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VIBRATION-BASED DAMAGE DETECTION OF BRIDGES UNDER INCOHERENT INPUT MOTIONS

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ABSTRACT

The aim of this paper is to represent a damage detection methodology for a bridge model via system identification results obtained from vibration measurements. A series of earthquake and white noise excitations were imposed to a three bent, two-span, reinforced concrete bridge by three different shaking tables, simultaneously at NEES-University of Nevada Reno. In real-life structures, the possibility of the bents to be exposed to identical inputs is low because of several reasons such as distance between bents or different soil types at which abutments are located. Therefore this study aims to observe the effect of incoherent inputs to the bents. The bridge model was shaken both by white noise and strong motion inputs as shown in and system identification techniques are used for white noise excitations and the damage level due to strong motions are determined. Moreover, the nonlinear analysis of the finite element model constructed via design drawings are carried out in order to predict the damage state of the structure when there is no vibration record is available. Finally, two results obtained from identification and finite element analysis are compared with each other and it is observed that there is a significant difference.

INTRODUCTION

Sophisticated highway system in a metropolitan area is supported by hundreds of bridges and viaducts. Lack of information about the post-earthquake structural integrity of these bridges can cause safety hazards to the traveling public, halt mobility of the transportation network, and disrupt emergency response. The current practice relies on visual inspection for damage detection, which is time consuming, insufficient, subjective and requires presence of the crew on the structure that is potentially hazardous after an earthquake.

Structural condition assessment of highway bridges has long relied on visual inspection, which involves subjective judgment of inspectors and detects only local and visible flaws. The frequency of visual inspection and the qualification of the inspectors were regulated by the National Bridge Inspection Standards (1996). The Federal Highway Administration (FHWA) Recoding and Coding Guide (1995) also provides guidance in terms of the condition ratings and the documentation in current practice. Even with these provisions, a recent investigation initiated by FHWA to examine the reliability of visual inspections reveals significant variability in the structural condition assignments by the inspectors (Phares et al., 2004). Moreover, visual inspection cannot quantitatively evaluate remaining capacity of a bridge. The Long Term Bridge Performance Program (LTBP) was recently initiated by FHWA, exploring sensor-based continuous monitoring of bridges under traffic

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conditions as well as during extreme events such as earthquakes (http://www.tfhrc.gov/structur/ltbp.htm).

Sensor-based structural health monitoring (SHM) can revolutionize the way of inspecting structures, particularly for post-earthquake damage assessment in a rapid, remote, automated and objective fashion. By installing appropriate sensors at critical locations on a bridge, transmitting the sensor data through a communication network, and analyzing the data through a software platform, the location and severity of bridge damage caused by earthquakes can be automatically, remotely and rapidly assessed.

Theoretical background of system identification and finite element model (FEM) updating techniques can be found in studies by Maia and Silva (2001) and Friswell et al. (2001). Some of the comprehensive reviews related with SHM are given by Doebling et al. (1996), Sohn et al. (2004), Carden and Fanning (2004), Brownjohn (2007). Application of these techniques to civil infrastructures based on vibration measurement can be found extensively in the literature (e.g., Beck and Jennings, 1980; Safak, 1991; Celebi and Safak, 1991; Ghanem and Shinozuka, 1995; Lus et al., 1999; Masri et al., 2000; Feng et al., 2003; Skolnik et al., 2006; Moaveni et al., 2010).

EXPERIMENTAL SETUP AND PROCEDURE

A two-span three-bent reinforced concrete bridge structure is exposed to a series of earthquake and white noise excitations by three shaking tables, simultaneously. As shown in Figure 1, each bent consists of two columns with the same circular cross-section, but differs in height. As a consequence, lateral stiffness value of each bent is different, affecting transverse modal characteristics. Additional masses are anchored to the deck, to represent adjacent spans of a typical bridge structure. Accelerometers are placed at eleven locations including the deck and the columns to measure vibration response throughout the tests with a sampling frequency of 100 Hz. First, sixth, and ninth sensors are located on bottom; fourth, seventh, and eleventh sensors are located on top of Bent-1, Bent-2, and Bent-3, respectively.

