EXPERIMENTAL-ANALYTICAL FRAMEWORK FOR DAMPING CHANGE-BASED STRUCTURAL HEALTH MONITORING OF BRIDGES

Dammika ABEYKOON [1)], Kosuke KAWARAI [1)], Hiroki YAMAGUCHI [1)], Yasunao MATSUMOTO [1)] and Tsutomu YOSHIOKA [2)]
[1)] Structural Mechanics and Dynamics Lab., Department of Civil and Environmental Engineering, Saitama University
[2)] Nippon Engineering Consultants Co., Ltd, Tokyo

ABSTRACT

Structural health monitoring (SHM) of bridges by visual inspection is not necessarily reliable as many damages are not discovered during the periodic inspection of bridges. A technique to assist the visual inspection is vibration-based SHM. It is believed that structural damages lead to changes in stiffness and damping properties, and change the dynamic characteristics of structures, such as natural frequency, mode shape, and modal damping ratio. Although changes in the modal damping ratio can be used as damage indicators in the field of vibration-based SHM, the method's accuracy remains concerning. This study investigated the analytical modal damping evaluation as a complementary method to the experimental SHM of bridges. An energy-based damping model was introduced to estimate the damping parameters of a steel arch bridge, such as the equivalent loss factors of structural components, and the modal damping ratios of the bridge were then analytically evaluated using the damping parameters. The results confirmed that the proposed methodology can identify the damping sources in bridges and their contributions to each modal damping ratio, and complements the experimental SHM of bridges. As an application of the method discussed, damage component detection method for steel bridges was discussed based on damping change in their local member coupled modes. Furthermore, possible improvement in analytical damping evaluation and experimental damping identification were also discussed.

KEYWORDS: Damping analysis; Modal damping ratio; Damage detection; Structural health monitoring

1. INTRODUCTION

The need for better structural health monitoring (SHM) of bridges is emphasized by many collapses or failures of such infrastructures, as their damages not being recognized during periodic visual inspections (Astaneh-Asl 2008; Fisher, et al. 2001; Bagnariol 2003). On the other hand, SHM techniques are very important, especially in developed countries, as many bridges have reached or are reaching the end of their theoretical design life. A technique that has been investigated worldwide to assist the visual inspection is vibration-based SHM (Doebling et al. 1996; Carden and Fanning 2004). Note that structural damages lead to changes in the mass, stiffness and damping properties of structures, hence producing changes in dynamic characteristics such as natural frequency, mode shape, and modal damping ratio, which can be identified through field vibration measurements. Many modal identification techniques have been introduced during last few decades (Pappa and Ibrahim 1981; Juang and Pappa 1985; Yang et al. 2003) and are presently used in vibration-based SHM. Most of these SHM techniques focus on either natural frequency or mode shape. However, natural frequency has low sensitivity to damages, unless a significant change in stiffness has occurred due to damages. The sensitivity of mode shape to damages is directly related to damage location, and therefore, mode-shape-based techniques need many sensors. On the other hand, many researches in the literature concern the damage-sensitive parameter of modal damping ratio (Curadelli et al. 2008; Frizzarin et al. 2010; Yoshioka et al. 2010). Nonetheless, there are concerns due to fluctuations in experimentally identified modal damping ratios in vibration-based SHM.

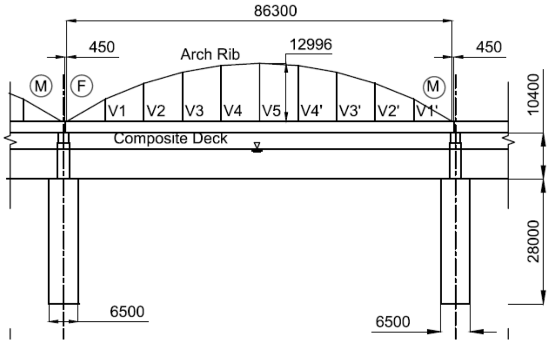
Analytical modal damping estimation is one of the possible approaches that can be used to justify the experimentally identified modal damping ratios of the structures. There are few studies on analytical modal damping estimation of structures in the literature. Energy-based damping estimation is the one of such analytical method that can be used (Yamaguchi et al. 1997; Yamaguchi and Matsumoto 2002). However, there is no extensive research on the application of analytical modal damping evaluation for the SHM of bridges.

This study focuses on the energy-based analytical method for evaluating modal damping ratios of bridges to complement experimental SHM. The fundamental concept is to decompose modal damping ratios into mode-independent damping parameters of energy-dissipating sources in bridges, such as the loss factor of each structural component and the friction coefficient of moveable supports. This provides the theoretical basis for modal damping evaluation and improves reliability by identifying the contributions of the energy-dissipating sources to each modal damping ratio and by using the change in damping parameters to detect the damaged structural components. A steel arch bridge is chosen as the case study. After conducting eigenvalue analysis and field vibration measurements, the analytically obtained mode shapes and experimentally identified modal damping ratios are used to estimate the equivalent loss factors of the bridge's structural components. The modal damping ratios of the bridge are then reevaluated considering the contributions of the damping sources by using those damping parameters. The procedure followed in the study can be introduced as a new framework for damping change-based SHM of bridges. Then, damaged member detection method for steel arch and truss bridge considering damping changes in their local member coupled modes was discussed as an application of the proposed framework. Finally, possible improvement in analytical damping evaluation and experimental damping identification was also discussed which could lead to improve the reliability of the proposed framework.

