Instrumentation and Finite Element Model Update of a Highway Bridge

Huseyin Colak[1] and Serdar Soyoz[2]

[1] Bogazici University, Turkey
[2] Bogazici University, Turkey

ABSTRACT: The current practice of damage detection relies on visual inspection, which is time consuming, insufficient, subjective and requires physical presence of damage on the structure. On the other hand, sensor-based structural health monitoring can revolutionize the way of inspecting structures, particularly for damage assessment, in a rapid, remote, automated, and objective fashion. Moreover, continuous monitoring of structures has an important role in the performance evaluation of bridges under variable environmental conditions. The aim of this paper is to present the instrumentation, monitoring and finite element model (FEM) update of a bridge as the initial stage of a long-term monitoring project. The investigated structure is a four-span, reinforced concrete, highway bridge. Eight accelerometers are located on the superstructure and columns of the bridge in order to obtain its dynamic characteristics whereas three sensors are located on the ground level to trigger the data acquisition system under an earthquake event. Since the environmental conditions have crucial effects on the dynamic features of bridges, a temperature sensor is used in addition to eleven accelerometers. According to design drawings of the structure, a FEM was constructed and it was updated according to system identification results in order to have a representative baseline model of the bridge.

1 INTRODUCTİON

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 conditions as well as during extreme events such as earthquakes (http://www.fhwa.dot.gov/research/).

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.

The aim of this paper is to present the instrumentation, monitoring and finite element model update of a bridge as the initial stage of a long-term monitoring project. The investigated structure is a four-span, reinforced concrete, highway bridge. Eight accelerometers are located on the superstructure and columns of the bridge in order to obtain its dynamic characteristics whereas three sensors are located on the ground level to trigger the data acquisition system under an earthquake event. Since the environmental conditions have crucial effects on the dynamic features of bridges, a temperature sensor is used in addition to eleven accelerometers. According to design drawings of the structure, a FEM was constructed and it was updated according to system identification results in order to have a representative baseline model of the bridge.

2 INVESTIGATED BRIDGE AND MONITORING SYSTEM

The investigated structure is a four-span, reinforced concrete, highway bridge. Total length of the bridge is 84m and the height of the columns is 6.5m. Eight accelerometers are located on the superstructure and columns of the bridge in order to obtain its dynamic characteristics whereas three sensors are located on the ground level to trigger the data acquisition system under an earthquake event. The third and forth spans and third column could not be instrumented due to high vehicles passing under the bridge. Therefore, another accelerometer was used at these locations in order to obtain the transverse and vertical mode shapes of the structure. Since the environmental conditions have crucial effects on the dynamic features of bridges, a temperature sensor is used in addition to eleven accelerometers. 1 shows the monitored bridge whereas figure 2 presents the example of acceleration time-history records of the transverse direction.

Figure 1: The Investigated Bridge

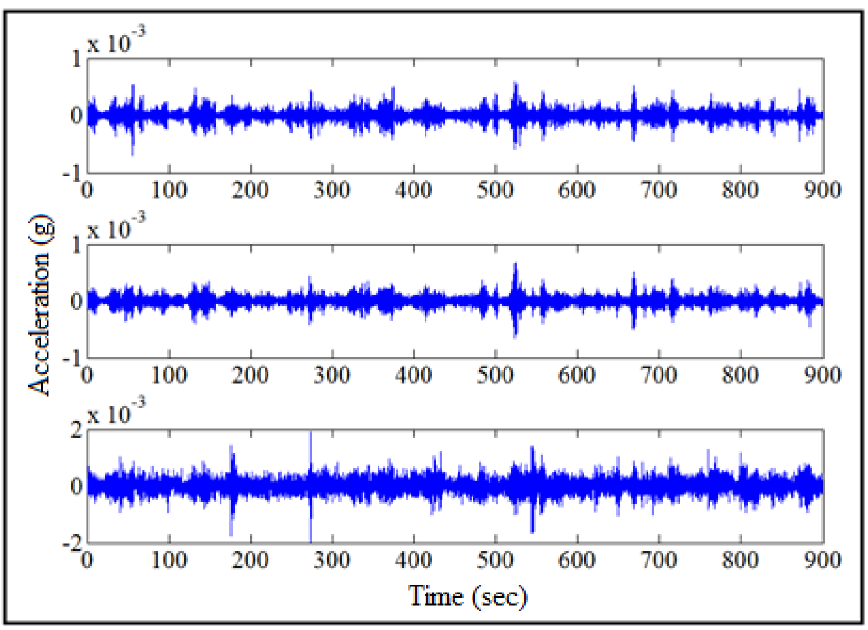

Figure 2: Acceleration Time-History of the Transverse Direction

3 FINITE ELEMENT MODEL OF THE BRIDGE

Nominal finite element model is developed in SAP2000 with frame elements based on design drawings. The abutment system of the bridge is a diaphragm abutment. The wall of the diaphragm was modelled with a frame element and the effects of the soil in transverse and longitudinal directions were modeled via link elements whose stiffness values were calculated according to Caltrans SDC 1.7 and Aviram et al. (2008). For the foundation systems of the columns, displacements in the transverse, longitudinal and vertical directions were assumed as fixed. In the rotational directions, the effect of the foundation systems was modelled with link elements and the stiffness values were calculated according to Priestley et al. (1996) and Aviram et al. (2008). Figure 3 presents the FEM of the bridge.

