USE OF SEISMIC RESPONSE MEASUREMENT FOR DAMAGE ASSESSMENT AND CAPACITY ESTIMATION

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ABSTRACT :

This paper presents a structural reliability estimation method incorporating structural parameter identification results based on the seismic response measurement. A large scale shaking table test of a three-bent concrete bridge model was performed. The bridge model was shaken to different damage levels by a sequence of earthquake motions with increasing intensities. Structural parameters, stiffness and damping values of the bridge were identified by extended Kalman filter method. An approach was developed to reveal the importance of structural parameter identification in the reliability estimation. Along this line, a set of bridge models with varying structural parameters were first generated based on the Monte Carlo simulation. Then, each of them was analyzed using nonlinear time history analyses to obtain damage levels at the specific locations. Nonlinear time history analysis rather than nonlinear static analysis (push-over approach) was used to consider the effect of the damping values. Lastly, reliability estimation was performed for two sets of structural parameters. The first set was obtained as discussed above by nonlinear time history analysis with the Monte Carlo simulated parameters which is called non-updated structural parameters. The second one was obtained by updating the first set in Bayesian sense based on the vibration-based identification results which is called updated structural parameters. In the scope of this paper, it was shown that residual reliability of the system estimated using the updated structural parameters is lower than the one estimated using the non-updated structural parameters.

KEYWORDS: seismic damage detection, extended Kalman filter, shaking table test, bridge structure, vibration measurement, Monte Carlo simulation, reliability estimation

1. INTRODUCTION

Estimation of structural reliability was aimed to be the ultimate goal of the structural health monitoring practice by Doebling et al. (1996), but so far little research has been done on this topic. Some of the available efforts along this line are summarized as follows: Park et al. (1985) developed damage index for the reinforced columns and correlate this to the real-world structural damageability. Singhal and Kiremidjian (1998) used the Park-Ang damage index and Bayesian updating method to incorporate the 1994 Northridge Earthquake structural damage inventory into fragility analysis. Shinozuka et al. (2000a) developed empirical fragility curves for the bridge structures using Northridge Earthquake and 1995 Kobe Earthquake using nonlinear dynamic analysis. They compared the fragility functions obtained by nonlinear dynamic and static analysis in Shinozuka et al. (2000b). In addition, they calibrated and verified their fragility model integrating information obtained from empirical, experimental and numerical simulation (Shinozuka et. al. 2003).

To obtain residual structural reliability after a damaging event, one uses a deterministic structure with known structural parameter values to analyze structural responses to different input motions. So the randomness in the response is due to the randomness in the input motion. In this study only one input motion was used for a specific level of shaking; however randomness in the response was due to randomness in the structural parameters. The following steps were followed to determine the residual structural reliability after a damaging event. Monte Carlo simulation was performed to obtain a group of structural parameter. Structural response was obtained using nonlinear time history analyses in terms of rotational ductility at the lower and the upper portions of each column for each structure in the simulated set. Identified structural parameters based on the seismic response measurements were used to update these results. As a result, two different distributions of response values in terms of ductility demand for each level of shaking were obtained: the first one from nonlinear time history analysis using Monte Carlo simulated structural parameters and the other one from Bayesian updated version of the first set based on the identification results. Afterwards, a threshold value was determined in terms of column rotational ductility. Failure probability, P_f, was defined as the probability that demand rotational ductility values exceed the threshold value under a given design earthquake. Residual structural reliability was evaluated using structural parameters either with or without Bayesian updated structural parameter values.

2. VIBRATION-BASED STRUCTURAL PARAMETER IDENTIFICATION

This section presents the application of an extended Kalman filtering (EKF) approach developed previously by the authors (Soyoz and Feng 2007). It was applied for damage assessment by identifying change in structural parameters, stiffness and damping values of the bridge structure under damaging seismic events. The seismic response accelerations analytically simulated using the identified stiffness and damping values agreed well with the measured accelerations, demonstrating the accuracy of the identified parameters.

