Journal of the Korean Society for Nondestructive Testing
Vol. 30, No. 6 (2010. 12)

Nondestructive Evaluation of Railway Bridge by System Identification Using Field Vibration Measurement

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Abstract This paper presents a nondestructive evaluation approach for system identification (SID) of real railway bridges using field vibration test results. First, a multi-phase SID scheme designed on the basis of eigenvalue sensitivity concept is presented. Next, the proposed multi-phase approach is evaluated from field vibration tests on a real railway bridge (Wondongcheon bridge) located in Yangsan, Korea. On the steel girder bridge, a few natural frequencies and mode shapes are experimentally measured under the ambient vibration condition. The corresponding modal parameters are numerically calculated from a three-dimensional finite element (FE) model established for the target bridge. Eigenvalue sensitivities are analyzed for potential model-updating parameters of the FE model. Then, structural subsystems are identified phase-by-phase using the proposed model-updating procedure. Based on model-updating results, a baseline model and a nondestructive evaluation of test bridge are identified.

Keywords: Model Update, Multi-Phase, System Identification, Railway Bridge, Field Vibration Test, Structural Health Monitoring

1. Introduction

During past decades, many steel girder railway bridges have been constructed for civil infrastructures. Therefore, the interest on the safety assessment of existing railway bridges has been increasing. Unfortunately, the occurrence of damage in those bridges is inevitable because they are subjected to extreme loading and environmental conditions not anticipated in design process. The damages occurred in critical subsystems of those bridges, which are not identified in timely manner, may result in tragic collapse of the bridges. To secure the serviceability and safety of the bridges, the structural health monitoring (SHM) has become an important topic. By monitoring the critical damage at its early stage, the structural safety and integrity will be guaranteed and the time and costs associated with appropriate maintenance and repair will be reduced (Adams et al., 1978; Doebling et al., 1998; Kwon et al., 1998; Yau and Yang, 2004; Feng, 2007).

The development of methodology for accurate and reliable condition assessment of civil structures has become increasingly important in recent years. In structural engineering analysis and design, the finite element (FE) analysis is a powerful and useful tool to simulate the behavior of real structures. Hence, accurate FE model is prerequisite for civil engineering applications such as damage detection, health monitoring and structural control. For complex structures, however, it is not easy to generate accurate baseline FE models for the use of SHM, because material properties, geometries, boundary

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conditions, and ambient temperature conditions of those structures are not completely known (Kim and Stubbs, 1995; Kim et al., 2007). Due to those uncertainties, an initial FE model based on as-built design may not truly represent all the physical aspects of an actual structure. Consequently, there exists an important issue on how to update the FE model using experimental results so that the numerically analyzed structural parameters match to the real experimental ones.

Many researchers have proposed model-updating methods for system identification (SID) by using vibration test results (Friswell and Mottershead, 1995; Stubbs and Kim, 1996; Zhang et al., 2000; Jaishi and Ren, 2005; Yang and Chen, 2009). Among those methods, the eigenvalue sensitivity-based algorithm has become one of the most popular and effective methods to evaluate baseline models for structural health assessment (Kim et al., 1997; Brownjohn et al., 2001; Wu and Li, 2004). The FE model update is a process of making sure that FE analysis results better reflect the measured data than the initial model. For the vibration-based SID for nondestructive evaluation, this process is conducted in the following steps: (1) measure vibration data to be utilized; (2) determine structural parameters to be updated; (3) formulate a function to represent the difference between the measured vibration data and the analyzed data from FE model; and (4) identify parameters to nondestructively evaluate the structure (Friswell and Mottershead, 1995; Kwon and Lin, 2004).

The objective of this paper is present a nondestructive evaluation approach for SID of real railway bridges using field vibration test results. In order to achieve the objective, the following approaches are implemented. First, a multi-phase SID scheme designed on the basis of eigenvalue sensitivity concept is presented. Next, the proposed multi-phase approach is evaluated from field vibration tests on Wondongcheon bridge which is a steel girder railway bridge located in Yangsan, Korea. On the bridge, a few natural frequencies and mode shapes are experimentally measured under the ambient vibration condition. The corresponding modal parameters are numerically calculated from a three-dimensional FE model which is established for the target bridge. Eigenvalue sensitivities are analyzed for potential model-updating parameters of the FE model. Then, structural subsystems are identified phase-by-phase using the proposed model-updating procedure. Based on model-updating results, a baseline model and a nondestructive evaluation of test bridge are identified.

2. Multi-Phase SID Scheme for Nondestructive Evaluation

Based on eigenvalue sensitivity concept that relates experimental and theoretical responses of the structure (Adams et al., 1978; Stubbs and Osegueda, 1990; Stubbs and Kim, 1996) proposed a model-updating method to identify a realistic theoretical model of a structure. Relative to the FE model, the fractional structural parameter change of the j^{th} member, $\alpha_j \geq -1$, and the structural parameters are related according to the following equation:

$$p_j^* = p_j(1+\alpha_j) \quad (1)$$

where p_j^* is an unknown structural parameter of the j^{th} member of a structure for which M eigenvalues are known; p_j is a known parameter of the j^{th} member of a FE model for which the corresponding set of M eigenvalues are known.

