Proc. of the 6th Intl. Symposium on Reliability Engineering and Risk Management (6ISRERM)
31 May – 1 June 2018, Singapore
Editor(s) Xudong Qian, Sze Dai Pang, Ghim Ping Raymond Ong, Kok-Kwang Phoon

VIBRATION MONITORING AND DAMAGE EXPERIMENT ON AN ACTUAL STEEL PLATE GIRDER BRIDGE

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This paper is intended to discuss about vibration monitoring and a field experiment carried out on an actual steel plate girder bridge. The aim of the monitoring and field experiment is to identify the dynamic parameters of the bridge before and after applying artificial damage. Fatigue cracks propagated from the base plate of bearings to the web plate were considered as the artificial damage. The bridge was instrumented with accelerometers at a number of locations on the lower flange of the bridge to record vertical vibrations. The bridge was excited via moving vehicle tests and the resulting acceleration signals were used to identify dynamic parameters, such as the bridge mode shape, natural frequency and damping constant. For structural analysis of the bridge, a finite element model of the bridge was created whose structural properties and dimensions were decided based on the design documents and measurements during the field experiment. In numerical analysis, the artificial damage is modeled by assuming the pseudo crack. The frequency, which is one of the typical modal parameters, was adopted as the damage sensitive feature to monitor changes in the damage sensitive feature due to the artificial damage. Observations demonstrated that the frequencies identified from the field experiment were varied according to the bridge condition.

Keywords: fatigue crack, damage detection, finite element analysis, steel plate girder bridge, vibration monitoring

1 Introduction

Maintenance of civil infrastructures is an important technical issue. Structural health monitoring (SHM) based on vibration measurement has been developed as one of the efficient techniques to maintain civil infrastructures. Modal parameters of bridges such as natural frequency, damping ratio and mode shape can be identified from vibration responses of bridges. The SHM techniques focusing on damage detection and damage localization are still highly expected (Zhang 2007, Reynders et al. 2010, Kim et al. 2013, Chang et al. 2016). As a bridge is damaged or deteriorated, modal parameters of the bridge could be changed according to the damage level. One of the typical modal parameters of bridges is the frequency, which is also changed due to damage.

This study is intended to investigate the feasibility of the vibration-based SHM to detect damage on an actual steel plate girder. Especially, changes in frequencies due to the damage of the plate girder bridge are examined. Stochastic subspace identification (SSI) (Overschee et al. 1991) is used as a modal identification method. The eigenvalue analysis utilizing the bridge FE model is conducted as a preliminary structural analysis in which structural parameters were calibrated by the model updating on the vibration characteristics. By comparing the frequencies identified from the vibration responses between the reference bridge condition and damaged bridge condition, this study investigates the feasibility of damage detection by means of the vibration-based SHM. In addition, the modal parameters from the eigenvalue analysis with respect to each bridge condition are also compared to those of system identification utilizing experiment data so as to confirm the accuracy of the FE model.

2 Field Experiments

The field experiment was conducted on a real steel plate girder bridge shown in Figure 1. Artificial fatigue cracks (hereafter called artificial damage) at the girder end near the base plate of supports are considered in the experiment by severing the lower flange and the web plate by an Oxyacetylene cutting torch. The location of the artificial damage is at the lower flange and the web plate of the east main girder on the bearing at A1 abutment as shown in Figure 1. The artificial damage comprises two damage levels. One is the lower flange cut (hereafter called DMG1) and another is the web plate cut (hereafter called DMG2) as shown in Figure 1. To prevent any possible bridge collapse due to the artificial damage on the girder, a protection device was installed near the damage location. In addition, a stop hole was drilled to prevent the crack propagation. The bridge condition before the artificial damage is assumed as the intact condition (hereafter called INT). Ten accelerometers were installed on the lower flange of the bridge girder to monitor the vertical vibrations.