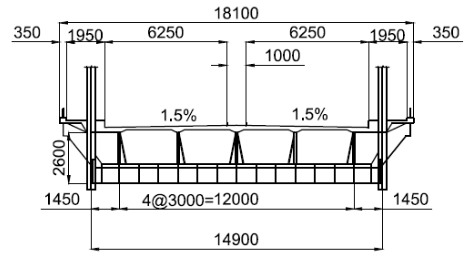
2. DETAILS AND EIGENVALUE ANALYSIS OF THE BRIDGE

2.1 Details of studied bridge

The studied steel bridge is a Langer type arch bridge of having the span length of 86.3 m and the width of 18.1 m, constructed in 1971, as shown in Fig. 1. The bridge has three spans over the river, which is a part of total bridge spread with length of 810m, supported by pin and roller bearings on either side of the span. The arch ribs have box-shaped cross-section whereas the vertical members have H sections. The bridge deck is a composite deck of steel plate girders and reinforced concrete slab. The bridge is in relatively healthy state at the time of field vibration measurement.



(a)



(b)

Fig. 1. Studied Langer-type arch bridge constructed in 1971: (a) elevation of bridge; (b) section of the deck. (Unit in millimeters)

2.2 Eigenvalue analysis of bridge

A finite element (FE) model was created for eigenvalue analysis of the studied bridge followed by nonlinear static analysis. All structural members were modeled by general sectioned beam element with assigned mass and geometric properties. The equivalent section concept was utilized in modeling the concrete deck slab with added mass and stiffness properties to the stringer beams and added stiffness to the cross beams. The pin and roller support conditions were used to support the bridge on either side of the span, and spring elements were used to model the dynamic friction at the moveable supports of bridge. The eigenvalue analysis was first conducted with fully released condition at the bridge's moveable support. Then, the eigenvalue analysis was repeated by releasing only the rotational degrees of freedom at the moveable supports in order to predict the dynamic characteristics of bridge under no support movement condition, as such situation was observed during field vibration measurements.

The calculated natural frequencies at the condition of with and without moveable supports are shown in Table 1. The change in support conditions significantly affects only on the natural frequency of the first symmetric vertical mode, which is associated with relatively high support movements. The mode shapes obtained with moveable support conditions are shown in Fig. 2 and categorized into three groups; global modes in which vibrations of girders and arch ribs are dominant (first symmetric vertical and torsional

Table 1. Results of eigenvalue analysis of the arch bridge and its comparison with experimental identifications

Mode Shape		With support movement			Without support movement			Modal Assurance Criterion**
		Exp. frequency (Hz) *	FEM frequency (Hz)	% Error wrt exp. value	Exp. frequency (Hz) *	FEM frequency (Hz)	% Error wrt exp. value	
Global Modes	First symmetric vertical	1.84	1.78	-3.3	1.85	1.92	3.8	0.93
	First symmetric torsional	2.91	2.74	-5.8	2.93	2.79	-4.8	0.96
Local Modes	V5u local	8.29	8.15	-1.7	8.31	8.14	-2.0	0.99
	V5d local	8.47	8.51	0.5	8.47	8.50	0.4	1.00
Coupled	V4 coupled	-	9.09	-		9.07	-	

* Average value of experimentally identified frequencies shown in Table 3.3

** Calculated by the analytical eigen vector of the FE model with support movement and the experimental mode vector identified from FV1

modes in Fig. 2 (a) and (b)); vertical member local modes, in which vibrations of particular vertical members dominate without any significant vibration in bridge girders and arch ribs (V5u local and V5d local modes in Fig. 2 (c) and (d)); and the coupled modes of vertical members, in which vibrations of several vertical members are coupled with those of girders and arch ribs (V4 vertical members coupled mode in Fig. 2 (e)).

The accuracy of the FE analysis was checked by comparing with the natural frequencies and mode shapes identified by the field experiment. The natural frequencies are compared in Table 1 by considering the relative errors of calculated frequencies with respect to the experimental values. As for the mode shapes, the modal assurance criterion (Allemang 2003) for indicating the coherence between two modal vectors was calculated between the numerically calculated and experimentally identified modal vectors. The maximum relative error and the lowest modal assurance criterion value were 5.8% and 0.93, respectively.

3. FIELD VIBRATION MEASUREMENT AND MODAL DAMPING IDENTIFICATION

3.1 Bridge vibration measurement

The vibration measurements of the bridge were performed during regular service for experimental modal damping identifications, which were then used in the analytical damping evaluation. Ten piezoelectric accelerometers and three servo velocimeters were placed on the bridge as shown in Fig.3 to measure the acceleration responses. The servo velocimeter was used for easy measurement of the bridge deck; however, their number was limited to three because of availability. The sensors on the bridge deck were placed directly above the main bridge girders in both upstream and downstream sides at the locations of L2u etc. as shown in Fig.3, in order to measure the vertical bending vibrations