The fractional structural parameter change α_j can be estimated from the following equation (Stubbs and Osegueda, 1990):

$$Z_i = \sum_{i=1}^{M} S_{ij}\alpha_j \quad (2)$$

where Z_i is the fractional change in the i^{th} eigenvalues between two different structural systems (e.g., an analytical model and a real structure). Also, S_{ij} is the dimensionless sensitivity of the i^{th} eigenvalue ω_i^2 with respect to the j^{th} structural parameter p_j (Stubbs and Osegueda, 1990; Zhang et al., 2000).

$$S_{ij} = \frac{\delta\omega_i^2}{\delta p_j}\frac{p_j}{\omega_i^2} \tag{3a}$$

$$Z_i = \frac{\delta\omega_i^2}{\omega_i^2} \tag{3b}$$

where δp_j is the first order perturbation of p_j which produces the variation in eigenvalue $\delta\omega_i^2$.

The fractional structural parameter change of NE members may be obtained using the following equation:

$$\{\alpha\} = [S]^{-1}\{Z\} \tag{4}$$

where $\{\alpha\}$ is a $NE \times 1$ matrix, which is defined by eqn. (1), containing the fractional changes in structural parameters between the FE model and the target structure; $\{Z\}$ is defined as eqn. (3b) and it is a $M \times 1$ matrix containing the fractional changes in eigenvalues between two systems; and $[S]$ is a $M \times NE$ sensitivity matrix, which is defined by eqn. (3a), relating the fractional changes in structural parameters to the fractional changes in eigenvalues. The sensitivity matrix, $[S]$, is determined numerically in the following procedure (Stubbs and Osegueda, 1990); (1) Introduce a known severity of damage (α_j, $j = 1, NE$) at j^{th} member; (2) Determine the eigenvalues of the initial FE model (ω_{io}^2, $i = 1, M$); (3) Determine the eigenvalues of the damaged structure (ω_i^2, $i = 1, M$); (4) Calculate the fractional changes in eigenvalues by $Z = (\omega_i^2/\omega_{io}^2 - 1)$; (5) Calculate the individual sensitivity components from $S_{ij} = Z_i/\alpha_j$; and (6) Repeat steps (2)-(5) to generate the $M \times NE$ sensitivity matrix.

For the above method, if the number of structural parameters is much larger than the number of modes, i.e., $NE \gg M$, the system is ill-conditioned and eqn. (4) will not work properly, which is a typical situation for civil engineering structures. To produce stable solution, therefore, the number of structural parameters should be equal to or less than the number of modes, $NE \le M$. In addition, for most complex structures, only a few vibration modes can be measured with good confidence and many sub-structural members are combined together with complex response motions in the vibration modes. In order to overcome these problems, a multi-phase model-updating approach is needed to be implemented for updating the FE models of the complex structures.

For a target structure which has experimental modal parameters ($\Phi_{i,m}$, $\omega_{i,m}^2$, $i = 1, M$), a multi-phase SID for nondestructive evaluation is designed as schematized in Fig. 1. First, an initial FE model is established to numerically analyze modal parameters ($\Phi_{i,a}$, $\omega_{i,a}^2$, $i = 1, M$). Second, NE structural parameters (p_j, $j = 1, NE$) are selected by grouping the FE model into NE sub-structures and analyzing modal sensitivities of the NE parameters up to M modes. Third, the number of phases NP is determined by computing $NP = NE/M$ and arrange the M number of structural parameters (p_j, $j = 1, M$) for each phase. Finally, the following five sub-steps are performed for phase K (i.e., $K = 1, NP$):

(1) Compute numerical modal parameters of a selected FE model;
(2) Compute sensitivities of structural parameters and the fractional change in eigenvalue between the target structure and the updated FE model (i.e., $M \times 1$ $\{Z\}$ matrix);
(3) Fine-tune the FE model by first solving eqn. (4) to estimate fractional changes in structural parameters (i.e., $NE \times 1$ $\{\alpha\}$ matrix) and then solving eqn. (1) to update