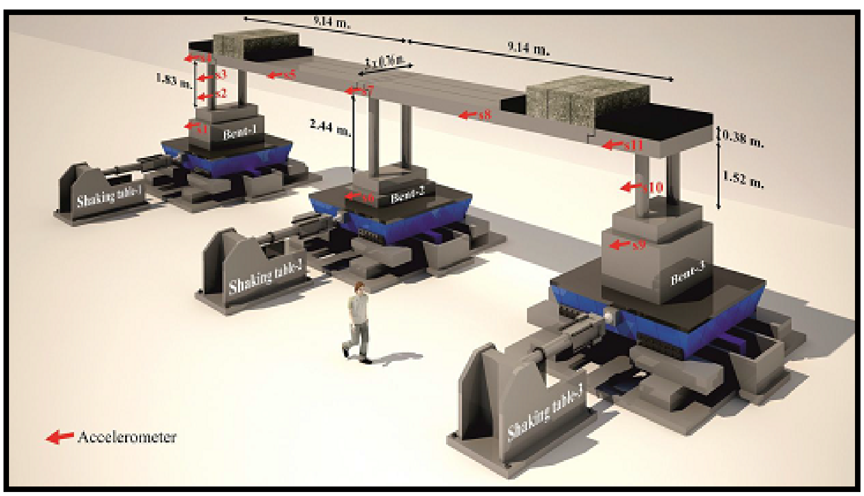

Figure 1: General Representation of Test Setup

Shaking table tests include incoherent earthquake excitations (damaging events with narrow-banded frequency content) and white noise excitations (non-damaging events with broad-banded frequency content) at the beginning and the end of each damaging event. Each bent is exposed to excitations by separate shaking tables. The whole ground motion excitation is illustrated in Figure 2 and Figure 3 shows the Input Motions of Test-1 for three bents.