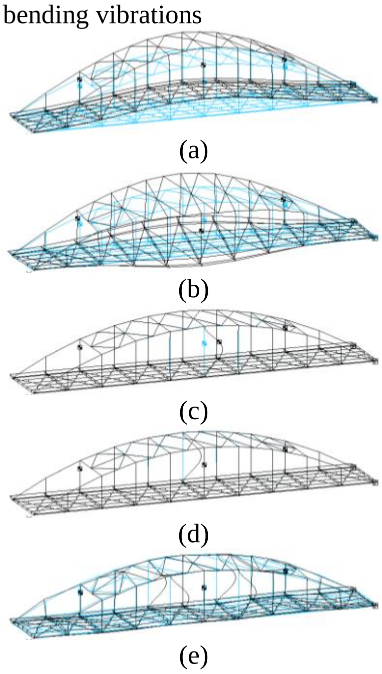


Fig.2. Numerically analyzed vibration modes of the arch bridge with moveable support conditions: (a) first symmetric vertical; (b) first symmetric torsional; (c) V5u vertical member local; (d) V5d vertical member local; (e) V4 vertical member coupled.

and torsional vibrations of bridge. The accelerometers were also attached to the vertical members of V5u etc. at the quarter points from the bottom to the top to measure their lateral bending vibrations. Furthermore, two displacement transducers were placed at the Du and Dd spots to measure the displacements at moveable supports. The ambient vibrations of the bridge under normal service conditions were recorded at a sampling frequency of 100Hz, whereas the service conditions of the traffic were relatively high. Five sets of measurements, each lasting for 10 min, were recorded.

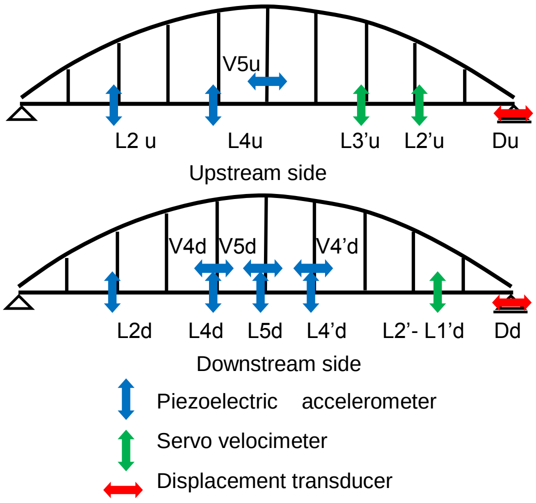


Fig.3. Position of accelerometers, velocimeters and displacement transducers placed during field vibration measurements of the arch bridge.

Ten sets of free vibration (FV) records were extracted from above mentioned five sets of ambient vibration records by checking the traffic free time on the bridge from the video of traffic recorded during the measurement. Figure 4 shows the FV time histories of the bridge deck measured at the mid-span, L5d, corresponding to the above mentioned 10 sets of FV records. The FV1 to FV5 records are classified as 'large vibration' with the maximum vibration acceleration is greater than 0.1 m/s^2, which induces some amount of movement at the bridge's moveable supports, whereas FV6 to FV10 are classified as 'small vibration' with smaller acceleration without any measurable support movement. The set of FV records were analyzed with the Eigensystem realization algorithm (ERA) (Juang & Pappa 1985) to identify the natural frequencies, mode shapes, and modal damping ratios of the bridge.

3.2 Modal damping identification

The modal identification results, especially the damping ratios, typically fluctuate for different system orders. Thus, it is important to carefully extract the modal damping ratios in order to increase the diagnostic accuracy. The extended modal amplitude coherence (EMAC) (Pappa & Elliott 1993) was, used in this study for screening the results identified by ERA. EMAC represents the correlation of the modal time series inside and outside the period of the analyzed time series, and is calculated using the differences in amplitude and phase. The screening of modal identifications by ERA was conducted with two steps. The EMAC and modal damping ratios were used in the first-step screening. Subsequently, in the second-step screening, the identical modes were evaluated to assure their stability for more than 10 sequential system orders. Although several modes were identified from each set of FV records, only the modes identified with an EMAC value greater than 0.6 were considered by quantitatively discriminating the system and noise modes by following the first-step screening. Furthermore, the modal assurance criterion values, greater than 0.9, calculated by the mode vector in system order n and n+1, were considered to improve the reliability of the experimental out-comes by following the second-step screening process.

3.3 Results of identification

The identified types of mode shapes and natural frequencies in each FV record are summarized in Table 2. The number of identified modes is varied with each FV record and the larger vibrations of FV1, FV2 and FV3 gave the largest number of identified modes. The modal identification for FV1 is shown in Fig. 5, which represents the relation between identified natural frequency and the modal damping ratio. Note that the identified modal damping ratios fluctuate for different system orders even with the stability criteria used in this study, while each natural frequency is stably identified.

Figure 6 shows the modal identification results for all FV records. In this figure, the natural frequency and associated modal damping ratio are uniquely determined for each mode of each FV record by taking the averages of identified values with some fluctuations. As shown in Fig. 6, the identified modal damping ratios fluctuate owing to differences in the FV records.

Figure 7 depicts the identified mode shapes in FV1. The unit normalized modal amplitudes at the sensors in the bridge deck and the vertical members are separately plotted with respect to the lower chord truss nodes from L1 to L1' and the vertical members from V1 to V1'. The figure also shows the analytically obtained mode shapes corresponding to the sensor positions. The modal assurance criterion in Table 1 between the experimentally identified and analytically calculated modal vectors was calculated by considering both modal vector components corresponding to the sensor locations. The closeness of two mode shapes in Fig. 7 as well as relatively large value of modal assurance criterion in Table 1, indicates a good match between the experimental and analytical results.