As shown in Figure1, each of the three bents was supported separately on a shaking table. Each of the bents has two columns. All the columns had the same design cross sections, but the bents were of different heights leading to different transverse stiffness. The shaking tables were driven by input acceleration in the transverse direction. Eleven FBA-11 (Kinemetrics Inc.) type accelerometers were used to obtain the vibration response of the bridge model in the transverse direction, with their locations indicated in Figure 1.

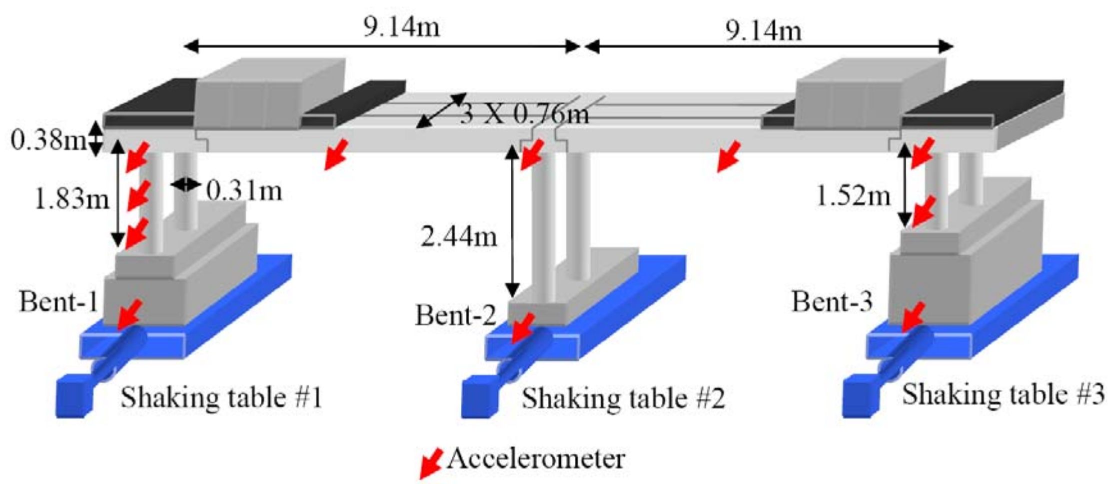

Figure 1 Schematic View of the Bridge Model and Sensor Layout

Strong ground input motions were classified into different levels including low, moderate, high, severe and extreme levels. Table 1 lists the sequence of the tests and the peak ground acceleration (PGA) of the inputs. Three shaking tables were driven by the same signal to produce coherent input. Different levels of damage were observed on the bridge after each strong ground motion. The damage description shown in Table 1 represents the damage observed visually.

Table 1 Test Procedure

Test	Ground Motion Description	PGA (g)	Damage Description
WN-1	White Noise in Transverse	0.07	
T-13	Low Earthquake in Transverse	0.17	Bent-1 yields
T-14	Moderate Earthquake in Transverse	0.32	Bent-3 yields
WN-2	White Noise in Transverse	0.07	
T-15	High Earthquake in Transverse	0.63	Bent-2 yields
WN-3	White Noise in Transverse	0.07	
T-19	Extreme Earthquake in Transverse	1.70	Bent-3 steel buckles
WN-4	White Noise in Transverse	0.07	

During the test, after each strong motion, cracks were marked and photos were taken to document the damage. Some examples were shown in Figures 2 and 3. Due to different transverse stiffness of the bents, dynamic behavior was highly dominated by the torsion demanding high transverse movement for the first and the third bent. This explains the severe damage on these two and comparatively lighter damage on the second bent. Nonlinear time history analyses were performed in SAP2000 (v8). Nonlinearity was modeled as Wen type links, at both the upper and the lower parts of each column, half of the diameter of the column away from the end.