Modal parameters of the bridge were identified from accelerations of the bridge for each bridge condition. The vehicle used for the moving vehicle test was a passenger car and the speed of car is about 20km/h. Higher speeds were not considered caused by limited accelerating and decelerating distances. The vibration test is conducted 30 round trips for the intact condition, and 15 round trips per damage condition. Sampling rate and length of the vibration data set were 200Hz and 15s, respectively. Air temperatures were 29ºC during INT experiment, and 34 ºC during DMG1 and DMG2 experiments.

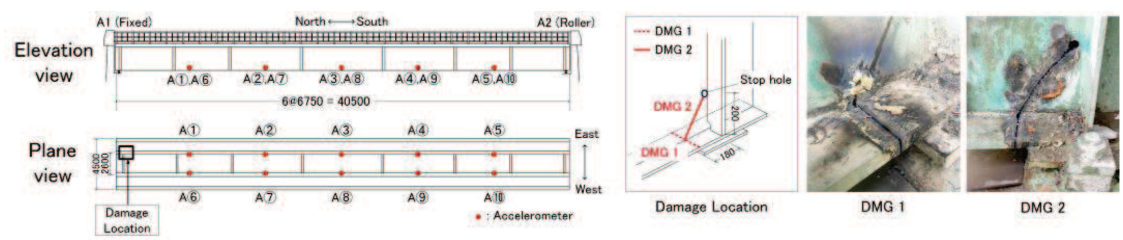


Figure 1. Observation bridge with sensor deployment and artificial damage.

3 Methodology

3.1 *Stochastic subspace identification*

The stochastic subspace identification (SSI) is adopted to identify the modal parameters such as frequency, damping constant and mode shape of the bridge. The dynamic system is modeled as the following state space model. (Overschee and Moor 1996, Heylen et al. 1997).

$$\mathbf{x}(\boldsymbol{k}+\mathbf{1}) = \mathbf{A}\mathbf{x}(\boldsymbol{k}) + \mathbf{w}(\boldsymbol{k});\ \mathbf{y}(\boldsymbol{k}) = \mathbf{C}\mathbf{x}(\boldsymbol{k}) + \mathbf{v}(\boldsymbol{k}) \tag{1}$$

where, $\mathbf{x}(k)$ and $\mathbf{y}(k)$ denote the state of structure and measurement at each time step k, respectively. $\mathbf{w}(k)$ and $\mathbf{v}(k)$ denote the process noise and measurement noise, respectively. They are assumed to be stationary white noise. System matrices **A** and **C** which contain the modal information are estimated by means of the least square method for minimal prediction error of the state $\mathbf{x}(k)$ given by the forward Kalman filter. The poles of the dynamic system provide modal properties of the dynamic system.

The algorithm for the SSI is described briefly. Firstly, we obtain the projection matrix $\boldsymbol{O}_i$ that is estimated as follows.

$$\boldsymbol{O}_i = \mathbf{Y}_\mathrm{f}\mathbf{Y}_\mathrm{p}^\mathrm{T}\left(\mathbf{Y}_\mathrm{p}\mathbf{Y}_\mathrm{p}^\mathrm{T}\right)^{\dagger}\mathbf{Y}_\mathrm{p} \tag{2}$$

where $(\ \cdot\ )^{\dagger}$ denotes the Moore-Penrose pseudo inverse matrix. $\mathbf{Y}_f$ and $\mathbf{Y}_p$ are the block Hankel matrices of the future and past outputs respectively, and defined as follows.

$$\left(\begin{array}{c}\mathbf{Y}_\mathrm{p}\\ \hline \mathbf{Y}_\mathrm{f}\end{array}\right) = \left[\begin{array}{ccc} \mathbf{y}(0) & \dots & \mathbf{y}(j-1) \\ \vdots & \ddots & \vdots \\ \mathbf{y}(i-1) & \dots & \mathbf{y}(i+j-2) \\ \hline \mathbf{y}(i) & \dots & \mathbf{y}(i+j-1) \\ \vdots & \ddots & \vdots \\ \mathbf{y}(2i-1) & \dots & \mathbf{y}(2i+j-2) \end{array}\right] \tag{3}$$