4. ENERGY-BASED MODAL DAMPING ANALYSIS OF STEEL ARCH BRIDGE

The sources of damping in the steel arch bridge are assumed to be the material viscous damping of bridge's arch ribs, vertical members and girders, and the frictional damping at the moveable supports. That is, the energy dissipation at the members' connection was assumed to be included in the energy dissipation of each

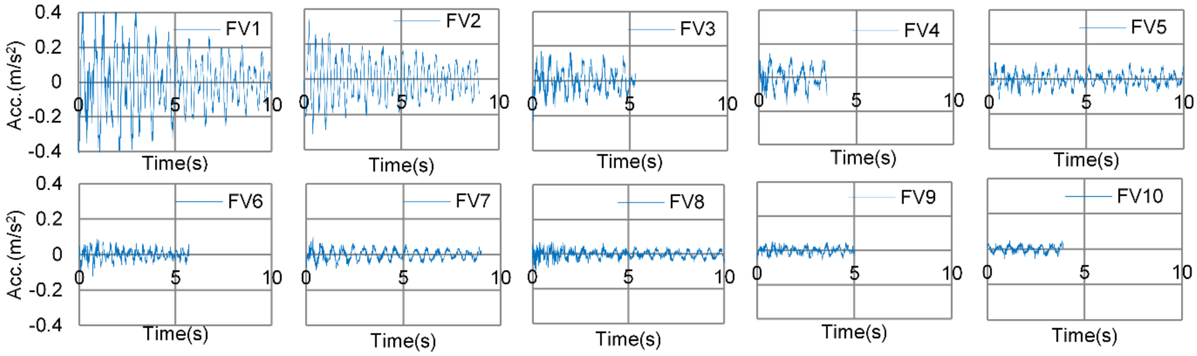


Fig.4. Free vibration time histories of bridge deck measured at L5d for all sets of FV records.

Table 2. Experimentally identified natural frequencies of the arch bridge

Mode shape		Experimentally identified natural frequency (Hz)											
		Large vibrations with support						Small vibrations without support					
		FV1	FV 2	FV 3	FV 4	FV 5	Avg	FV6	FV 7	FV 8	FV 9	FV10	Avg.
Global modes	First symmetric vertical	1.83	1.84	1.84	1.84		1.84		1.85		1.85	1.83	1.85
	First symmetric torsional	2.90	2.91	2.91		2.93	2.91	2.94	2.93		2.92		2.93
Local modes	V5u local	8.29	8.26	8.31	8.29	8.30	8.29	8.30		8.31		8.31	8.31
	V5d local	8.47	8.47	8.47	8.47	8.47	8.47	8.47	8.47	8.47	8.47	8.48	8.47

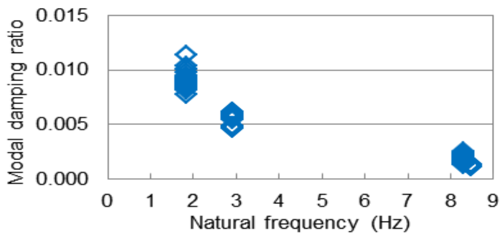


Fig.5. Relation of screened natural frequencies and modal damping ratios from FV1.

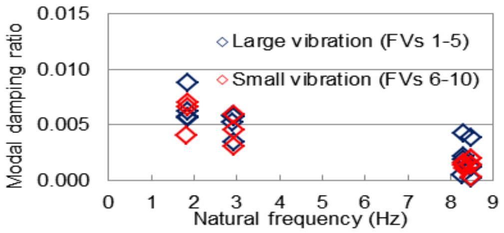


Fig.6. Modal identification results for all FVs.

member and the energy dissipation in the girder implicitly accounts for that of deck slab. Using the energy-based damping definition, the n-th modal damping ratio, ξ_n , can then be expressed in the following form (Yamaguchi et al. 1997).

$$\xi_n = \frac{2\pi\eta_a V_{a,n}}{4\pi U_n} + \frac{2\pi\eta_g V_{g,n}}{4\pi U_n} + \frac{2\pi\eta_v V_{v,n}}{4\pi U_n} + \frac{8A_{s,n}\mu_s R}{4\pi U_n} \quad (1)$$

where η_a , η_g , η_v and $V_{a,n}$, $V_{g,n}$, $V_{v,n}$ are the equivalent loss factors and the n-th modal strain energies of the arch ribs, girders and vertical members, respectively. $A_{s,n}$ is the n-th modal amplitude at the moveable supports and U_n is the total potential energy in the n-th mode.

The equivalent loss factors are assumed independent of vibration mode, and thus Eq. (1) is expanded to a set of equations for 'n' numbers of modes.

$$\begin{Bmatrix} \xi_1 \\ \vdots \\ \xi_n \end{Bmatrix} = \begin{bmatrix} \frac{2\pi V_{a,1}}{4\pi U_1} & \frac{2\pi V_{g,1}}{4\pi U_1} & \frac{2\pi V_{v,1}}{4\pi U_1} & \frac{8A_{s,1}R}{4\pi U_1} \\ \vdots & \vdots & \vdots & \vdots \\ \frac{2\pi V_{a,n}}{4\pi U_n} & \frac{2\pi V_{g,n}}{4\pi U_n} & \frac{2\pi V_{v,n}}{4\pi U_n} & \frac{8A_{s,n}R}{4\pi U_n} \end{bmatrix} \begin{Bmatrix} \eta_a \\ \eta_g \\ \eta_v \\ \mu_s \end{Bmatrix} \quad (2)$$

Eq. (2) is then solved by applying the least square method using experimentally identified modal damping ratios and the energy quantities calculated from the relevant experimental and numerical results. Once these damping parameters are estimated, the modal damping ratio of respective modes can be analytically evaluated by back substituting the estimated parameters to Eq. (2).

