytical fragility curves are developed for typical bridges in the Memphis, Tennessee area on the basis of a nonlinear dynamic analysis.

Two-parameter lognormal distribution functions were traditionally used for fragility curve construction. This was motivated by its mathematical convenience in relating the actual structural strength capacity with the design strength primarily through a seismic factor of safety which can be factored into a number of multiplicative safety factors, each associated with a specific source of randomness and/or uncertainty. When the lognormal assumption is made for each of these factors, the overall seismic safety factor also distributes lognormally due to the multiplicative reproducibility of the lognormal variables. This indeed was the underpinning and practical assumption that was made in the development of probabilistic risk assessment methodology for nuclear power plants in the 1970s and in the early 1980s (NRC, 1983).

2. Analytical Fragility Curves

It is assumed that the fragility curves can be expressed in the form of two-parameter lognormal distribution functions, and the estimation of the two parameters (median and log-standard deviation) is performed with the aid of the maximum likelihood method. For this purpose, the PGA (Peak Ground Acceleration) is used to represent the intensity of the seismic ground motion, although the use of intensity measures other than PGA such as PGV (Peak Ground Velocity), SA (Spectral Acceleration), SI (Spectral Intensity) and MMI (Modified Mercalli Intensity) are possible.

The likelihood function for the present purpose is expressed as

$$L=\prod_{i=0}^{N}[F(a_i)]^{x_i}[1-F(a_i)]^{1-x_i} \tag{1}$$

where $F(.)$ represents the fragility curve for a specific state of damage, a_i is the PGA value to which bridge i is subjected, $x_i=1$ of 0 depending on whether or not the bridge sustains the state of damage under PGA$=a_i$, and N is the total number of bridges inspected after the earthquake. Under the current lognormal assumption, $F(a)$ takes the following analytical from

$$F(a)=\Phi\left[\frac{\ln\left(\frac{a}{c}\right)}{\zeta}\right] \tag{2}$$

in which "a" represents PGA and $\Phi[.]$ is the standardized normal distribution function.

The two parameters c and ζ in Eq. 2 are computed as c_0 and ζ_0 satisfying the following equations to maximize $\ln L$ and hence L;

$$\frac{d\ln L}{dc} = \frac{d\ln L}{d\zeta} = 0 \qquad (3)$$

This computation is performed by implementing a straightforward optimization algorithm.

3. Numerical Application

3.1 Bridge Model

To demonstrate the development of analytical fragility curves, two of representative bridges with a precast prestressed continuous deck in the Memphis, Tennessee area studied by Jernigan and Hwang (1997) are used. The plan, elevation and column cross-section of Bridge 1 are depicted in Fig. 1. Geometry and configuration of Bridge 2 is similar to Bridge 1. Bridge 2 also has a precast prestressed continuous deck. However, the deck is supported by 2 abutments and 4 bents with 5 spans equal to 10.7 m (35′), 16.8 m (55′), 16.8 m (55′), 16.8 m (55′) and 10.7 m (35′). Each bent has 3 columns of 5.8 m (19′) high with the same cross-sectional and reinforcing characteristics as those of Bridge 1. Following Jernigan and Hwang (1997), the strength f_c of 20.7 MPa (3000 psi) concrete used for the bridge is assumed to be best described by a normal distribution with a mean strength of 31.0 MPa (4500 psi) and a standard deviation of 6.2 MPa (900 psi), whereas the yield strength f_y of grade 40 reinforcing bars used in design is described by a lognormal distribution having a mean strength of 336.2 MPa (48.8 ksi) with a standard deviation of 36.0 MPa (5.22 ksi). Then, a sample of ten nominally identical but statistically different bridges are created for each of Bridge 1 and 2 by simulating ten realizations of f_c and f_y according to respective probability distribution functions assumed. Other parameters that could contribute to variability of structural response were not considered in the present analysis under the assumption their contributions can be disregarded