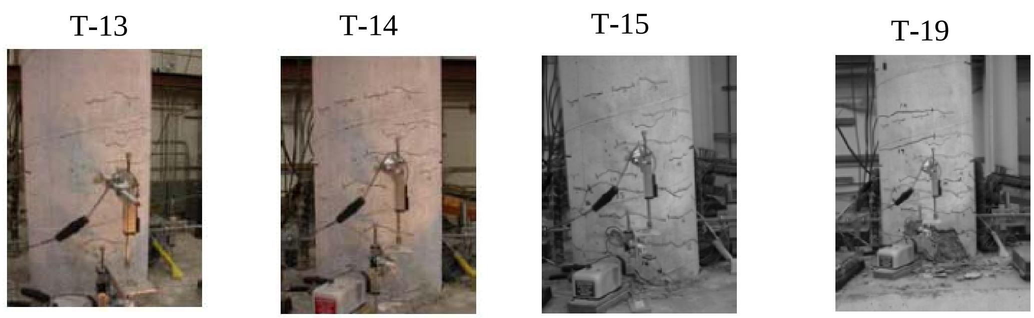

Figure 2 Damage Observed on Bent-1 after Each Test

Figure 3 Damage Observed on the Lower and Upper Portion of Bent-3 after T-19

3. PROPOSED APPROACH FOR STRUCTURAL RELIABILITY ESTIMATION

This section presents the proposed method for structural reliability estimation in more details. Firstly, structural reliability estimation based on nonlinear time history analysis with the Monte Carlo simulated, non-updated parameters were discussed. Then, the structural reliability estimation based on the Bayesian updated parameters was explained.

3.1. Reliability Estimation Based on Non-updated Structural Parameters

Figure 4 shows the flowchart to estimate the reliability using structural parameters generated by Monte Carlo simulation, without Bayesian updating of these parameters.

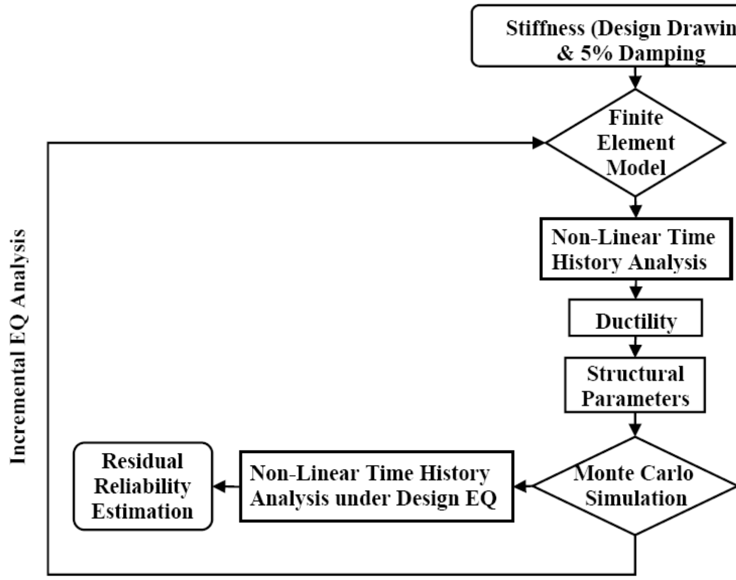

Figure 4 Flowchart of the Residual Reliability Estimation with Non-updated Parameters

Firstly, stiffness values of the bridge were determined based on the structural design drawings. Five percent modal damping values were assumed for the first and second modes and the corresponding Rayleigh damping coefficients were found based on Equation 3.1.

$$C = aK + bM$$
$$\xi_i = \frac{a}{2} w_i + \frac{b}{2}\frac{1}{w_i} \tag{3.1}$$

where C is damping coefficient, K is stiffness and M is mass matrices. *a* and *b* are the two Rayleigh damping coefficients. ξ_i is the damping ratio and w_i is the frequency of the i^{th} mode.

Monte Carlo simulation was performed to generate 20 samples for the six stiffness values, at the lower and the upper portion of three bents and two Rayleigh damping coefficients based on normal distribution. Coefficient of variation for the distributions was 0.1. Nonlinear time history analysis was performed using the first test, T-13 as the input motion. Response values in terms of rotational ductility were obtained at six locations: lower and upper parts of the three bents. Stiffness values were calculated as being the inverse of ductility values and the mean of the stiffness values was found. Equation 3.2 was used to obtain the modal damping value at a specified damage; ductility level and the mean of the damping values were found (Kowalsky et. al. 1994).

$$\xi_{eff} = 0.05 + \frac{1}{\pi}\left(1 - \frac{0.95}{\sqrt{\mu}} - \frac{0.05}{\sqrt{\mu}}\right) \tag{3.2}$$

where μ is the ductility level.