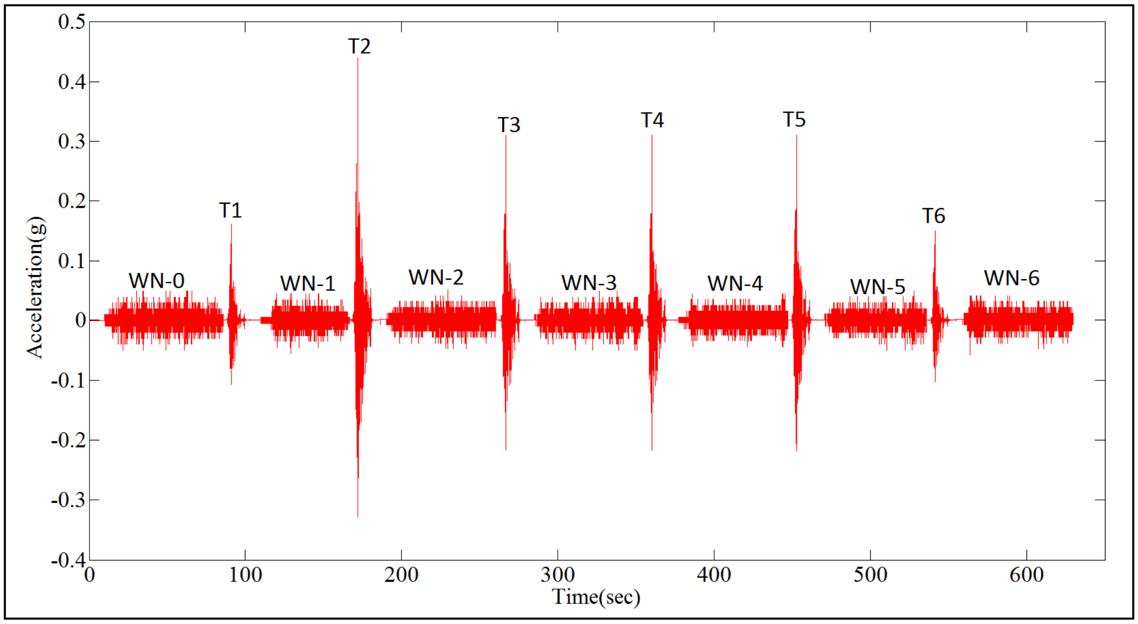

Figure 2: Input Motion

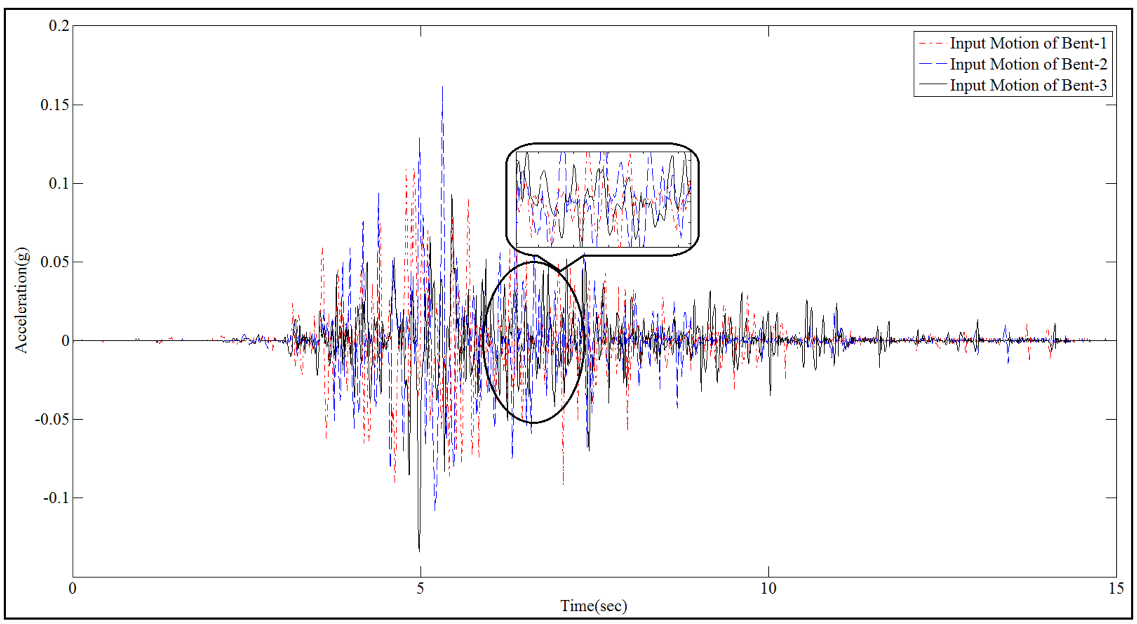

Figure 3: Different Input Motions of Test-1 for Three Bents

SYSTEM IDENTIFICATION

In this study, system identification is carried out in the frequency domains using vibration measurements of white noise excitations (low amplitude, non-damaging events). The identification strategy is based on the fact that the system is linear time-invariant, which means that structure experiences no damage-change throughout the observation.

Nominal FEM is developed in SAP2000 V15 based on design drawings, and code requirements. Inspecting the structural system, it is expected that the deck would experience insignificant damage, while the vast amount of damage will take place on the columns and mostly on the column ends. In other words, structural parameters which are the major sources of change in the system, are the stiffness values of the bents.

The states of a structure are defined as the initial state, and the damaged states. In the initial state, where no damage is expected, system identification is utilized for determining the stiffness of column as a whole, as each portion of the column will have the same material and sectional properties. For a damaged state, identification is carried out only for the column-end regions as the major damage is expected at those regions. A set of candidate finite element models are generated whose modification factors range between 0.2 and 1. Identified parameters refer to the modification factors of columns which minimize the error between the simulated and the measured vibration responses.