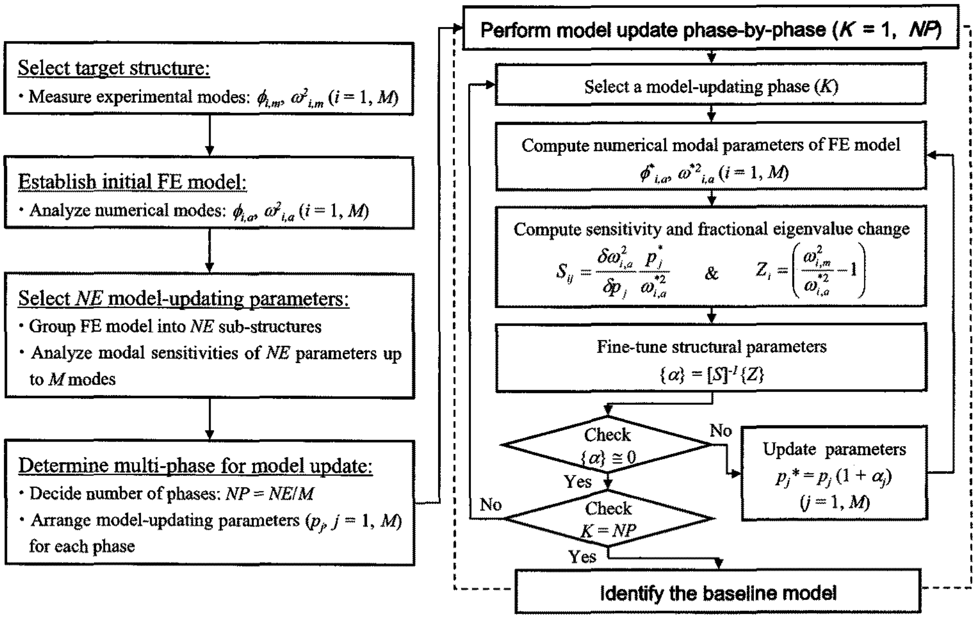


Fig. 1 Multi-phase SID scheme for nondestructive evaluation

the structural parameters of the FE model;

(4) Repeat the whole procedure until $\{Z\}$ or $\{\alpha\}$ approach zero when the parameters of the FE model are identified; and

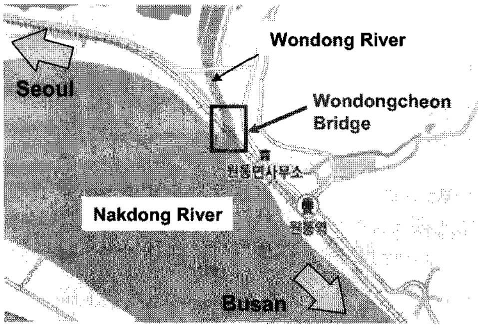
(5) Estimate the baseline model after the parameters are identified from phase K.

In each phase, the selection of structural parameters is based on the eigenvalue sensitivity analysis and the number of available modes. Primary structural parameters which are more sensitive to structural responses will be updated in the prior phases. It is also expected that the error will be reduced phase after phase, and, as a result, the accuracy of the baseline model will be improved consequently. Note that numerical modal analysis is performed by using commercial FE analysis software.

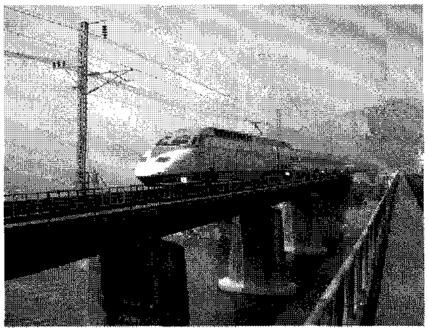
3. Field Vibration Test on Wondongcheon Railway Bridge

3.1 Description of Test Bridge

The Wongdongcheon railway bridge located on Nakdong river, Yangsan, Korea, is one of the bridges on train line between Seoul and Busan (Fig. 2). Two parallel bridges were constructed, one for the train line from Seoul to Busan, and one for the opposite line. The bridge is a steel girder railway bridge by five simply supported spans (i.e., G1-G5), and has a total length 99.1 m (Fig. 3). Each span consists of two main I-type girders, stiffeners, brace system, cover plates reinforced at mid-span, wooden sleepers, rail system and walkway system. The details of target span (i.e., span G1) of test bridge are shown in Fig. 4. The main I-type girder reinforced four flange angles L-150×150×15 mm has the cross section I-1880×365×12×12 mm. The angle sections L-90×125×10 mm and L-90×90×10 mm were used for the stiffeners and brace system, respectively. The brace system consists of diagonal top and bottom braces (i.e., horizontal braces), and X-type vertical braces. The sleepers used rectangular wooden bars 230×230 mm with 3.1 m in length. The walkway system includes a steel deck supported by frames and handrail placed along the bridge.