The singular value decomposition (SVD) is then applied to factorize $\boldsymbol{O}_i$ as

$$\boldsymbol{O}_i = \mathbf{USV^T} = (\mathbf{U_1U_2})\begin{pmatrix}\mathbf{S_1} & \mathbf{0}\\ \mathbf{0} & \mathbf{S_2}\end{pmatrix}(\mathbf{V_1V_2})^\mathbf{T} \simeq \mathbf{U_1S_1V_1^T} \tag{4}$$

where $\mathbf{U}$ and $\mathbf{V}$ are unitary matrices with an appropriate size, and **S** is a diagonal matrix with non-negative elements. The diagonal elements of **S** are known as singular values of $\boldsymbol{O}_i$. Singular values in **S** are listed in descending order. Therefore, the components in $\mathbf{U_1S_1V_1^T}$ contain most of the information defining the elements in $\boldsymbol{O}_i$ and components in $\mathbf{U_2S_2V_2^T}$ are regarded as trivial components. Theoretically, the optimal state sequence $\mathbf{X}_i = [\mathbf{x}(i) \quad \mathbf{x}(i+1) \quad \dots \quad \mathbf{x}(i+j-1)]$ predicted by the Kalman filter in least square sense is obtained as follows.

$$\mathbf{X}_\mathrm{i} = \mathbf{S_1^{1/2}V_1^T} \tag{5}$$

The significant components of orthogonal vectors in the state sequence can be extracted by applying SVD to $\boldsymbol{O}_i$, and the system matrices are obtainable from $\mathbf{X}_i$. The number of poles corresponds to the number of singular values determined in Eq. (5). In other words, we can extract the significant modal components of the bridge from the measured acceleration data by the SVD.

3.2 *FE model calibration*

To investigate the structural characteristics of the bridge, the eigenvalue analysis using the FE model of the bridge was conducted by utilizing ABAQUS 6.14. The bridge model, shown in Figure 2, was built based on the documents of periodic inspection and information obtained from the field experiment as the original design documents of this bridge are not available. The bridge consists of concrete deck, main girders, vertical stiffeners, horizontal stiffeners, cross frames and lateral bracings. The concrete deck, main girder, vertical stiffener and horizontal stiffeners were modeled with shell element, and cross frame and lateral bracing were modeled with beam element. For the boundary condition, fixed support was assumed for the support at the A1 abutment and roller support with longitudinal spring was assumed for the support at the A2 abutment. Specific gravity of steel is 7.85 and concrete is 2.40. Young's modulus of steel was assumed as 210 GPa.

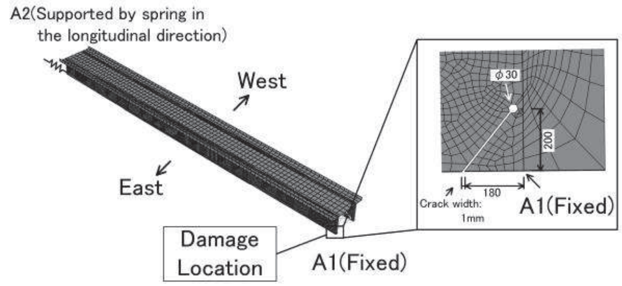


Figure 2. FE model of the bridge and pseudo cracks on plate girder.

For the Young's modulus of reinforced concrete it was assumed as 22.02 GPa that was obtained from compression test of the concrete core.

Since the bridge model has some differences from the actual bridge due to deterioration and design errors, the bridge model needs model updating for calibrating parameters of the bridge model. The cross entropy (CE) method was applied to update the bridge model so that the natural frequencies of the bridge model become comparable to those of the actual bridge. The CE method involves an interactive procedure where each interaction is divided into two steps (McGetrick et al. 2015). First step is to generate a random data sample according to a certain probability distribution. Second step is to update the parameters of the probability distribution based on the data generated in the first step in order to produce better samples in the next interaction.