FREQUENCY DOMAIN IDENTIFICATION

Structural parameter identification in the frequency domain has two steps. The first step is the identification of the modal values using the acceleration measurements of the bridge. The second step is the minimization of the error between the modal values obtained from the FEM and the measurements.

An output-only method, the Frequency Domain Decomposition (FDD) method (e.g., Otte et al. 1990, Brinker et al. 2001) was used to extract modal parameters from the vibration measurements without requiring information for input. The FDD method is capable of identifying closely coupled modes, thus obtaining better estimates compared to other modal identification methods (Otte et al. 1990). In this method, spectral density matrix S (w) YY of the response vector ***Y(t)*** is decomposed by singular value decomposition, as illustrated in the
Equation 1,

$$S_{YY}(w) = U(w)\sum(w)U^{H}(w) \, , \qquad (1)$$

where
$\Sigma(w)$ = diagonal matrix of the singular values,
$U(w)$ = unitary matrix of the singular vectors,
the superscript H denotes the complex conjugate and transpose.

It has been shown by (Otte et al. 1990) that, when the structure is loaded with the broadband excitation, near the modal frequencies, $\Sigma(w)$ contains a set of functions which are approximations of the auto-spectral density functions of the modes' equivalent single degree of-freedom systems in the normal coordinates, while the vectors in $U(w)$ are the modal shapes of the corresponding modes. Figure 4 shows power spectra of the first singular value for different damage states. With the help of the figure, increase in structural damage could be tracked by observing the decrease in modal frequencies.

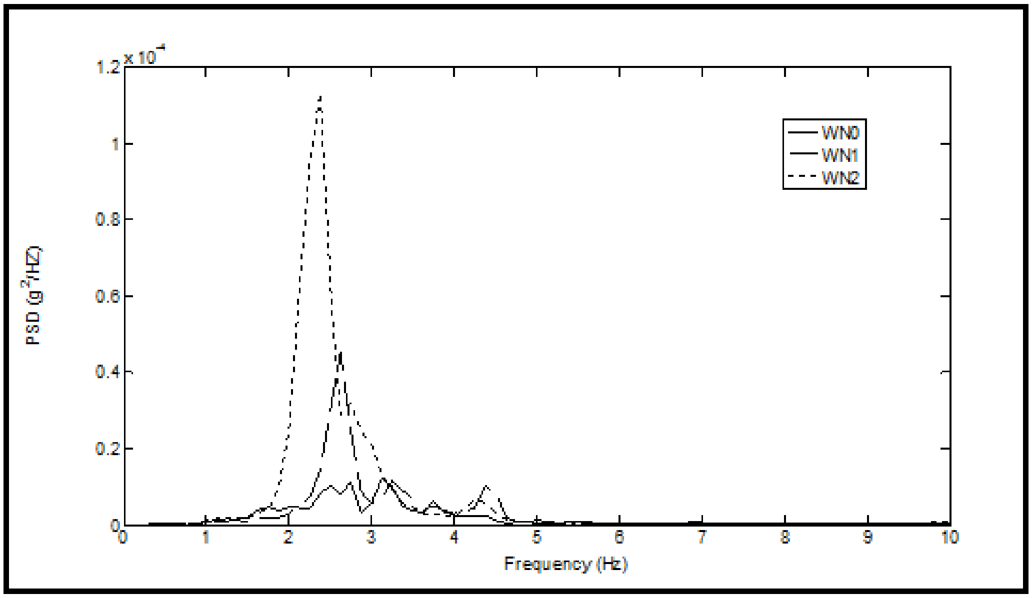

Figure 4: Natural Frequencies of Different Whiten Noise Excitations

In this study, the summation of the Test-1 and the Test-2 is chosen as the earthquake excitation since the most of the nonlinear behavior is observed in the second test. Consequently White Noise-0 and White Noise-2 are considered as the pre-earthquake and post-earthquake states. Modal parameters, such as 1^{st}, 2^{nd} and 3^{rd} modal frequencies are obtained as in Table 1 and the Figure 5 presents the first three mode shape of the WN-0 and WN-2.