(a)



(b)

Fig. 2 Wondongcheon railway bridge; (a) Bridge location, (b) Real view

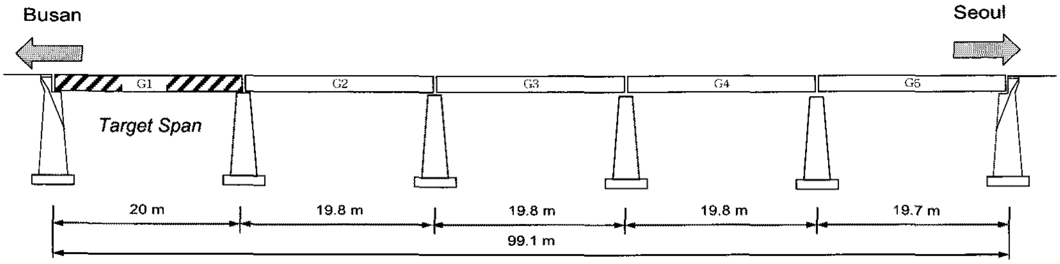


Fig. 3 Global view of test bridge

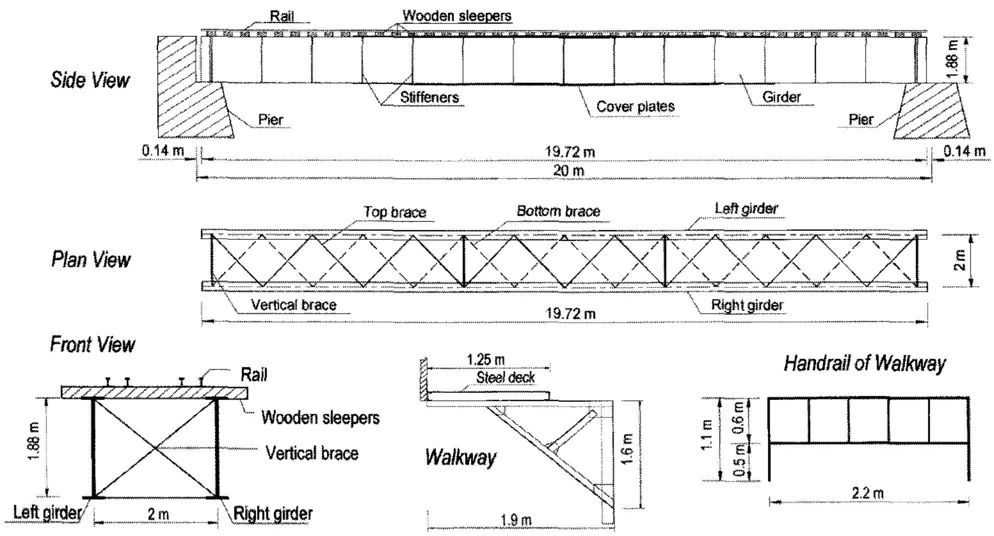
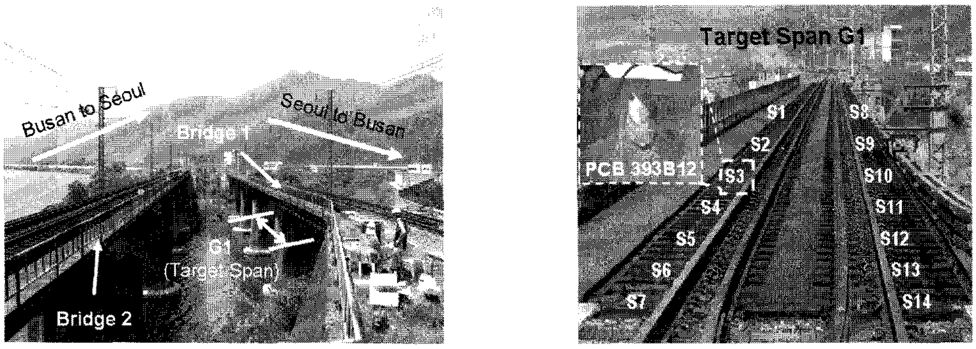


Fig. 4 Schematic of target span G1 of test bridge

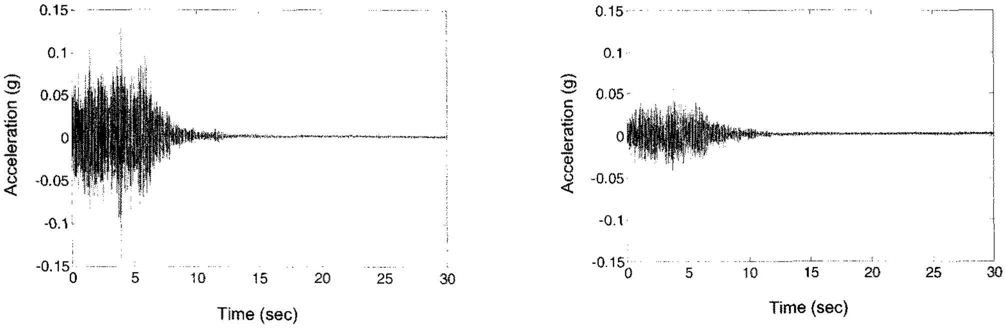
3.2 Field Vibration Tests on Test Bridge

Dynamic tests were performed on the Wondongcheon railway bridge to determine the experimental modal parameters under ambient vibration condition. Fig. 5 shows the field vibration test setup on the bridge. The span G1 of test bridge for the train line from Seoul to Busan was selected as the target span (Fig. 5a). For acceleration measurement, fourteen ICP-type accelerometers (i.e., S1-S14), PCB 393B12, with the nominal sensitivity of 10 V/g and the