5. RESULTS OF DAMPING ANALYSIS AND DISCUSSION

5.1 Identified Equivalent Loss Factors

From the FVs with large vibration with support movement (Case 1) and small vibration without support movement (Case 2), damping parameters were estimated for two categories of modal identification. Each analysis case follows two steps in the parameter estimation process. The equivalent loss factor of vertical member was first estimated in Step 1 by only considering the local

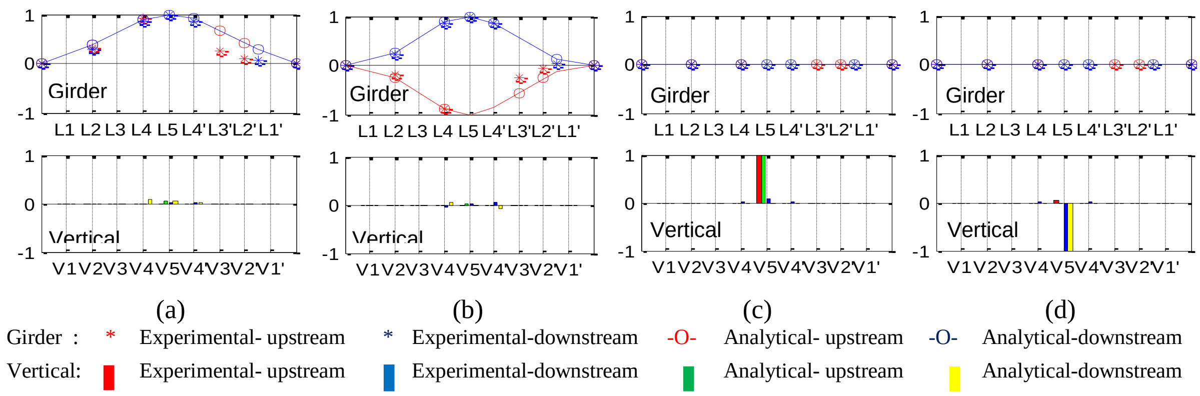


Fig.7. Experimentally identified vibration mode shapes from FV1: (a) first symmetric vertical; (b) first symmetric torsional; (c) V5u vertical member local; (d) V5d vertical member local.

modes of the V5u and V5d vertical members. It is assumed here that the vibration energy in these modes mainly dissipated owing to the material damping of the vertical members. The equivalent loss factors of the arch ribs and girders were next estimated in Step 2 by considering the first symmetric vertical and torsional modes. The mean value of the vertical member loss factors separately estimated in the first step is the known parameter in the second step. Non-negative least squares were used to estimate the parameters. In Step 2 of Case 1, the dynamic friction coefficient μ_s= 0.005 is based on the current research on the dynamic friction coefficient of metal bearings, simply because the maximum number of unknowns is limited to two owing to the two vibration modes accurately identified in the field measurements. Whereas, support friction damping was neglected in Step 2 of Case 2 as there was no measurable support movement at corresponding FVs.

Different loss factors were estimated for different vibration levels with relatively large coefficient of variation. The detailed results of the analysis is included in Dammika et al (2014a). The appropriateness of estimated equivalent loss factors is explained in there by the fact that all estimated values are larger than the loss factor of steel, which ranges from 0.0001 to 0.0006, reflecting the assumed energy dissipation in the structural joints.

Although proposed damping parameter estimation method identified reasonable equivalent loss factors for different damping sources, the fluctuation in the identified loss factors cannot be negligible. The proposed damping parameter estimation was based on the experimentally identified modal damping ratios of the bridge which were identified by following the screening process discussed. Further improvement in the experimental modal damping identifications from field vibration measurements is required to more accurately identify damping loss factors. In addition, the number of vibration modes identified, which was larger than the number of unknown damping loss factors, had effects on the estimation of loss factors by applying the least-square method and caused the fluctuations observed in the results.

5.2 Analytically evaluated modal damping ratios

The modal damping ratios were analytically evaluated by back substituting the averages of estimated loss factors to Eq. (2) for both Cases 1 and 2. The comparison of obtained results with the experimentally identified values were also included in Dammika et al (2014a). Well matching between analytically evaluated and experimentally identified modal damping ratios from most of FV records can be seen in results shown there. Hence, authors concluded that the proposed analytical method, which is based on field vibration, can evaluate the modal damping ratios reliably. Furthermore, as most important output of the analysis, the proposed analytical method can quantitatively evaluate the damping contributions from different damping sources to each modal damping ratio.

Referring to the capabilities of the discussed damping evaluation method, it could be able to consider as a new framework for damping change-based SHM of bridges which combined both experimental and analytical approaches. The application of the framework in damage detection and, possible improvement in analytical damping evaluation and experimental damping identification, which can improve the reliability of the method are discussed in next sections.

6. DAMAGED COMPONENT DETECTION OF STEEL BRIDGES BASED ON MODAL DAMPING CHANGES

When the dynamic characteristics of steel truss bridge or steel arch bridge is considered, if local members like diagonal or vertical members of those bridges have damaged, they can be easily coupled with global vibration modes of those bridges. Modal damping changes can be expected in such coupled vibration modes due to coupling of damaged members as it was experimentally revealed by Yoshioka et al. (2010) for a steel truss bridge. More importantly, experimental identification of such vibration modes of the bridges can be done by placing appropriate minimum number of sensors as employed in the truss bridge studied by Yoshioka et al. (2010). It could be an advantage in practical application of SHM of bridges. Once the modal damping change is detected in a global vibration mode of a bridge respect to its early stage of the life span, it can be due to the existence of damages. Analytical damping evaluation can be conducted to verify the identified damping changes while finding the responsible component categories for such damping change.