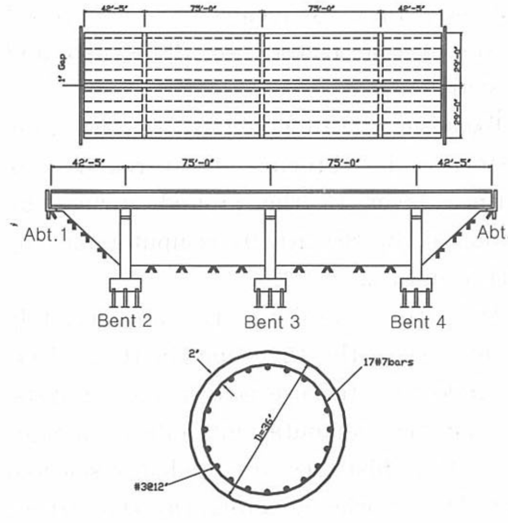

Fig. 1 A representative Memphis Bridge (Bridge 1)

3.2 Seismic Ground Motion

For the seismic ground motion, the time histories generated by Howard et. al (1996) at the Center for Earthquake Research and Information, the University of Memphis are used. These time histories are generated by making use of the Fourier acceleration amplitude on the base rock derived under the assumption of a far-field point source by Boore (1983). In fact, the study area is located 40 km to 100 km from Marked Tree, Arkansas, the epicenter of the 1846 earthquake which magnitude is 6.5.

Upon using seismologically consistent values for the parameters in the Boore and other related models and converting the Fourier acceleration amplitude to the power spectrum, corresponding time histories are generated in terms of sample function of a normal (Gaussian) stationary process on the base rock by means of the spectral representation method by Shinozuka and Deodatis (1991).

The seismic wave represented by these time histories is propagated through the surface layer to the ground surface by means of the SHAKE 91 computer code by Idriss and Sun (1992).

And the results are appropriately modulated in the time domain(Howard et. al, 1996), for the use of response analysis. To minimize computational effort, samples of 10 time histories are randomly selected from 50 histories generated by Howard et. al (1996) for each of the following eight combinations of M (magnitude) and R (epicentral distance); M=6.5 with R=80 km and 100 km, M=7.0 with R=60 km and 80 km, M=7.5 with R=40 km and 60 km, and M=8.0 with R=40 km and 60 km.

Typical ground motion time histories for two extreme combinations M=8.0 with R=40 km and M=6.5 with R=100 km are shown in Fig. 2. The spectral accelerations for the above input ground accelerations are shown in Fig. 3 to provide an insight

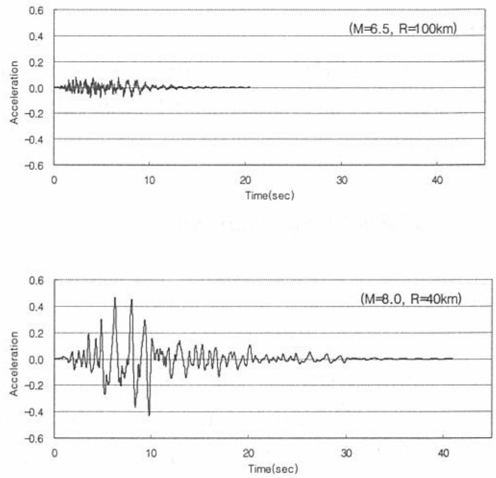

Fig. 2 Typical Input Ground Acceleration

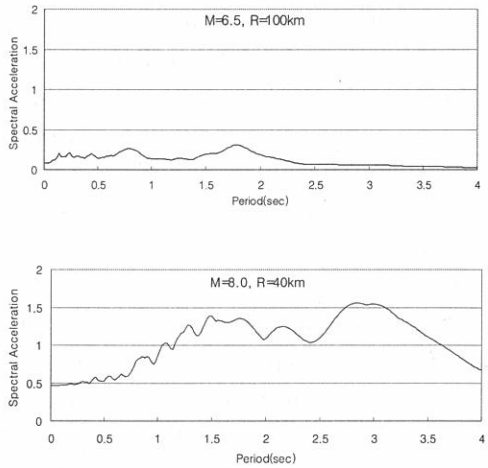

Fig. 3 Acceleration Response Spectra

to the frequency content of these ground motion time histories. For the purpose of response analysis, a sample of ten time histories generated from each M and R combination is matched with a sample of ten bridges in a pseudo Latin Hypercube format. Hence, each statistical representation of Bridges 1 and 2 are subjected to 80 ground motion time histories.