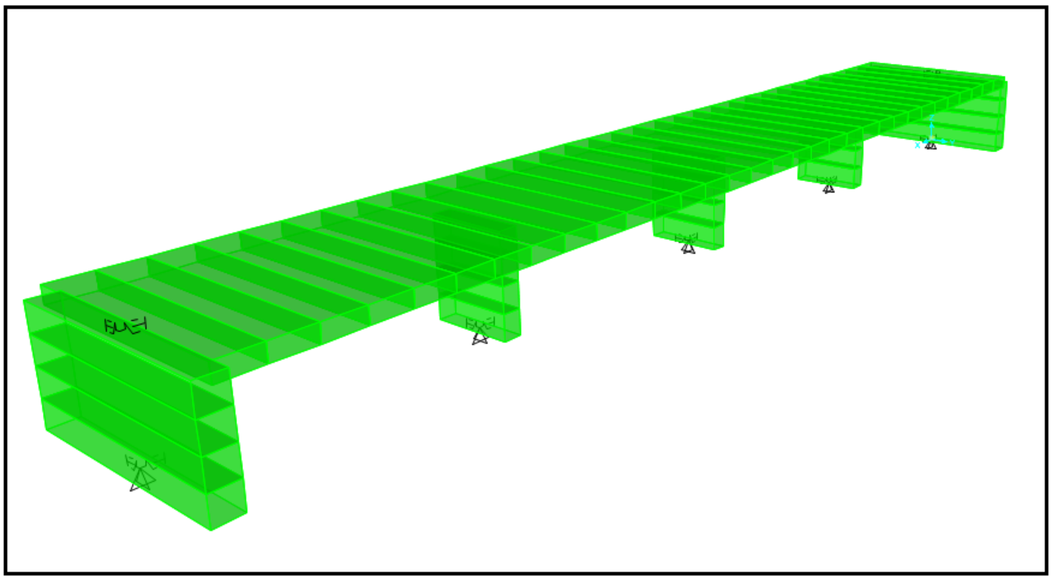

Figure 3: Finite Element Model of the Bridge

4 SYSTEM IDENTIFICATION

In this study, system identification is carried out in frequency domain using vibration measurements. 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.

3.1 Frequency Domain Decomposition

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 $S_{YY}(w)$ 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 in the transverse direction. Table 1 presents the comparison of frequencies obtained from identification and non-updated FEM.

Table 1: Comparison of Frequencies (non-updated FEM)

Mode	Measured	Simulated	Error
First	6.12 Hz	6.73 Hz	%9.06
Second	10.45 Hz	9.86 Hz	%5.65
Third	15.82 Hz	13.91 Hz	%12.07

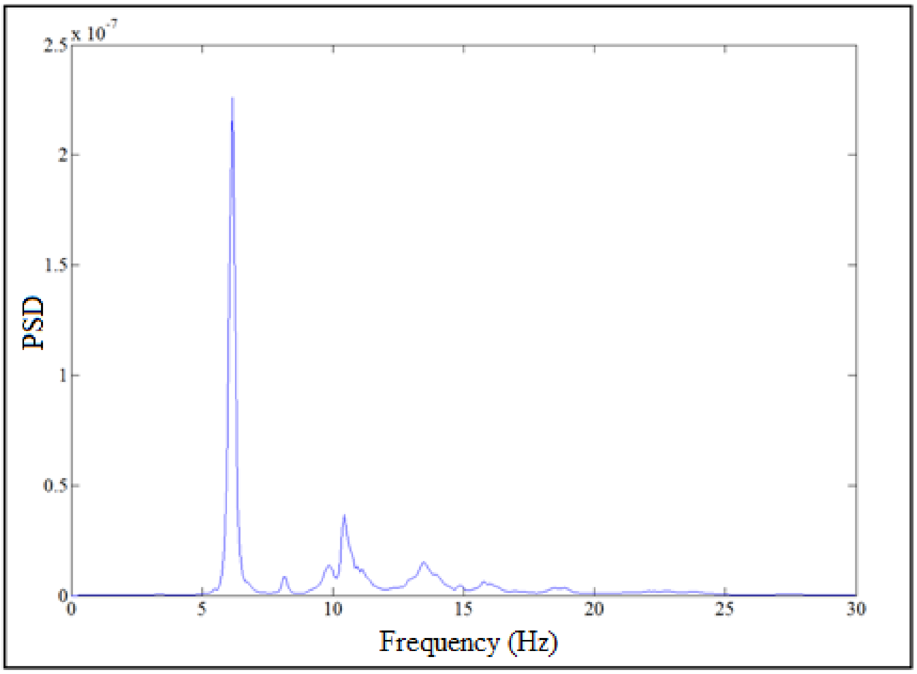

Figure 4: Power Spectra of the First Singular Value in the Transverse Direction

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

$$E(\alpha) = \sum_{i=1}^{3} \left(k_i . \left[\left(f_i^* - f_i \right) / \ f_i \right]^2 + h_i . \left[1 - MAC_i \right]^2 \right) \quad (2)$$

