Damage Detection and Reliability Estimation Using Earthquake Response Measurement

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SUMMARY:
This paper presents a structural reliability estimation methodology incorporating vibration-based structural parameter identification results. This approach was developed to reveal the importance of structural parameter identification in reliability estimation process. This methodology was applied on a three-bent concrete bridge model which was shaken simultaneously with three shaking tables to different damage levels by a sequence of earthquake motions with increasing intensities. First, modal frequencies and shapes were identified using vibration response of the bridge. Then, stiffness values of columns were obtained by finite element model updating methodologies. Finally, reliability estimation was carried out using fragility curves of updated and non-updated finite element models. Fragility curves at a given damage state were obtained by non-linear time history analyses. At a given damage state, updated model was constructed using stiffness values obtained from vibration-based identification whereas non-updated model was constructed using stiffness values obtained from non-linear time history analyses. In the scope of this paper, it was shown that residual reliability of the system estimated with the updated structural parameters is significantly different than the one estimated with the non-updated structural parameters.

Keywords: Vibration-based identification, Structural Reliability, Fragility Curve, Shaking Table Test

1. INTRODUCTION

Structural reliability estimation is the ultimate goal of the structural health monitoring (Doebling et.al, 1996); but so far little research has been done on this topic. Some of the available efforts along this line can be summarized as follows. Damage index for the reinforced columns and correlation of this index to the real-world structural damageability was introduced (Park et.al, 1985). A methodology to update structural model and their uncertainties based on measured data in Bayesian framework was developed (Beck and Katafygiotis, 1998 and Katafygiotis and Beck, 1998). Park-Ang damage index and Bayesian updating method was utilized to incorporate the 1994 Northridge Earthquake structural damage inventory into fragility analysis (Singhal and Kiremidjian, 1998). Empirical fragility curves for the bridge structures using Northridge Earthquake and 1995 Kobe Earthquake using non-linear dynamic analysis were developed (Shinozuka et.al, 2000a) and the fragility functions obtained in that study by non-linear dynamic and static analysis were compared (Shinozuka et.al, 2000b). In addition, the fragility model integrating information obtained from empirical, experimental and numerical simulations was calibrated and verified (Shinozuka et.al, 2003).

To obtain residual structural reliability after a damaging event, one can use a finite element model with known structural parameters to obtain response to different input motions. It was also shown that differences in structural parameters affected structural reliability (Soyoz et.al, 2010).

In this study, structural reliability is expressed by fragility curves and the importance of system identification is revealed by obtaining fragility curves both for updated and non-updated models at a given damage state. Stiffness values for updated model are obtained from vibration-based identification results and stiffness values for non-updated model are obtained from non-linear time history analyses. In the following sections, first, experimental setup and procedure is explained and then system identification procedures are discussed. Finally, reliability estimation procedure is presented.

2. EXPERIMENTAL SETUP AND PROCEDURE

The shaking table experiment was conducted at the University of Nevada, Reno on the behalf of NEES projects (http://nees.unr.edu). Figure 2.1 shows the bridge model which is a two-span reinforced concrete structure with three bents. Each bent consists of two columns with the same circular cross-section which is 30.5 cm in diameter and has 16 #3 reinforcements. The columns differ in length; therefore, the stiffness values of bents are different, which is especially influential in transverse modal characteristics. Additional masses represent the mass source of adjacent spans of a typical bridge structure. Acceleration sensors are located at eleven locations of the bridge to measure vibrations throughout tests.