3.3 Fragility Curves

This study utilizes the SAP 2000 finite element code, which can in approximation simulate the state of damage of each bridge under ground acceleration time history. This computer code can provide hysteretic elements that are in essence bilinear without strength or stiffness degeneration.

The states of damage considered for both Bridges 1 and 2 are major (all the columns subjected to ductility demand ≥ 2) and at least minor (all the columns subjected to ductility demand ≥ 1) under the longitudinal applications of ground motion.

Fig. 4 shows the fragility curve of Bridge 1 for exceeding major damage state. Eighty diamonds plotted on the two horizontal axes represent $x_i=0$ (for state of no damage) and $x_i=1$ (for state of major damage) in relation to Eq. (1) under the eighty earthquakes generated. The corresponding fragility curves are derived on the basis of these diamonds in

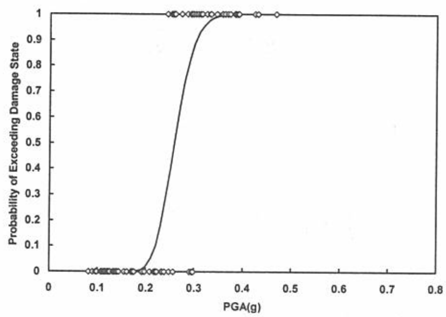

Fig. 4 Fragility Curves for Bridge 1-major damage

conjunction with Eqs. (1)-(3). Fig. 5 shows the fragility curves associated with major and minor states of damage for Bridge 1 and 2. The analysis performed under the ground motion in the transverse direction produced states of lesser damage and hence not reported in this paper.

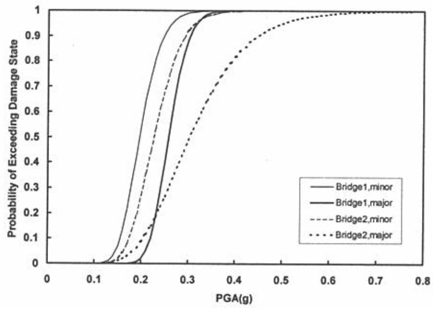

Fig. 5 Fragility Curves for Memphis Bridge

3.4 Development of Combined Fragility Curves

Use of a fragility curve representing a family of bridges with a similar structural attributes, primarily categorized in specific structural types, expedites the process of urban earthquake disaster estimation. A well known example of such a categorization is found in ATC 13(1989). Bridges 1 and 2 in the Memphis area analyzed in an earlier section belong to such a family of bridges that can be categorized as precast prestressed continuous deck bridges with short to medium length. This section demonstrates how a combined fragility curves for a category of bridges can be derived from individual fragility curves constructed for member bridges in the category.

The fragility curves (associated with specific states of damage) analytically developed for Bridges 1 and 2 in the Memphis area can be combined in the following fashion in order to develop a combined fragility curve for a mixed set of population of bridges in which there are and N_1 and N_2, of Bridges 1 and 2 respectively. In this case, the combined fragility curve $F_c(a)$ is obtained as

$$F_c(a) = P_1 \cdot F_1(a) + P_2 \cdot F_2(a) \tag{4}$$

where $F_i(a)$ is the fragility curve for Bridge i and

$$P_i = \frac{N_i}{(N_1 + N_2)} \tag{5}$$

is the probability with which a Bridge i will be chosen at random from the combined population. It is noted that $F_c(a)$ thus developed is no longer a lognormal distribution.