On the other hand, vertical or diagonal member coupled modes are one of the vibration characteristic of steel truss and arch bridges. For an example, V4 vertical member coupled mode in studied arch bridge (Fig. 2 (e)) explained the previous observation. If some damage occurred in a local member which could have coupled vibration characteristic, then modal damping changes can be expected in those coupled modes. Field vibration measurement of these types of bridges can also be conducted with the target of such vibration modes by utilizing minimum number of sensors.

Therefore, as a practical method of steel bridge monitoring, discussed damping change-based SHM framework can be easily applied to detect damaged components of steel bridges targeting the damping change in their local member coupled modes.

7. IMPROVEMENT IN ANALYTICAL DAMPING EVALUATION

As it is discussed in previous, the estimated damping parameters of steel arch bridge are highly fluctuating among different FV records. The errors that can be associated in discussed analytical approach including errors in FE analysis can cause on the accuracy of estimated damping parameters and analytically evaluated damping ratios. Therefore, improvement is required in analytical approach to improve the reliability of the introduced SHM framework, and such possible improvement is discussed in this section.

One of the main assumptions of analytical damping evaluation method introduced in this study is summing up of damping contribution from all damping sources to the objective bridge's modal damping. Therefore, inclusion of all possible damping sources of the bridge to its analytical damping model is important to make improvement in analytical damping evaluation. On the other hand, detailed FE modelling of the bridges is also playing an important role in accuracy of studied analytical damping evaluation method. Detailed FE model can improve the validity of the model as simplified assumptions used in modeling can be reduced. Detailed FE models of the bridges is a compulsory requirement when diversified damping model is used in which damping contribution from different structural components are considered as separate damping sources. Such detailed damping evaluation by considering diversified damping sources may improve not only the accuracy of analytical damping evaluation but also it may provide space for damage detection of any part of bridges based on existence of damping changes in relevant component categories.

8. IMPROVEMENT IN EXPERIMENTAL DAMPING IDENTIFICATION

8.1 Background of the study

The fluctuation that exists in experimentally identified modal damping ratios is the main source that cause to the fluctuations exists in analytically estimated damping parameters of the studied steel arch bridges. Therefore, stable modal damping identifications of bridges with less fluctuation are important to improve the reliability of estimated damping parameters of the bridges. Possible improvement in modal damping identification was studied by applying experimental modal identification to few pre-stressed concrete (PC) bridges with different ages and discussed in here.

The selection of PC bridges for this study has another objective other than the possible improvement in experimental damping identifications. In PC bridges, the loss of pre-stressing force and rupture of pre-stressing tendons due to corrosion damage has been identified as a serious damage which could lead to catastrophic failures without early warning that can be detected during regular visual inspections (Pearson, et al. 2004). Thus, early detection of such invisible damages is important to ensure the safety of structures. Therefore, as a possible technique, vibration-based SHM techniques can be used. When vibration-based SHM techniques are considered, damping-based techniques could be a sensitive damage indicator against corrosion induced damages in PC bridges as it may have the possibility to detect the nonlinear, dissipative effects produced by cracks and other internal defects (Sohn, et al. 2004). Therefore, stable modal damping identification of PC bridges is required in order to use them in their SHM purposes.

8.2 Details of PC bridges

A set of PC bridges were selected for the study which were having service life ranging between 20 to 36 years as shown in Table 3, including the details of their geometric configurations and specifications of PC girders. All the selected bridges were single span structures located over drains and canals. The criteria for the selection of bridges were the similarity of geometric and structural configuration which would theoretically exhibit similar dynamic characteristics. Three skew PC bridges (Bridges I-III) were selected based on this criterion while the fourth one was non-skew bridge (Bridge IV) with slightly different configuration. The structural form of the bridges was in-situ slab on pre-tensioned concrete girders together with concrete infill in between girders at their top flange level. Cross diaphragms were also located at both end of the girders and the middle of the span. The bridge deck was transversely post-tensioned through the top flange and across the web of the PC girders through the cross diaphragms. Each PC girder was rest on bearing pads kept on abutments. Typical cross section of bridges is shown in Fig. 8.

8.3 Field vibration measurement and extraction of FV records

The vibration measurement was performed during the regular service of the bridges. Field vibration measurement of Bridge IV was conducted in the beginning of 2013. Four wired Servo type velocimeters (SPC-51, Tokyo Sokushin Co.,Ltd.) were placed in order to detect vertical bending and torsional vibration of bridge deck. Several number of vibration measurements were recorded when vehicles passing over the bridge with the sampling frequency of 200Hz. Field vibration measurements of Bridges I-III were next conducted at the end of 2013. Wireless acceleration sensing system composed of six wireless channels (BHELMO, JIP Techno Science Cooperation) were used for acceleration measurements with the sampling frequency of 100Hz. The sampling duration of the wireless sensors was 3 minutes due to the limitation of the wireless data acquisition system and, therefore, a number of sample measurements were acquired. The video camera was set to capture the traffic flow during the entire measurement period of every bridge.