Since a protection device was installed on the bridge as previously stated, the structural characteristics of the bridge was a little different from those before the damage experiment. However, any possible influence caused by the protection devices was not considered in the model update. To update the bridge model, an elastic spring in the longitudinal direction was introduced into the roller support, and the spring stiffness was calibrated by the model update. In addition, to investigate the change in structural characteristics, the artificial damage was modeled to pseudo cracks in the updated FE model. This crack was created by removing the connection of elements (see Figure 2). The eigenvalue analysis was conducted for each damage scenario such as INT, DMG1 and DMG2.

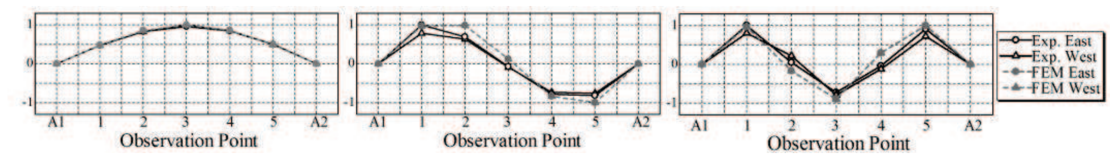
4 Results

4.1 *Identified vibration characteristics*

Identified frequencies and mode shapes from the field experiment data are shown in Figure 3. In fact, two modes for 9.55 Hz and 13.7 Hz showed similar modes relevant to the 2nd bending mode, but this study examined the mode around 9.55Hz as the 2nd bending mode from the eigenvalue analysis was around 9.33 Hz. Mean values of the identified frequencies for the 1st, 2nd and 3rd bending mode before damage experiment were 3.16Hz, 9.55Hz and 21.88Hz, respectively. Mode shapes from the eigenvalue analysis are shown in Figure 3 with those from experimental data. Mode shapes of eigenvalue and experiment result were comparable with each other.

4.2 Comparison of identified frequencies among each condition

Histograms of the identified frequencies from the monitoring data in each bridge condition are shown in Figure 4 with normal distribution fits, where the bin width was chosen as 0.01Hz. The



1st bending mode 2nd bending mode 3rd bending mode

Figure 3. Mode shapes both identified from the field experiment data and eigenvalue analysis.

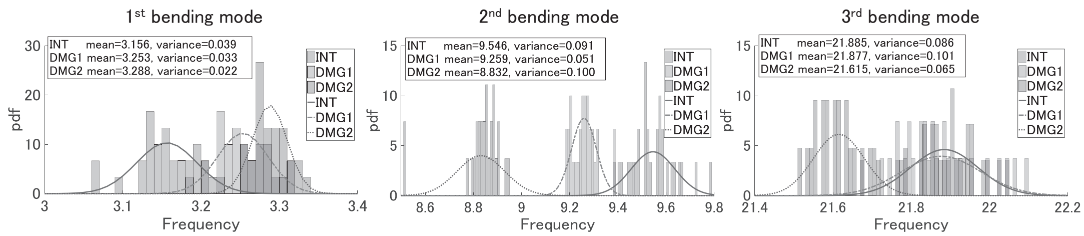


Figure 4. Histograms of the frequencies identified from the experiment data in each bridge condition: horizontal and vertical axes denote frequency and probability density function respectively.

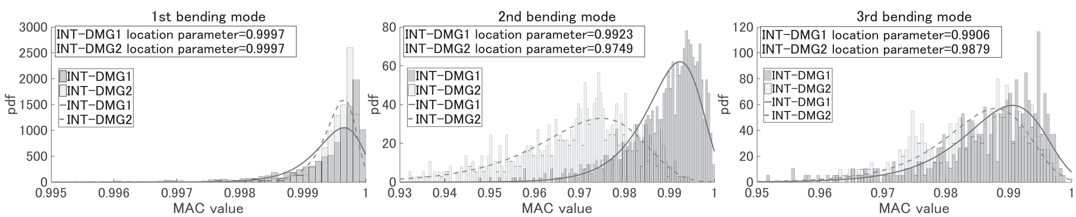


Figure 5. Histograms of MAC value of identified modes between INT, DMG1 and DMG2: horizontal and vertical axes denote MAC value and probability density function respectively.