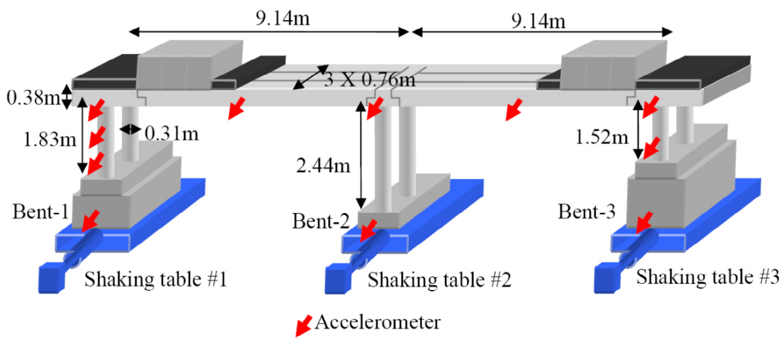

Figure 2.1. Schematic view of the model and sensor layout

Tests involve several damaging events and white noise excitations between damaging events. Ground motions are imposed to the structure by three shaking tables acting on each bent. Therefore, each bent is exposed to excitations separately, but simultaneously. In other words, the structure experiences multiple support excitations, which could be represented with uniform excitation, as the difference of inputs imposed to different bents is insignificant. Throughout the earthquake excitations, the structure experienced progressive damage. Each white noise excitation corresponds to a different damage state which is to be quantified and located with structural parameter identification using vibration response measurements. Moreover, visual inspection and strain gauge measurement were carried out along with the tests. Earthquake excitations were classified into different levels including low, moderate, high, severe and extreme. Table 2.1 lists the sequence of tests and corresponding peak ground acceleration (PGA) of the inputs. Different levels of damage were observed on the bridge after each strong ground motion. Table 2.1 presents visually observed damage and Figure 2.2 shows the time history of earthquake and white noise excitations.

Table 2.1. Test procedure

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

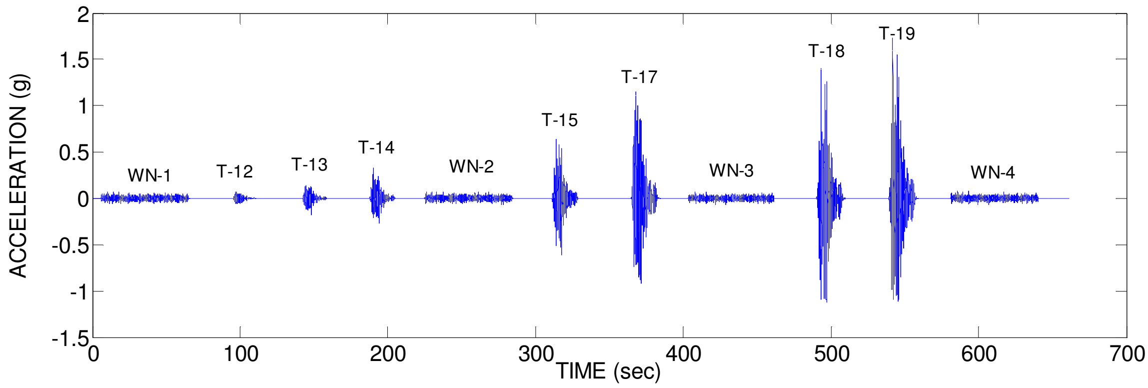

Figure 2.2. Input motions

After each strong motion, cracks were marked and photos were taken to document the damage. Some examples were shown in Figure 2.3 and Figure 2.4. Due to different transverse stiffness of the bents, dynamic behavior was highly dominated by the torsion demanding high transverse movement of the first and third bent. This explains severe damage on these two bents and comparatively light damage on the second bent.