The recorded bridge acceleration data were post processed and separated into a number of FVs corresponding to the conditions when bridge underwent free vibrations after passing of each individual vehicle. The FV time histories of each bridge were recognized by looking at both traffic data video and acceleration time history plots.

Table 3. Details of investigated PC bridges.

Name	Compl. year	Length (m)	Width (m)	Skew (°)	PC girder type(Nos.)
Bridge I	1978	19.60	9.2	14.0	Bulb Tee* (9)
Bridge II	1985	19.30	16.8	26.5	Bulb Tee* (16)
Bridge III	1991	17.30	12.8	30.0	Bulb Tee* (13)
Bridge IV	1994	14.95	11.0	0.0	Tee** (11)

* follows the specification of JIS A 5316

** follows the specification of JIS A 5316-1991

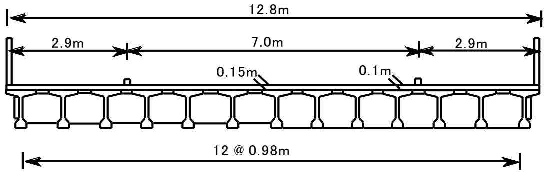


Fig.8. Typical section of PC Bridges (Bridge III).

8.4 Modal identification by ERA

The extracted FV time histories from all the bridges were utilized in modal identifications of the respective bridges by applying ERA. Results of modal identification, especially damping identification, by ERA were usually fluctuated for different system orders and therefore results were subjected to step by step screening processes in order to increase the diagnostic accuracy. In the first step of screening, identifications with the Modal Amplitude Coherence (MAC) (Juang & Pappa 1985) greater than 0.9 and with the modal damping ratios of non-negative and less than 10 % were selected. The EMAC values calculated for modal identifications of three PC bridges where wireless sensors were used appeared to be comparatively less and therefore its modal identifications were not screened based on EMAC value. Modal identifications with EMAC greater than 0.6 were selected for the bridge with wired sensor measurements. This different behavior in EMAC values with respect to two measurement systems may occur due to lower signal-to-noise ratio for the wireless sensor system.

In the second step of screening, the judgment of identical modes was assured by their stability for more than 10 sequential system order and the modal assurance criterion values, calculated by mode vector in system order n and n+2, of more than 0.9. Furthermore, the maximum limit of difference of frequency was kept at 0.1 and limit of difference of modal amplitude ratio was kept in between 0.8 – 1.2. Modal identifications having comparatively less modal amplitudes were removed at the end of this step to avoid the results from the data with low signal-to-noise ratio. The mean values of modal parameters identified with different system orders were obtained from this step of screening process. The identified natural frequencies from each FV time history are plotted against its system order and compared with spectrum of corresponding acceleration records as shown in Fig.9 in order to check their existence. Though two steps of screening process were followed, the results of damping identification were still found to be fluctuating for different system orders. Therefore, another step of screening process was conducted in order to identify modal damping ratio more reasonably and stably, which is explained in Section 8.6.

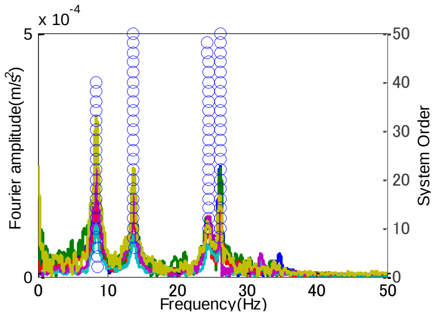


Fig.9. Spectrum and system identification-Bridge III (FV 9).

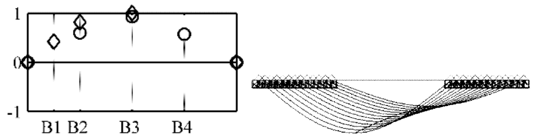
8.5 Theoretical modal analysis

FE models were created for eigenvalue analysis of all the studied bridges. All PC girders and transverse diaphragms located at both ends and the middle of the span were modeled by general sectioned beam element with assigned mass and geometric properties. The equivalent section concept was utilized in modeling the PC girders together with intermediate infill slab at top flange level of the girders. Concrete deck slab was modeled with four node quadrilateral plate elements. The material density of concrete was considered as 2400 kN/m^3 and the elastic modulus of concrete materials used for PC girders and rest of the concrete elements were considered as 33 GPa and 25.6 GPa, respectively, while the Poisson ratio was considered as 0.2. The mass of the asphalt layer was added as distributed load on deck slab elements. The vertical translational degrees of freedom at either end of each PC girder was restrained and spring elements were used to model the action of the bearing pads in its two horizontal translational directions in the plane of bearing pads. The pre-tensioned conditions of the bridge were not separately modeled in FE models and it was assumed that the stiffness properties calculated based on un-cracked sections of respective concrete members may represent the effect of pre-tensioning and post-tensioning.

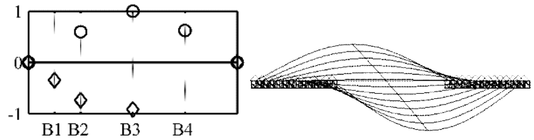
Four mode shapes namely, symmetric vertical bending (SV); symmetric torsional (ST); symmetric vertical-transverse bending (SVT) and asymmetric vertical-transverse bending (AsVT) were analytically obtained for most of the bridges from the FE models. Figure 10 compares the analytically obtained mode shapes of Bridge III with its experimentally identified mode shapes (from FV2) followed by two steps of screening. The plate elements are not shown in analytical mode shapes shown in Fig.10 for the clarity of the figures.