Table 1: Identified modal frequencies

	Modal frequencies (Hz)		
Motion	Mode 1	Mode 2	Mode 3
WN 0	2.75	3.125	10.81
WN 2	2.18	2.81	10.19

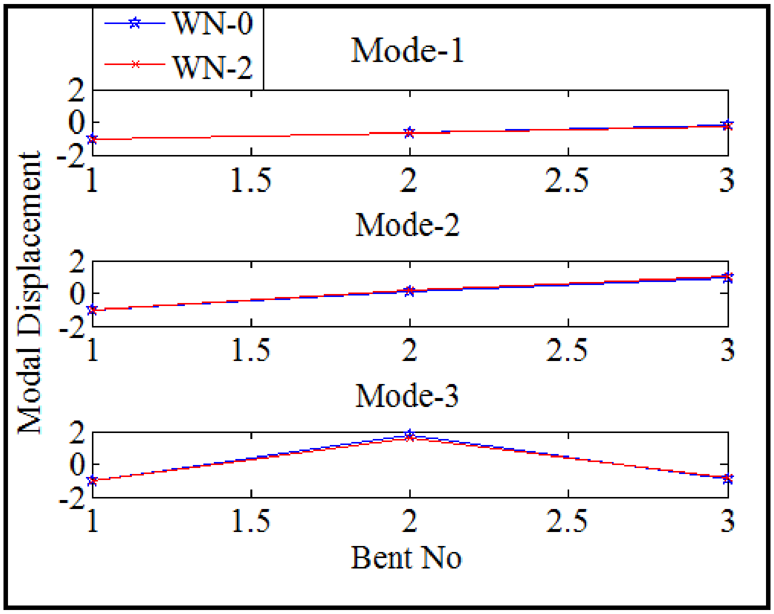

Figure 5: Identified Mode Shapes (WN-0 & WN-2)

As the second step, minimization between the modal values obtained from the FEM and the measurements was performed. Error function considers modal frequencies, mode shapes and weighing coefficients depending on the confidence level of corresponding modal parameter. Simulated modal frequencies and mode shapes are obtained from eigenvalues and eigenvectors of finite element models, respectively; whereas measured modal frequencies and mode shapes are obtained from FDD. Error function, defined in Equation 2, characterized by bent stiffness values, is defined as

$$E(\alpha_1,\alpha_2,\alpha_3)=\sum_{i=1}^{3}\left(k_i.\left[\left(f_i^*-f_i\right)/\,f_i\right]^2+h_i.\left[1-MAC_i\right]^2\right), \qquad (2)$$

where

i : mode number

k_i : weighing coefficient for i^{th} modal frequency

h_i : weighing coefficient for i^{th} MAC value

f_i^* : measured modal frequency of i^{th} mode

f_i : simulated modal frequency of i^{th} mode

MAC_i : modal assurance criteria between the i^{th} mode shapes obtained from simulation and measurement

Modal Assurance Criteria defines the similarity between two mode shapes; here, it defines the similarity between the modes shapes obtained from FEM simulation and measurement. Weighing coefficients are determined to represent the confidence levels of modal parameters. Modal frequency and mode shape of the first mode should be accurately estimated as it is the primary representative of vibration characteristics of a structure. Accordingly, the first, the second and the third modal frequencies and the first mode shape are intended to have high accuracies, and k_1, k_2, k_3 and h_1 are all set equal to 1 and the remaining weighing factors are set equal to 0.