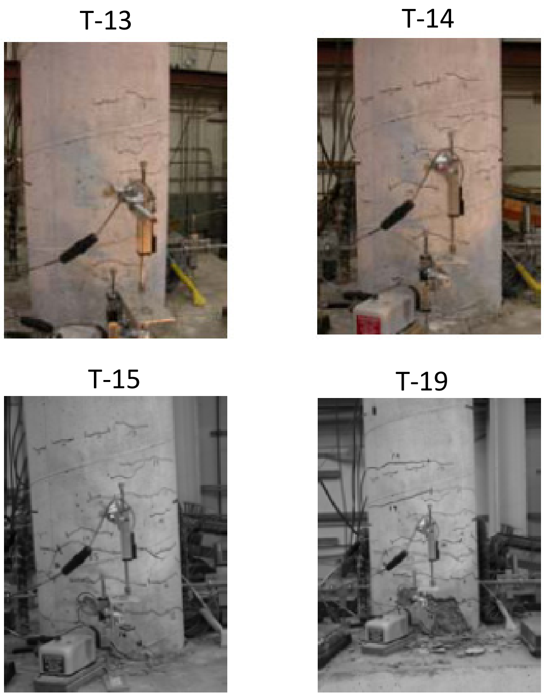

Figure 2.3. Damage observed on Bent-1 after each test

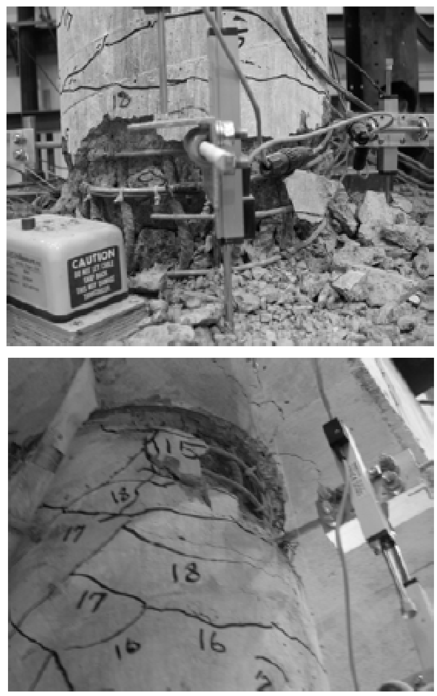

Figure 2.4. Damage observed on the lower and upper portion of Bent-3 after T-19

3. SYSTEM IDENTIFICATION

White noises are the excitations with low amplitudes. Therefore, vibration characteristics of the structure at white noise excitations could be represented in linear range. Frequency domain decomposition (FDD) method was used to obtain modal frequencies and shapes of the structure. Figure 3.1 shows the power spectral densities obtained at each white noise excitation. Decrease in modal frequencies due to damage can be easily observed.

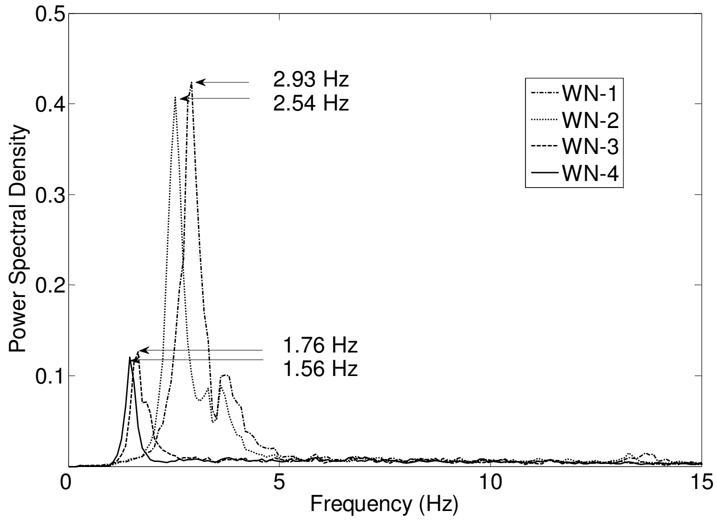

Figure 3.1. Identified modal frequencies

In addition to experimental determination, modal values were also obtained analytically. To do this, the finite element model of the bridge was developed in OpenSees platform. Figure 3.2 shows modal frequencies and shapes obtained analytically and experimentally using WN1.

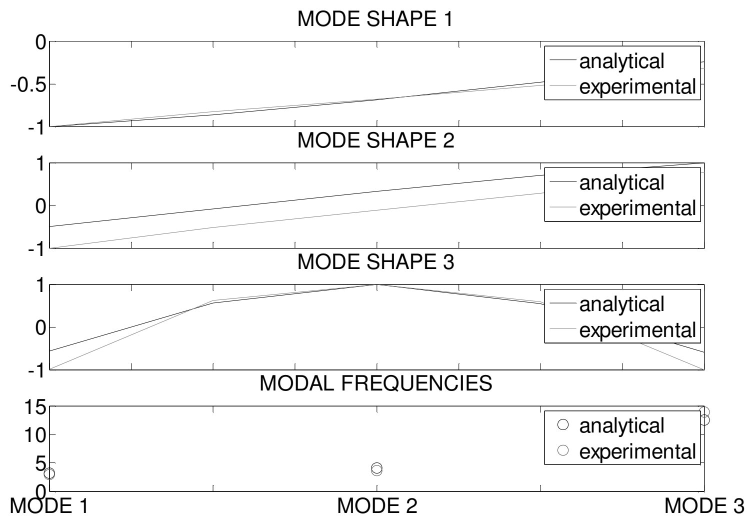

Figure 3.2. Experimental and analytical modal values

4. DAMAGE DETECTION

Damage detection i.e. stiffness identification for updated and non-updated models was performed as follows.

For non-updated model, column end rotation is the damage indicator, where ductility demand of a damaging event is reversely proportional with stiffness coefficients of damaged zones i.e. effective stiffness of plastic hinges at column ends. Hinge behavior could be idealized by elastic-perfectly plastic moment rotation relationship. This relationship can be obtained from moment-curvature behavior of the cross-section as shown in Figure 4.1. For each non-linear time history analysis, the damage state is evaluated based on maximum hinge rotations of bents.