8.6 Improvement of modal damping identification

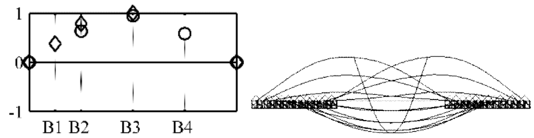
As results of modal damping identifications were fluctuated for different system orders even after two steps of screening described, the stability of the identified modal damping ratios against the system order was examined at the third step of screening. The ranges of system order which gave stable modal damping ratios with less fluctuation for respective modal identifications were identified for each FV time histories of all the bridges. It was observed that most of the modal damping identifications were highly fluctuating in their initial appearances with low system orders. Furthermore, modal damping identifications from some of the FV time histories were not stable throughout all system orders used in the analysis and, therefore, such identifications were removed at this step. The mean values of natural frequencies and modal damping ratios from their stable appearance range in the system order were taken as experimental identifications from respective FV time histories. The results of modal damping identifications of Bridge-III after second and third screening steps are shown in Fig.11 (a) and (b), respectively as an example of the screening process. As can be seen in the figures, damping identifications from FV 3, 4 and 5 were removed during third step of screening while including only the stable modal damping identifications from other FV time histories. The figures demonstrate the possibility of achieving stable modal damping identifications of the PC Bridge through the screening procedures used in this study. Summary of the modal identifications of all PC bridges obtained after all screening steps were included in Dammika et al (2014b) with detailed discussion on of it.



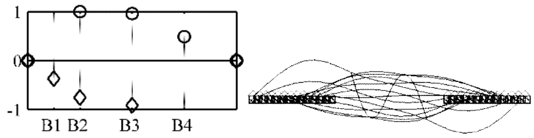
(a) symmetric vertical bending mode.



(b) symmetric torsional mode.

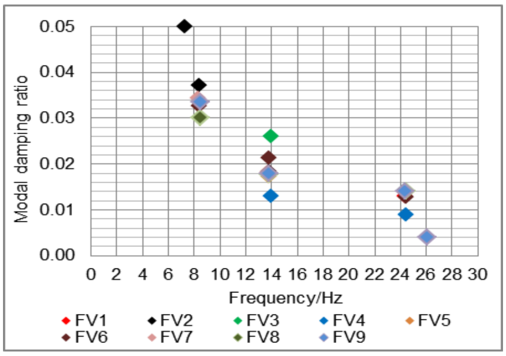


(c) symmetric vertical-transverse bending mode.

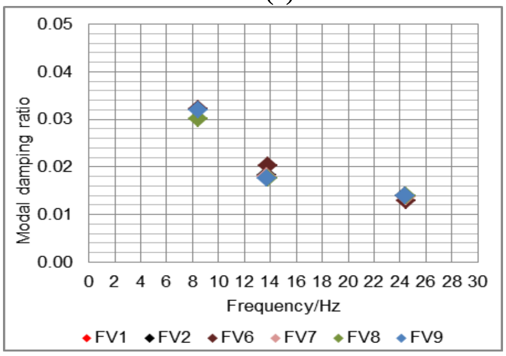


(d) asymmetric vertical-transverse bending mode.

Fig.10. Comparison of experimental and analytical mode shapes of Bridge III.



(a)



(b)

Fig.11. Variation of natural frequency vs modal damping ratio of Bridge III: (a) after second step of screening; (b) after third step of screening.

In conclusion, reffering to the obtained results of all studied bridges, reliability of modal identification was improved by properly screening and correctly interpreting the results of experimental identification. Further study may be required to develop an automatic screening process for damping identification and, more importantly, establish a quantitative relation between damping parameter and degree of damage or deterioration.

9. CONCLUSIONS

In this study, an energy-based analytical method for evaluating the modal damping ratios of bridges is introduced and tested on a steel arch bridge to complement experimental SHM. Then, an application of the method for damage component detection of steel bridges is also discussed. Furthermore, possible improvement in analytical damping evaluation and experimental damping identification is also discussed. The main conclusions of the study are as follows:

1. The proposed modal damping evaluation can estimate the mode-independent damping parameters of energy-dissipating sources in bridges; hence, modal damping ratios can be analytically evaluated.
2. The proposed modal damping evaluation can quantify the damping contributions of different damping sources to the modal damping ratios of the bridge; hence, it improves the reliability of the experimental damping identification.
3. The proposed analytical method can be used to complement the detection of damaged bridge components by using experimentally identified modal damping variations and checking the existence of significant changes in the damping contribution from the relevant components.
4. With the introduction of diversified damping models for the bridges together with its detailed FE models, the accuracy of analytical damping evaluation can be improved as contribution from each damping source to its modal damping ratios can be accounted.
5. Detailed FE models can also improve the accuracy of analytical damping evaluation by producing the accurate enough numerical representation of the bridges.
6. Detailed damping evaluation of bridges can also be used for damage detection of any part of the bridges based on experimentally identified modal damping changes.
7. Improvement in experimental damping identification is studied through an investigation conducted for stable modal damping identification of existing PC bridges. The reliability of modal damping identification was improved by properly screening and correctly interpreting the results of experimental identification.
8. Possibility of using modal damping ratio as an indicator of invisible damages in PC bridges is also emphasized through this study. More importantly, establishing a quantitative relation between damping parameter and degree of damage or deterioration could be necessary when using modal damping for detection of invisible damages in PC bridges in practice.

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