In Table 2, the accuracy of parameter identification procedure is presented. The difference between the first modal frequencies obtained from the FEM and the measurements reduces from 8.3% to 0.4% due to updating.

Table 2: The Comparison of the Modal Frequencies of the Updated and Non-updated FEMs with Identified ones for the Pre-Earthquake State

	System Identification	Non-Updated FEM	Error (%)	Updated FEM	Error (%)
Mode 1	2.75 Hz	3.28 Hz	19.27	2.72 Hz	1.09
Mode 2	3.125 Hz	4.25 Hz	36	3.30 Hz	5.6
Mode 3	10.81 Hz	9.17 Hz	15.17	8.77 Hz	18.87

NONLINEAR TIME HISTORY ANALYSIS

The purpose of this section is to give an outline to predict stiffness values when vibration measurement is not available. In this section, modeling assumptions for nonlinear time history analyses are explained. In reinforced concrete bridge structures, a widely used damage indicator is rotation at column ends. Certain nonlinear modeling techniques would provide different results, and different accuracies. Definition of stiffness and strength degradation, and decision whether plasticity is concentrated or distributed plays an important role in nonlinear analysis (e.g. Yazgan and Dazio, 2011). In this study, it is a reasonable approach to define nonlinearity with rotational springs representing concentrated plasticity at upper and lower column-end regions as the effect of damping in response turns out to be more important than the effect of stiffness. In this context, moment-curvature relationship of cross-section is developed using Response-2000 (Bentz and Collins, 2000) sectional analysis program. Figure 6 shows the cross-sectional properties of the column and its momentcurvature relationship as elastic-perfectly plastic behavior. In literature different rules are suggested to obtain this bilinear behavior. In this study, rules suggested by Gardoni et al. (2002) were adopted.

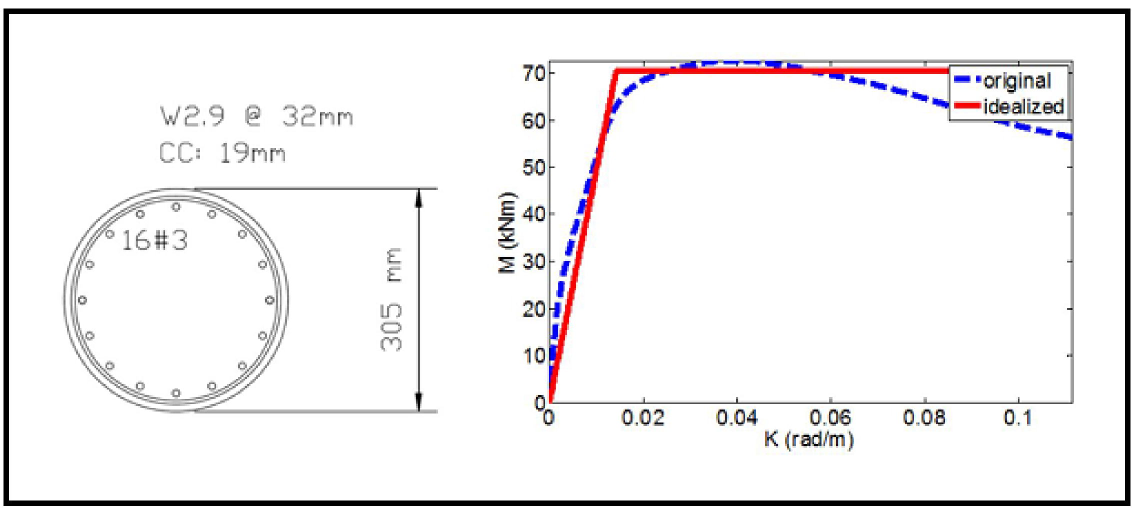

Figure 6: Cross-Sectional Properties and Moment-Curvature Relationship of Columns