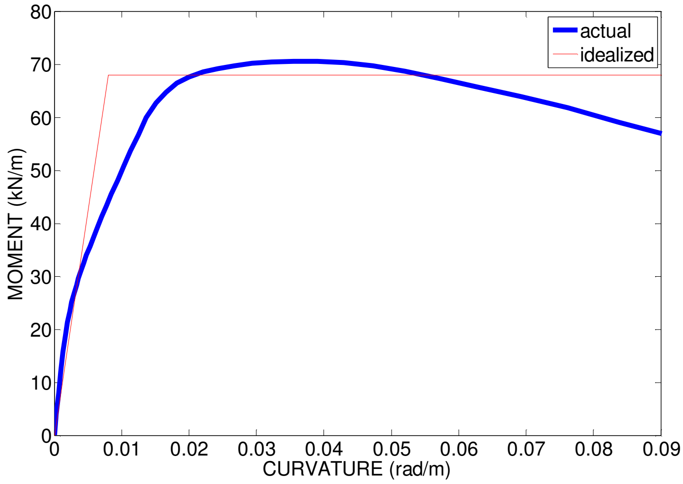

Figure 4.1. Moment-Curvature relationship

For updated model, stiffness coefficients of damaged zones are to be obtained using vibration-based identification results. For this, a set of finite element model was developed with different bent stiffness values which resulted in a set of modal frequencies and shapes. The similarity of modal values obtained analytically and experimentally creates an error. This error was represented with an objective function which involves first, second, third modal frequencies, and first mode shape of structure. System identification would imply the determination of bent stiffness values which minimize the objective function.

As a result, corresponding bent stiffness values were assigned at each damage state to develop updated and non-updated finite element models. Figure 4.2 shows the damage progress both for updated and non-updated models and Table 4.1 summarizes the stiffness values.

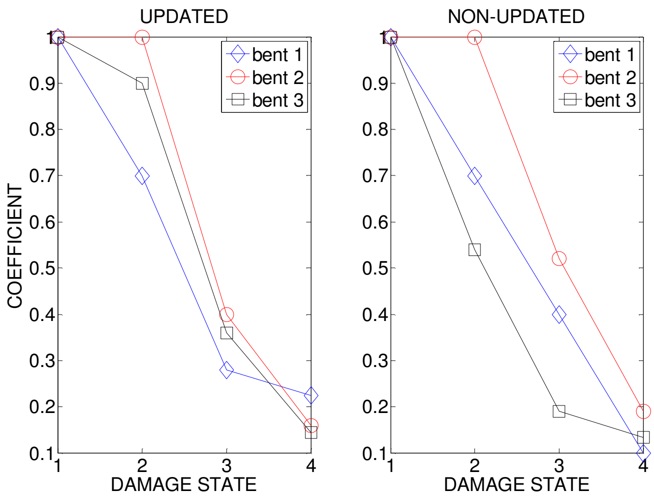

Figure 4.2. Damage progress

Table 4.1. Stiffness values

		WN1	WN2	WN3	WN4
UPDATED	BENT 1	1.00	0.70	0.28	0.22
	BENT 2	1.00	1.00	0.40	0.16
	BENT 3	1.00	0.90	0.36	0.14
NON-UPDATED	BENT 1	1.00	0.70	0.40	0.10
	BENT 2	1.00	1.00	0.52	0.19
	BENT 3	1.00	0.54	0.19	0.13

5. RELIABILITY ESTIMATION

Reliability of a structure can be estimated by fragility curves for a broad range of demand parameters. Demand parameters can be expressed in different ways but still the most popular one is PGA. In the current state-of-the-art, fragility curves are developed analytically and then if possible updated empirically. In this study, only the analytical development of fragility curves is discussed. In the analytical development of fragility curves, certain damage levels such as almost no, minor, moderate, major, and collapse are considered and failure or no failure of a structure is determined by non-linear time history analyses. In this study, 60 input ground motions with PGA ranges from 0.1 to 1.3 g are considered. Rotational ductility demand, M, was obtained by dividing the rotation value from analyses by the yield rotation value. Damage states were determined using threshold values in Table 4.2 (Dutta and Mander, 1998). Maximum likelihood method is used to determine the mean values, and standard deviations of fragility curves.