Monte Carlo simulation was performed again based on the mean values of the stiffness and damping coefficients considering they follow normal distribution. The second test, T-14 was used as the input for the nonlinear time history analysis and the same procedure was carried out for the other tests, T-15 and T-19. The stiffness and damping values generated at each damage level were also used in the nonlinear time history analyses under the design earthquake to determine structural response values in terms of rotational ductility. Design earthquake was chosen to be the last event, T-19 due to the fact that bridge structure experienced major damage under this input motion. Finding the remaining structural reliability which is $1\text{-}P_f$ under the given design earthquake then turns out to be straightforward.

3.2. Reliability Estimation Based on Updated Structural Parameters

Figure 5 shows the flowchart to estimate the structural reliability using Bayesian updated structural parameter values based on the seismic response measurement. In this case, response values in terms of rotational ductility were obtained and considered to follow normal distribution as discussed previously; however this distribution was updated using the identified structural parameters based on the vibration measurement. New stiffness values and damping coefficients were determined based on this updated distribution. Structural reliability estimation was performed in the same way. It is emphasized that the pre-event structural parameters, stiffness and damping values are not the design values, but those identified from ambient vibration measurement. Similarly, the post-event structural parameters are not from the simulation, but identified from the seismic response measurement.

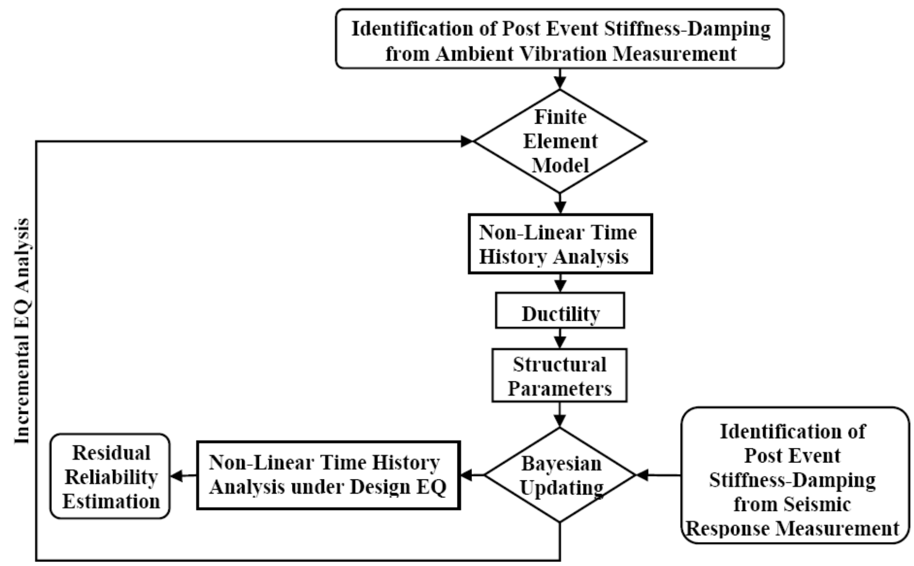

Figure 5 Flowchart for of the Residual Reliability Estimation with Updated Parameters

4. NONLINEAR TIME HISTORY ANALYSIS WITH NONUPDATED STRUCTURAL PARAMETERS

In this section structural response values obtained from nonlinear time history analyses using Monte Carlo simulated parameters, not the measured ones, are discussed. Figure 6 shows the rotational ductility values under four different input motions. The PGA values are: 0.17g, 0.32g, 0.65g and 1.70g. One can easily notice the similarities between this figure and the fragility curves obtained by many researchers for different purposes. The main difference as mentioned before is that the randomness in the response values is due to the randomness in the structural values but not due to the randomness in the input motion. Another observation is that Bent-1 suffers more damage than the other bents due to the torsional behavior in the response. In Figure 6 and the following figures, only the rotational ductility values in the lower portion of each bent are presented, due to the fact that these values are higher than the ones obtained in the upper portions of the bents.