(a) (b)

Fig. 5 Field vibration test on Wondongcheon railway bridge; (a) Two parallel bridges, (b) Experimental setup on target span G1



(a) (b)

Fig. 6 Acceleration response signals of test bridge; (a) S3 sensor, (b) S10 sensor

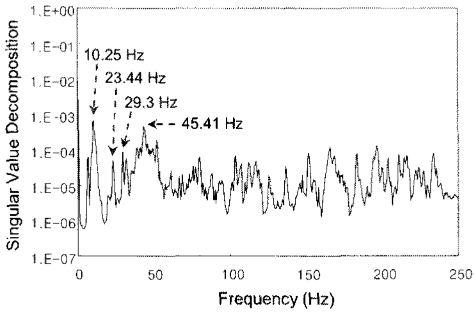
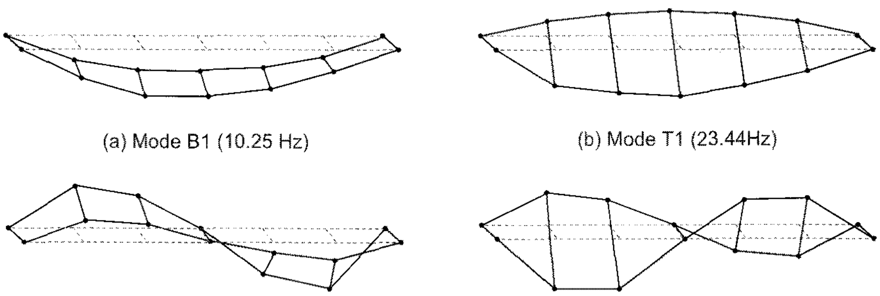


Fig. 7 Singular value decomposition of power spectrum density matrix



(a) Mode B1 (10.25 Hz) (b) Mode T1 (23.44Hz)

(c) Mode B2 (29.3 Hz) (d) Mode T2 (45.41 Hz)

Fig. 8 Experimental mode shapes of test bridge measurement range 0.5g were placed along the left and right girder with a constant interval 3.2 m. Fig. 5b shows the sensor locations and their arrangements on test bridge. The accelerometers were mounted on the top flange of the girder. The data acquisition system consists of a 16-channel PCB signal conditional, terminal block and MATLAB software. Dynamic responses in vertical direction were measured by the PCB accelerometers with the sampling frequency of 500 Hz. Impact force was applied to test bridge by the Korea Train eXpress (KTX) train which passed the adjacent bridge for the train line from Busan to Seoul (i.e., ambient vibration).

Fig. 6 shows the acceleration response signals measured from two sensors S3 and S10 out of the fourteen accelerometers. The frequency domain decomposition (FDD) method (Brincker et al., 2001; Yi and Yun, 2004) was employed to extract modal parameters from the acceleration signals. The singular value decomposition (SVD) of power spectrum density (PSD) matrix is shown in Fig. 7. Experimental mode shapes and natural frequencies of the first four modes that include two bending modes (i.e., mode B1 and mode B2) and two twist modes (i.e., mode T1 and mode T2) are given in Fig. 8.

4. Multi-Phase SID for Nondestructive Evaluation of Wondongcheon Railway Bridge

4.1 Initial FE Model of Test Bridge

A structural analysis and design software, SAP2000 (Computers and Structures Inc. 2005), was used to model the bridge. As shown in Fig. 9, the bridge was constructed by a three-dimensional FE model using shell and frame elements. For the boundary conditions, spring restraints were assigned at the supports. Initial values of material properties and boundary conditions of the FE model were assumed as follows: (1) for steel material, elastic modulus $E_s = 2\times10^{11}$ N/m^2, mass density $\rho_s = 7850\,\text{kg/m}^3$, and Poisson's ratio $\nu_s = 0.3$; (2) for wood material, elastic modulus $E_w = 2\times10^{10}$ N/m^2, mass density $\rho_w = 610\,\text{kg/m}^3$, and Poisson's ratio $\nu_w = 0.4$; and (3) the stiffness of vertical and horizontal springs $k_v = k_h = 10^9$ N/m. From the numerical modal analysis performed on the initial FE model, initial natural frequencies of the first two bending modes (i.e., mode B1 and mode B2) and two twist modes (i.e., mode T1 and mode T2) were computed. Fig. 10 shows the numerical mode shapes and natural frequencies analyzed from the initial FE model.