where ,

α : stiffness correction coefficient

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 Tables 2 the accuracies of parameter identification procedure are presented whereas figure 5 shows the first three mode shapes of the bridge obtained from identification and updated finite element model.

Table 2: Comparison of Frequencies (updated FEM)

Mode	Measured	Simulated	Error
First	6.12 Hz	6.12 Hz	-
Second	10.45 Hz	9.50 Hz	%9.09
Third	15.82 Hz	13.67 Hz	%13.59

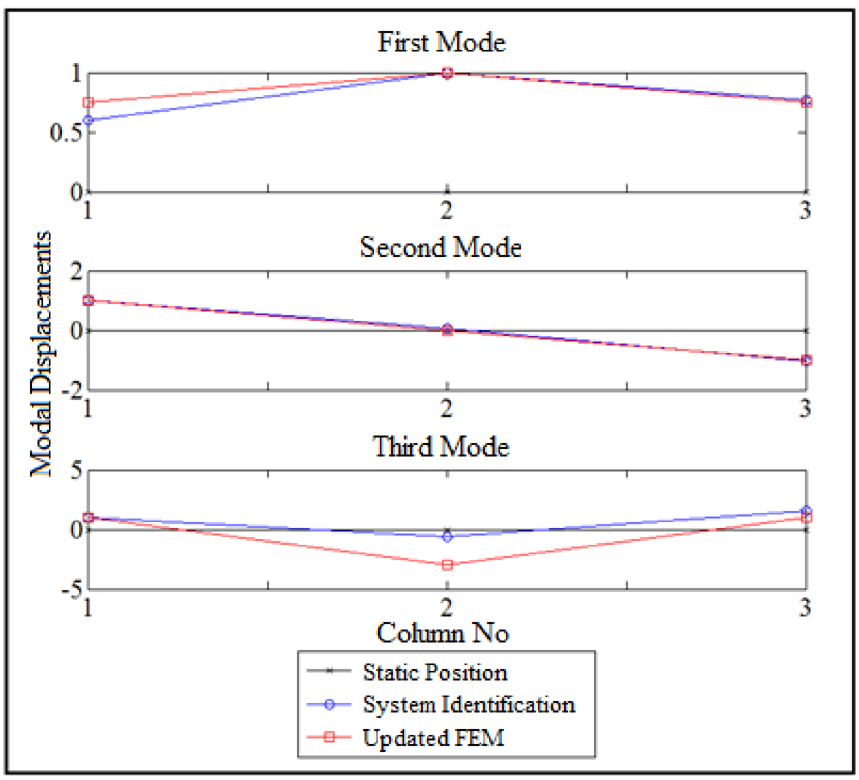

Figure 5: Comparison of Mode Shapes

5 CONCLUSION

Instrumentation, monitoring and finite element model update of a four-span, reinforced concrete, highway bridge is presented in this paper. Monitoring system consist eleven accelerometers. Eight accelerometers are located on the superstructure and columns of the bridge in order to obtain its dynamic characteristics whereas three sensors are located on the ground level to trigger the data acquisition system under an earthquake event. Dynamic characteristics of the structure were obtained from vibration measurements under ambient conditions. Finite element model of the bridge was developed in SAP2000 with frame elements based on design drawings. Since the differences between measured and simulated modal parameters are significant, FEM update of the structure was carried out to minimize these differences.

6 REFERENCES

Aviram A., Mackie K.R., Stodjadinovic B., (2008). Peer Reports 2008/03 Guidelines for Nonlinear Analysis of Bridge Structures In California

Brinker, R., Zhang, L. And Andersen, P., (2001). Modal Identification of Output-Only System Using Frequency Domain Decomposition, Smart Materials And Structures, 10(3), 441–55.

Caltrans (2013). Caltrans Seismic Design Criteria version 1.7. California Department of Transportation, Sacramento, California.

Priestley, M.J.N., Seible, F., and Calvi, G.M., (1996). Seismic Design and Retrofit of Bridges, Wiley, New York.

Otte, D., Ponseele, P. V. D. And Leuridan, J., (1990). Operational Shapes Estimation as A Function of Dynamic Loads, In Proceedings of 8th International Modal Analysis Conference, Society For Experimental Mechanics, Orlando, Fl, 413–21.