Since the density function of $F_c(a)$ can be written as

$$f_c(a) = P_1 f_1(a) + P_2 f_2(a) \tag{6}$$

and since $f_i(a)$ is a lognormal density function, the expected value of $\alpha = \ln a$ for the combined distribution is given by

$$\overline{\alpha} = P_1 \ln c_1 + P_2 \ln c_2 = P_1 \overline{\alpha_1} + P_2 \overline{\alpha_2} \tag{7}$$

in which c_i is the median associated with $f_i(a)$ and

$$\overline{\alpha_i} = \ln c_i \tag{8}$$

If $\overline{\alpha}$ is defined as

$$\overline{\alpha_i} = \ln a_c \tag{9}$$

then

$$a_c = c_1^{P_1} \cdot c_2^{P_2} \tag{10}$$

In order to obtain the standard deviation ζ_c of $\alpha = \ln a$ of the combined distribution, one recognizes

$$\zeta_c^2 = E(\alpha - \overline{\alpha})^2 = E(\alpha^2) - \overline{\alpha}^2 \tag{11}$$

and

$$\begin{aligned} E(\alpha^2) &= P_1 \int \alpha^2 \phi_1(\alpha)\, d\alpha + P_2 \int \alpha^2 \phi_2(\alpha)\, d\alpha \\ &= P_1 \overline{\alpha_1}^2 + P_2 \overline{\alpha_2}^2 \end{aligned} \tag{12}$$

where $\varphi_i(\alpha)$ is normal density function of α with mean $\ln c_1$ and standard deviation ζ_i. One further recognizes that

$$\begin{aligned} E_i(\alpha - \overline{\alpha})^2 &= \int (\alpha - \overline{\alpha})^2 \phi_i(\alpha)\, d\alpha \\ &= E_i(\alpha^2) - \overline{\alpha_i}^2 = \zeta_i^2 \end{aligned} \tag{13}$$

from which it follows that

$$E_i(\alpha^2) = \int \alpha^2 \phi_i(\alpha)\, d\alpha = \zeta_i^2 + \overline{\alpha_i}^2 \tag{14}$$

Combining equations (7), (11), (12), (13) and (14) gives

$$\zeta_c^2 = P_1\zeta_1^2 + P_2\zeta_2^2 + P_1(1-P_1)\,\overline{\alpha_1}^2 + P_2(1-P_2)\,\overline{\alpha_2}^2 - 2P_1P_2\,\overline{\alpha_1\alpha_2} \qquad (15)$$

The combined fragility curve is not lognormal as explained earlier. It seems reasonable to assume, however, that the combined curve is lognormal with the mean and variance estimated respectively by Eqs. (7) and (14). This approximation is expected to be particularly valid when the bridges under consideration are designed under the same design codes. Fig. 6 shows a combined fragility curve for Bridge 1 and 2. Solid curve is in fact indistinguishable from the lognormal distribution with the median computed (in approximation) from Eq. (10) and log-standard deviation from Eq. (14).

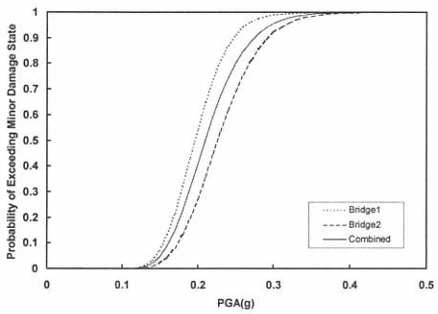

Fig. 6 Combined Fragility Curve for Bridge 1 and 2

4. Conclusions

This study presented a generation of fragility curves for bridge. Analytical fragility curves were obtained for typical bridges in the Memphis, Tennessee area with the aid of dynamic analysis. Two parameter lognormal distribution functions were used to represent the fragility curves with the two parameters estimated by means of the maximum likelihood method.

In addition, the method of combining two fragility curves is presented. Statistical procedures were presented to get the median and log-standard deviation of combined fragility curve. It can be generalized for any number of fragility curves.

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