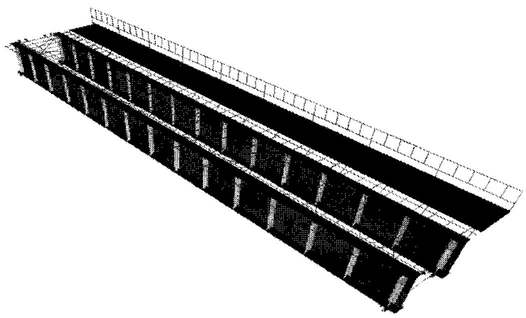
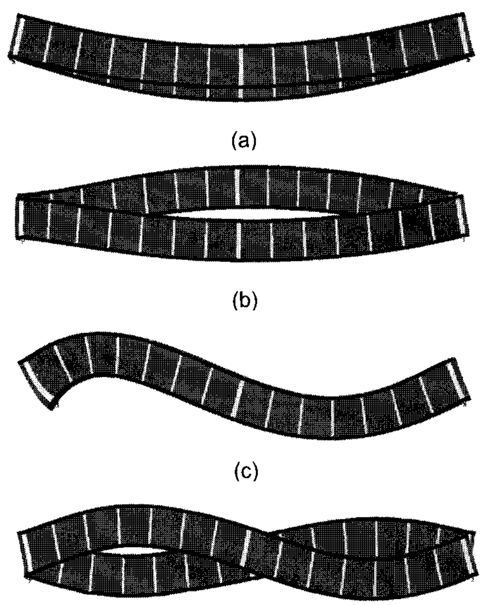


Fig. 9 Initial FE model of test bridge



(a)

(b)

(c)

(d)

Fig. 10 Numerical mode shapes of initial FE model; (a) Mode B1 (11.05 Hz), (b) Mode T1 (22.84 Hz), (c) Mode B2 (30.15 Hz), (d) Mode T2 (48.01 Hz)

4.2 Model-Updating Parameters of Test Bridge

Choosing appropriate structural parameters is an important step in the FE model-updating procedure. All parameters related to structural geometries, material properties, and boundary conditions can be potential choices for adjustment in the model-updating procedure. The model-updating parameters should be selected on the basis of the following two facts. Physically, the structural parameters which are uncertain in the model due to the lack of knowledge on structural properties should be selected. Also, the structural parameters which are relatively sensitive to vibration responses should be selected. The modal sensitivities with respect to the main structural parameters are first calculated and then the most sensitive and insensitive parameters to the responses are examined next.

As shown in Fig. 11, for test bridge, seven potential groups of model-updating parameters were selected as follows: (1) stiffness of girders $(EI)_g$, (2) stiffness of cover plates $(EI)_{cp}$, (3) stiffness of walkway's frames $(EI)_{wf}$, (4) stiffness of brace system $(EI)_b$, (5) stiffness of stiffeners $(EI)_s$, (6) stiffness of horizontal support springs $(k)_h$ and (7) stiffness of vertical support springs $(k)_v$. On estimating the initial FE model, the initial values of the model-updating parameters were assumed as follows: $(EI)_g = 2.81\times10^9\,\text{Nm}^2$, $(EI)_{cp} = 4.87\times10^4\,\text{Nm}^2$, $(EI)_{wf} = 1.18\times10^5\,\text{Nm}^2$, $(EI)_b = 2.56\times10^5\,\text{Nm}^2$, $(EI)_s = 6.36\times10^5\,\text{Nm}^2$ and $(k)_h = (k)_v = 10^9\,\text{N/m}$. Then, the eigenvalue sensitivity analysis for the seven groups of model-updating parameters was carried out, as summarized in Table 1. From the results, the stiffness of girders was the most sensitive parameter for both bending and twist modes. Meanwhile, the stiffness of walkway's frame and stiffness of brace system were more sensitive for twist modes. The stiffness of stiffeners and stiffness of support springs were relatively less sensitive parameters, as shown in Fig. 12. That is, those less sensitive parameters were expected to contribute less intensively on the model update.

Due to the availability of the four modes, two model-updating phases were chosen in order to update the seven model-updating parameters. Based on their sensitivities as listed in Table 1, the order of model update was arranged as follows: 1) In phase I - stiffness of girders $(EI)_g$, stiffness of cover plates $(EI)_{cp}$, stiffness of walkway's frames $(EI)_{wf}$ and stiffness of brace system $(EI)_b$; 2) In phase II - stiffness of stiffeners $(EI)_s$, stiffness of horizontal support springs $(k)_h$ and stiffness of vertical support springs $(k)_v$.