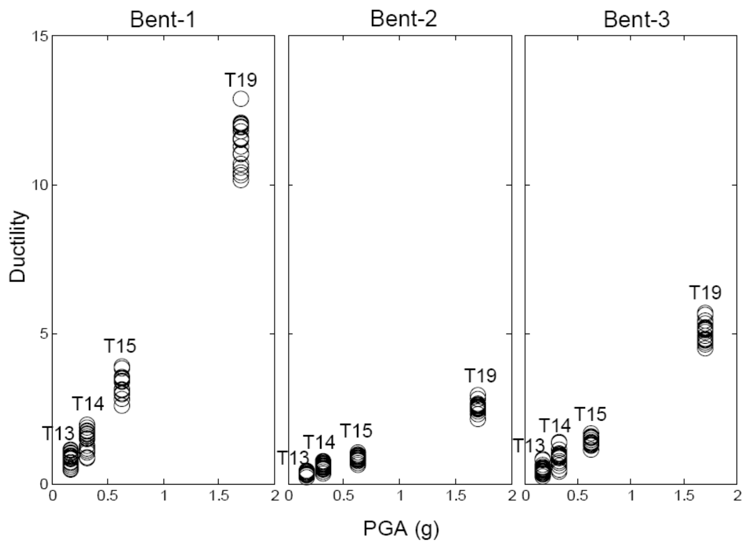

Figure 6 Response Values for Non-updated Structural Parameters

5. NONLINEAR TIME HISTORY ANALYSIS WITH UPDATED STRUCTURAL PARAMETERS

In this section, structural response values obtained from nonlinear time history analyses using Bayesian updated were discussed. The logic behind nonlinear time history analyses was very similar to the one presented in the previous section; however in this case structural parameters were first updated in a Bayesian sense using the identification results.

Figure 7 shows the response values in terms of ductility computed from the updated and non-updated structural values i.e. prior and posterior distributions. The ductility values were bigger when Bayesian updated structural values are used. It implies that actually the structure experienced more damage than the one estimated based on non-updated structural parameter values. Figure 7 also shows that under T-19, Bent-3 experiences more damage than Bent-1 does, when the Bayesian updated structural parameter values are used. This is consistent with the experimental observation. However, if non-updated structural parameters are used, Bent-1 experiences more damage than Bent-3 does. This demonstrates that integration of the vibration-based identification results not only results in more reliable estimation of the ductility but also the failure mode.