Table 1. Frequencies and MAC values from eigenvalue analysis.

Bridge condition	INT	DMG1	DMG2
	Frequency (Hz) / MAC		
1st bending mode	3.09 / N.A.	3.09 / 1.000	3.07 / 0.997
2nd bending mode	9.33 / N.A.	9.28 / 0.997	9.13 / 0.943
3rd bending mode	21.6 / N.A.	21.4 / 0.999	20.8 / 0.932

histogram was created utilizing 30 samples of the identified frequencies. The statistical distribution showed that frequency distributions for the 2nd and 3rd bending modes moved to lower frequency region as damage becomes severe, i.e. those distributions for the DMG1 and DMG2 scenarios moved to the lower frequency region from the distribution of the INT scenario. It demonstrates decrease of bending stiffness of the plate girder due to the artificial damage.

However, for the frequency distribution of the 1st bending mode moved to higher frequency region in the order of INT, DMG1 and DMG2, and one reason might be change in the boundary condition at A2 abutment as the monitored longitudinal displacements at the support decreased due to the damage. However it needs more comprehensive investigations. In addition, histograms of MAC (Modal Assurance Criterion) values calculated by identified modes are shown in Figure 5 with the Type I extreme distribution fits. Comparing the distribution of MAC values for the 1st bending mode of DMG1 and DMG2, little change between DMG1 and DMG2 was observed. For the distribution of 2nd bending mode, on the other hand, clear change was observed. For the distribution of 3rd bending mode, small change was observed.

4.3 *Comparison of eigenvalue analysis and field experiment*

In the model update, using the experiment data as reference, the initial finite element model is correlated and fine-tuned towards frequencies and MAC values. The updated FE model of the bridge led to eigen-frequencies of 3.09Hz, 9.33Hz and 21.6Hz for the 1st, 2nd and 3rd bending modes, respectively. Frequencies of eigenvalue analysis considering pseudo cracks as damage of the plate girder are summarized in Table 1. The frequencies for the 1st, 2nd and 3rd bending modes decrease in the order of damage severity. MAC values for these modes also decrease in the order of damage severity. Changes in frequency due to the damage obtained from the eigenvalue analysis showed similar tendency to those from experiment data except the 1st bending mode.

5 Summary

This study investigated the feasibility of damage detection of an actual steel plate girder bridge utilizing identified modal parameters from vehicle induced vibrations of the bridge. The frequency and MAC value were considered as damage sensitive features of the bridge. Frequencies identified from the experimental data were change due to the artificial damage. Frequencies of the 2nd and 3rd bending modes decreased depending on the damage severity. However, the frequency of the 1st bending mode resulted in an increase due to damage. Distributions of MAC values for the 2nd and 3rd bending modes also led to changes due to the damage, and the clearest change in MAC value was observed in the distribution of the 2nd bending mode. The eigen-frequencies were also decreased due to the artificial damage. Tendency of changes in frequency due to the artificial damage obtained from the eigenvalue analysis was similar to that from experiment data except the 1st bending mode.

The next step for this study will focus on explaining reasons for increase of the first bending frequency from experimental data due to damage and verifying validity of the updated FE model for static analysis by comparing static loading experiment data. Moreover, reasons for the observations of two similar mode relevant to the 2nd bending mode around 9.55 Hz and 13.7 Hz were not clear yet, and more comprehensive investigations are needed.

Acknowledgments

This study is partly sponsored by a Japanese Society for the Promotion of Science (JSPS) Grant-in-Aid for Scientific Research (B) under project No.16H04398, which is greatly appreciated.

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