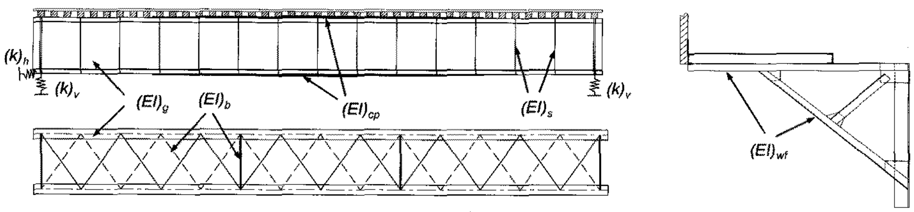


Fig. 11 Model-updating parameters for test bridge

Table 1 Eigenvalue sensitivities of seven model-updating parameters

Mode	Eigenvalue Sensitivities						
	$(EI)_g$	$(EI)_{cp}$	$(EI)_{wf}$	$(EI)_b$	$(EI)_s$	$(k)_h$	$(k)_v$
B1	0.46738	0.23903	0.07704	0.01760	0.01690	0.00126	0.00551
B2	0.69282	0.07092	0.05721	0.00434	0.03303	0.05026	0.00209
T1	0.22009	0.05586	0.15502	0.20310	0.02601	0.00789	0.00933
T2	0.69310	0.03225	0.03940	0.12055	0.08453	0.00031	0.00221

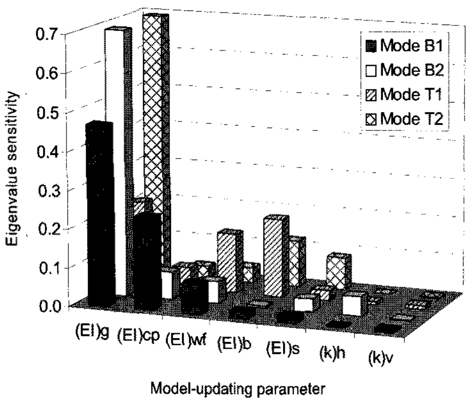


Fig. 12 Eigenvalue sensitivity analysis of seven model-updating parameters

4.3 Multi-Phase SID of Test Bridge

After selection of dynamic responses and structural parameters, an iterative procedure was carried out for model update. In this study, the baseline natural frequencies were identified by using the eigenvalue sensitivity-based multi-phase model-updating method described previously and also depicted in Fig. 1. It should be noted that two phases were considered for updating the model. These phases are updated by phase-by-phase. Four and three model-updating parameters were iteratively updated for phase I and phase II, respectively. Consequently, the analytical natural frequencies determined at the end of iterations should gradually approach those experimental values.

The SID results are summarized in Table 2 and also shown in Figs. 13 and 14. For test bridge, natural frequencies of the first two bending modes and two twist modes were used to update the FE model through fifteen iterations (i.e., ten iterations for phase I and five iterations for phase II). Table 2 shows natural frequencies of FE model during fifteen iterations of multi-phase model update. Fig. 13 shows convergence errors of the natural frequencies of updated model during fifteen iterations, with compared to target natural frequencies. Natural frequencies were converged within 0.29%, 0.0%, 0.28% and 0.0% error for the mode B1, mode B2, mode T1 and mode T2, respectively. Also,

Table 2 Natural frequencies of FE model during multi-phase model update

Mode	Initial Freqs. (Hz)	Updated Frequencies (Hz) at Each Iteration															Target Freqs. (Hz)
		Phase I										Phase II					
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	
B1	11.05	10.18	10.22	10.21	10.25	10.25	10.25	10.25	10.25	10.29	10.31	10.26	10.27	10.28	10.28	10.28	10.25
B2	30.15	28.09	29.09	28.88	28.87	28.99	28.99	28.99	28.97	29.07	29.13	29.18	29.27	29.30	29.30	29.30	29.30
T1	22.84	23.26	23.02	23.26	23.39	23.38	23.39	23.41	23.42	23.44	23.43	23.33	23.36	23.37	23.38	23.38	23.44
T2	48.01	45.21	45.77	45.35	45.43	45.44	45.42	45.44	45.41	45.60	45.69	45.37	45.40	45.41	45.41	45.41	45.41

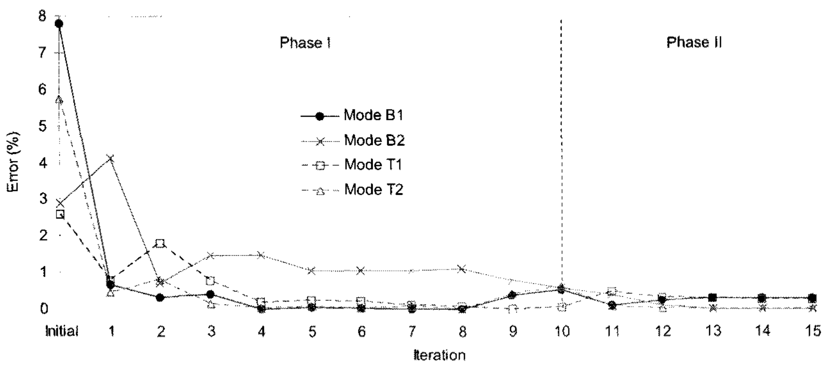


Fig. 13 Convergence errors of natural frequencies between experiment and FE model