Equations 3, 4, and 5 relate the sectional moment curvature behavior with effective stiffness. Yield curvature could be interpreted in terms of yield rotation such as

$$\phi_y = \frac{\theta_y}{L_p} \qquad (3)$$

Where
ϕ_y is yield curvature,
θ_y is yield rotation,
L_p is plastic hinge length.
A relationship between stiffness and moment-curvature could be established through such as

$$EI = \frac{M}{\phi} \qquad (4)$$

where EI is initial stiffness,
M is moment,
ϕ is curvature,

which could be used to define effective stiffness in terms of initial stiffness and ductility (Priestley, 1998) such as

$$EI_{eff} = \frac{EI}{\mu}, \qquad (5)$$

where EI_{eff} is effective stiffness,
μ is ductility.

In other words, after a damaging earthquake event, the stiffness of the damaged region, and the primary stiffness of moment-rotation behavior would be inversely proportional with damage severity designated with ductility. Figure 7 shows the response of Bent-1 in terms of rotation.

The procedure of the nonlinear time history analysis due to an incoherent ground motion has some differences from a classical nonlinear analysis. The defined time history functions of any acceleration record are directly designated to all supports by SAP2000. Therefore, the finite element analysis of a multi-support excitation will not be carried out with acceleration records. On the other hand, any displacement time history function will be sufficient in this type of analysis since dissimilar displacement records can be assigned as a joint load to different support locations. However, in the real-life structures, it is almost impossible to obtain a displacement time history record because of the lack of a stationary reference frames. Hence the acceleration data sets must be converted to the displacement ones. Two methods, one of which is mathematical based and the other one is finite element analysis based, are suggested for this process. In this study, the second suggestion is carried out. Once the displacement time history of each shaking table is obtained, they can be assigned to related supports with a unit displacement load pattern and the nonlinear time history analysis would be finalized.