Table 4.2. Damage states and rotational ductility capacities

Damage State	Limiting Rotational Ductility
Almost no	$1.00<M<1.52$
Minor	$1.52<M<3.10$
Moderate	$3.10<M<5.72$
Major	$5.72<M<8.34$
Complete	$8.34<M$

In this study, fragility curves of undamaged structure and the structure at a given damage state were obtained. Therefore, the effect of damage on the reliability of the structure can be observed. Figure 5.1 shows the fragility curves of undamaged structure.

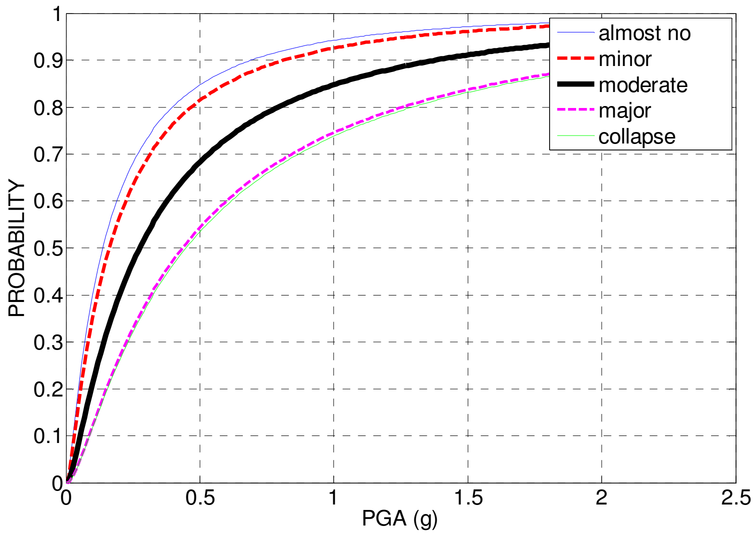

Figure 5.1. Fragility curves for undamaged structure

Figure 5.2 shows the fragility curves for non-updated and updated models after T13 and T14. Based on this figure, it can be concluded that incorporation of vibration-based identified parameters into reliability estimation process has decreased the estimated reliability values.

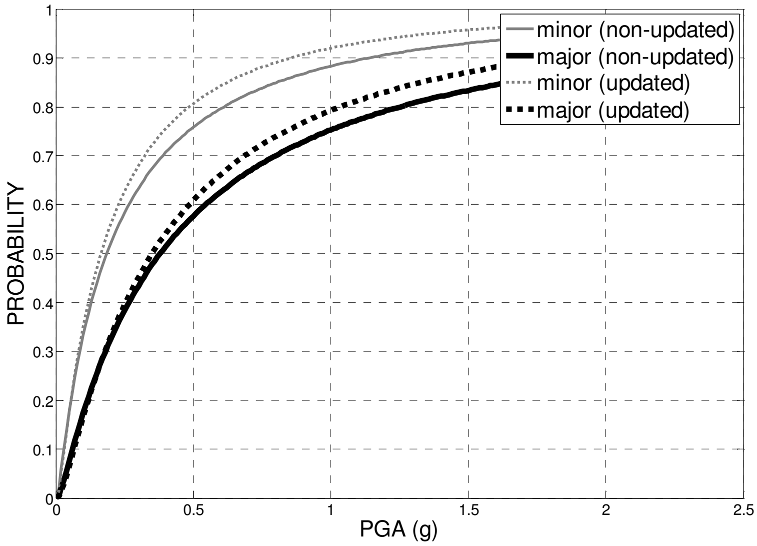

Figure 5.2. Fragility curves for updated and non-updated models

6. CONCLUSION

This paper reveals the importance of structural parameter identification both for post-event damage assessment and residual reliability estimation. 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. Modal frequencies and shapes of the bridge were identified and stiffness values of columns were updated accordingly. Based on the identified stiffness values, residual reliability of the bridge was estimated using fragility curves. In the scope of this paper, two major conclusions can be drawn as follows:

The extent of the damage determined by non-linear time history analyses was different than the vibration-based identification results. For example, after T13 and T14, stiffness values of the column plastic hinge regions in terms of original values of bent-1, bent-2 and bent-3 are 0.7, 1.0 and 0.54 for non-updated model and 0.7, 1.0 and 0.9 for updated model.

As a result of the difference in stiffness values, structural reliability estimated using updated and non-updated models was also different. For example, probability of having major level damage is 80% for updated model and 75% for non-updated model for a PGA of 1g.

ACKNOWLEDGEMENT
The shaking table experiments were conducted at University of Nevada, Reno in conjunction with Dr. Saidii's and Sander's NSF-NEES project. The authors are thankful for conducting vibration measurements during shaking table tests.

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