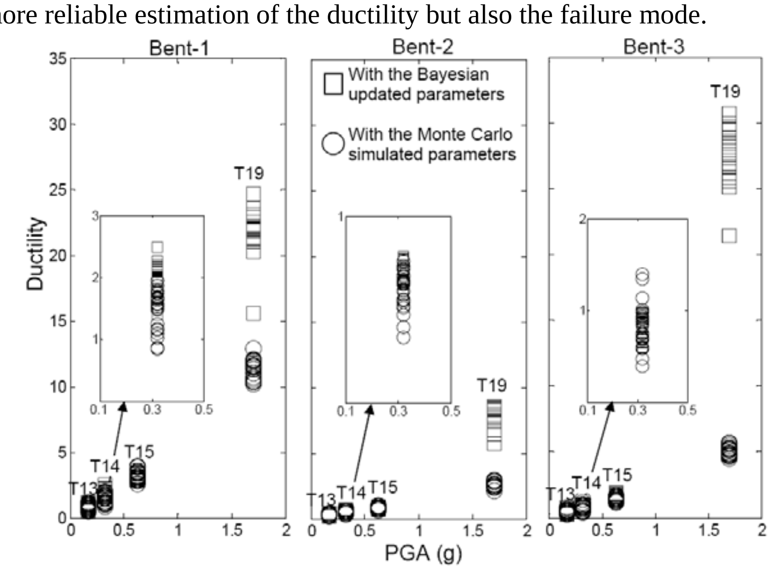

Figure 7 Incremental Response Values for Simulated Structural Values

6. STRUCTURAL RELABILITY ESTIMATION

In this section, residual reliability estimation of the bridge structure was performed. In section 5, two distributions for the structural parameters were obtained; namely updated and non-updated distributions. In this section, these distributions were used for the nonlinear time history analyses under design earthquake. Figure 8 shows the residual reliability of the bridge structure under design earthquake after each damaging event. Design earthquake was chosen to be T-19 which had the input acceleration level sufficiently high to cause the structure to have major damage. The threshold level was determined based on Banerjee and Shinozuka (2008), in which they performed experimental observations and suggested different damage levels such as minor, moderate, major in terms of the ductility demand. In this research, the threshold rotational ductility was taken to be 9.42, above which major damage was reported by Banerjee and Shinozuka (2008).

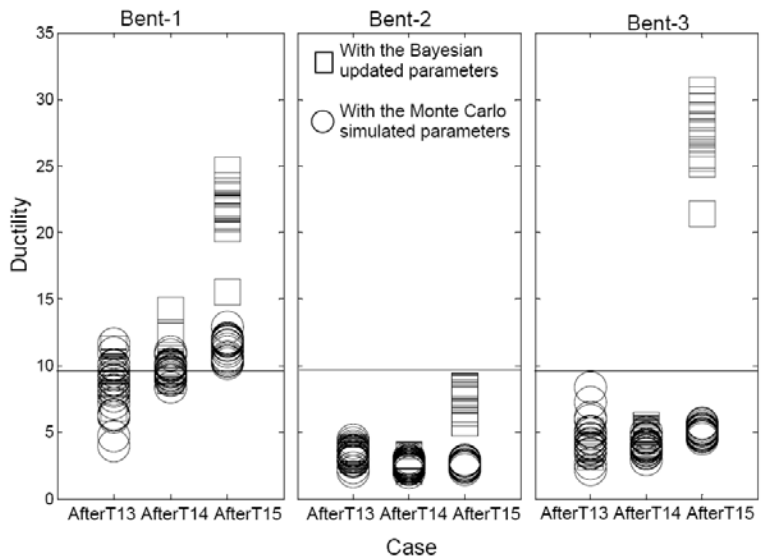

Figure 8 Residual Structural Reliability Estimation

Firstly, normal distribution is fit to the ductility response values after each event. Afterwards, the structural reliability was determined as 1-P_f. Table 2 clearly shows that in the scope of this example the remaining capacity is lower if the identification results from the measurement are incorporated. In other words, the estimation based on the non-updated structural parameters will overestimate the reliability of the structure.

Table 2 Residual Structural Reliability Values

	After T13	After T14	After T15
Bayesian Updated	53%	23%	0%
Non-updated	75%	44%	3%

7. CONCLUSION

This paper presents the use of the structural parameter values identified based on vibration measurement and updated in Bayesian sense for the estimation of the reliability of a bridge structure after a damaging event. In this study, a large-scale shaking table test of a three-bent concrete bridge model was performed in order to verify the proposed reliability estimation method. The bridge model was shaken to different damage levels by a sequence of earthquake motions with increasing intensities. The stiffness and damping values of the structure were instantaneously identified in real time during the damaging earthquake excitations, using the EKF approach previously developed by the authors. Based on the identified stiffness and damping values, residual structural reliability of the bridge was estimated. Following conclusions were made:

- In the scope of this paper, it was shown that structural reliability estimated using the Bayesian updated structural parameters was lower than the one estimated using non-updated structural parameters mainly due to fact that the level of both stiffness and damping were considerably different after the updating procedure.
- Slight damage on Bent 1 was identified after T-13 based on vibration measurement. This level of damage is difficult to assess by visual inspection. Nonlinear time history analysis using the design values of the structural parameters could not simulate the stiffness degradation either.
- After T-14, damage could be inspected visually. The extent of the damage could be determined by nonlinear time history analysis, but it was always lower than the identification results based on vibration measurement.

These observations clearly reveal the importance of structural parameter identification both for post-event damage assessment and residual reliability estimation.

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