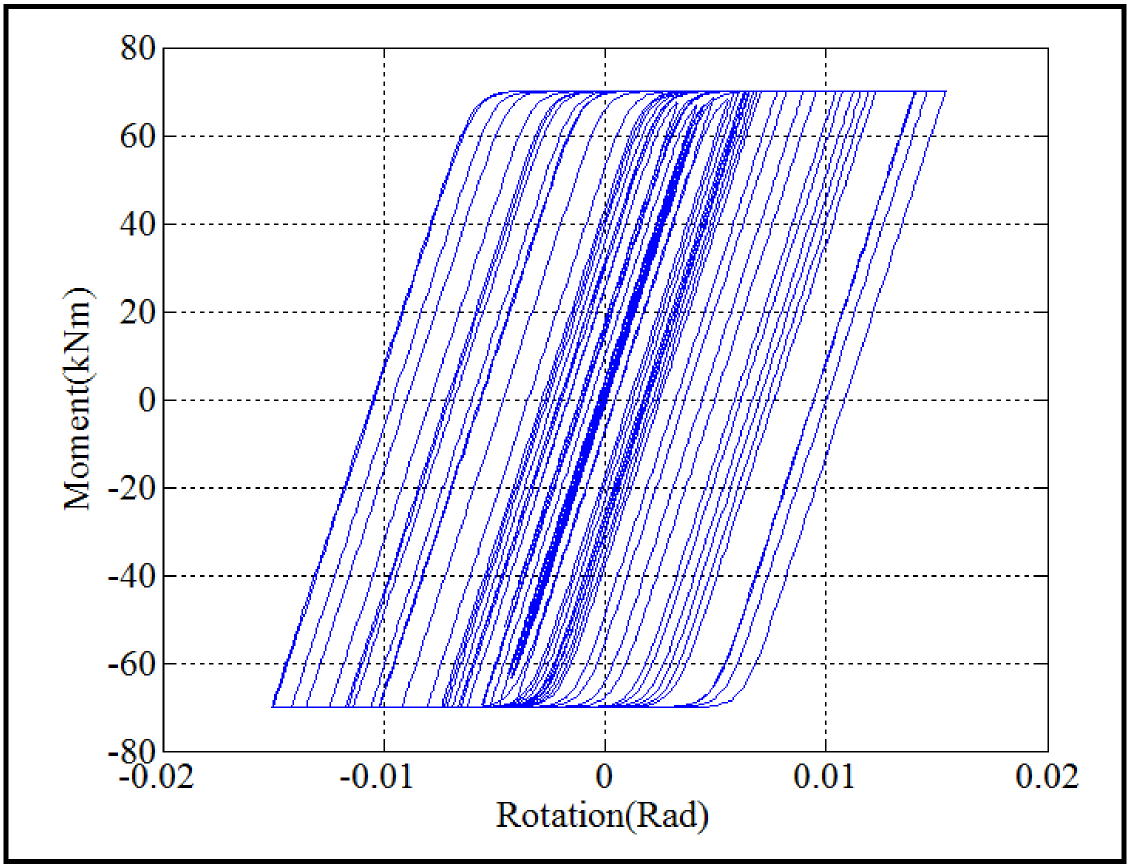

Figure 7: The Response of Bent-1 in Terms of Rotation

IDENTIFIED AND PREDICTED DAMAGE PROGRESS

This section presents an overview of damage progress throughout the excitation tests, based on two different approaches; namely, identified and predicted approaches. Identified approach is based on system identification and FEM updating, whereas predicted approach involves nonlinear time history analysis for damage detection at a certain state. In other words, both approaches intend to determine modification factors for structural parameters. Consequently, two sets of finite element models are constructed which could be defined as updated (identified) and non-updated (predicted) finite element models. Both models reflect the change in structural parameters with respect to damage state. Table-3 represents the accuracy of the modal frequencies obtained from updated FEM and nonlinear analysis of the non-updated FEM with the system identification results. Table-4 presents the comparison of the stiffness coefficient of the Bent-2 of updated and non-updated FEMs for both pre-earthquake and post-earthquake states.

Table 3: The Comparison of the Modal Frequencies of the Updated and Non-updated FEMs with Identified ones for the Post-Earthquake State

	System Identification	Non-Updated FEM	Error (%)	Updated FEM	Error (%)
Mode 1	2.18 Hz	2.31 Hz	5.96	2.19 Hz	0.46
Mode 2	2.81 Hz	2.62 Hz	7.25	2.60 Hz	7.47
Mode 3	10.19 Hz	8.36 Hz	17.96	8.59 Hz	15.70

Table 4: The comparison of the Stiffness Coefficient of the Bent-2 of Updated and Non-updated FEMs for both Pre-Earthquake and Post-Earthquake States.

	Pre-Earthquake State	Post-Earthquake State	Detected Damage
Vibration Based	0.9	0.7	22.2
Nonlinear Analysis Based	1.0	0.65	35
Error (%)	11.1	7.14	57.66

CONCLUSIONS

This paper presents the use of the vibration-based identified structural parameters in damage detection of bridges under incoherent motions. A large-scale shaking table test of a three-bent concrete bridge model was considered. Three bents of the bridge were shaken with different inputs simultaneously. Stiffness values of the structure was identified in the frequency domain via white-noise records and based on the identified stiffness values, damage on the bents of the bridge was obtained. Moreover, the earthquake excitation records were used in the nonlinear time history analysis of the non-updated FEM of the structure and the damage level of the columns were predicted according to moment-rotation relation. Finally, the detected damages of the bridge obtained from two different methods were compared in order to find the difference of the predicted model with the identified and updated one. Following conclusions were drawn:

- Stiffness value of the non-updated FEM is almost 10% higher than the stiffness value of the updated FEM.
- According to non-updated FEM, the prediction of the damage was 35% when the damage measured from identification results was 22%.
- Therefore, the deviation of the damage detection by two different methods was almost 58% which was a significant difference for the crucial structures such as bridges.

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