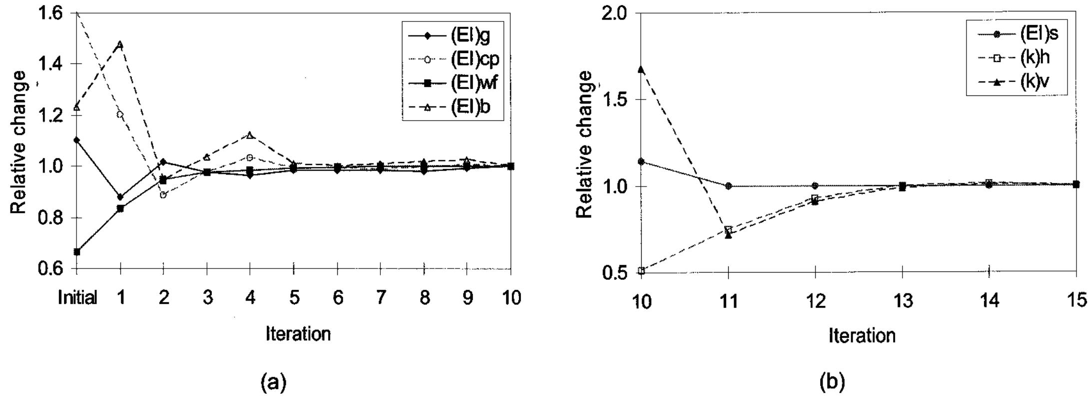


Fig. 14 Relative change in model-updating parameters of FE model; (a) Phase I, (b) Phase II

the relative changes in structural parameters of FE model due to phase-by-phase model update are shown in Fig. 14. Those fractional changes in model-updating parameters were computed with respect to tenth iteration for phase I and fifteenth iteration for phase II as the reference. As shown in Fig. 14, the parameters have a large change in several early iterations and converge in later iterations (i.e., converge to one). Based on model-updating results, a numerical baseline model which represents dynamic responses of test bridge can be analyzed in terms of the model-updating parameters. Finally, the identified baseline model can be the role of the reference structure to make diagnosis or prognosis on the structure of interest.

4.4 Nondestructive Evaluation of Structural Parameters of Test Bridge

In order to evaluate the current state of bridge, we assume that the designed system (i.e., designed bridge) is represented by initial FE model and the aged system (i.e., aged bridge) is represented by updated FE model. Each system consists of structural parameters and modal parameters. For the designed system, structural parameters may include structural geometries, material properties and boundary conditions; meanwhile, modal parameters (i.e., natural frequencies and mode shapes) are usually obtained from numerical analysis. For the aged system, structural parameters are not available; meanwhile, modal parameters are obtained from experimental measurement. Therefore, the structural parameters of aged system need to identify by matching the modal parameters between two systems.

Based on the updated results, two systems (i.e., designed bridge and aged bridge) were compared. Tables 3 and 4 show values of natural frequencies and structural parameters of designed versus aged bridge, respectively. The natural frequencies of aged bridge were lower than the designed bridge for two bending modes and second twist mode; otherwise, the natural frequency of first twist mode is higher. Almost structural parameters of aged bridge were less than the designed one, except for stiffness of walkway's frames and stiffness of horizontal support springs. As shown in Table 1, the sensitivity of stiffness of walkway's frames to first twist mode was much higher than to two bending modes and second twist mode. Thus, stiffness of walkway's frames increased for the natural frequency convergence of first twist mode in model update. In addition, the stiffness of horizontal support springs assumed in the initial FE model was uncertain parameter. As a result, the current bridge was damaged to girders, cover plates, brace system and stiffeners.

Table 3 Value of natural frequencies (Hz): designed bridge vs aged bridge

Mode	B1	B2	T1	T2
Designed Bridge	11.05	30.15	22.84	48.01
Aged Bridge	10.28	29.30	23.38	45.41
Change (%)	-6.96	-2.80	2.36	-5.41

Table 4 Value of structural parameters: designed bridge vs aged bridge

Parameter	$(EI)_g$ Nm^2	$(EI)_{cp}$ Nm^2	$(EI)_{wf}$ Nm^2	$(EI)_b$ Nm^2	$(EI)_s$ Nm^2	$(k)_h$ N/m	$(k)_v$ N/m
Designed Value	2.81E+9	4.87E+4	1.18E+5	2.56E+5	6.36E+5	1.0E+9	1.0E+9
Aged Value	2.55E+9	3.03E+4	1.77E+5	2.08E+5	5.56E+5	1.96E+9	5.98E+8
Change (%)	-9.19	-37.64	49.98	-18.77	-12.65	95.56	-40.23

5. Summary and Conclusions

A nondestructive evaluation approach was developed for SID of real railway bridges using field vibration test results. In order to achieve the objective, the following approaches were implemented. First, a multi-phase SID scheme designed on the basis of eigenvalue sensitivity concept was presented. Next, the proposed multi-phase approach was evaluated from field vibration tests on Wondongcheon bridge which is a steel girder railway bridge located in Yangsan, Korea.

On the bridge, natural frequency and mode shape of first two bending modes and two twist modes were experimentally measured under ambient vibration condition. The corresponding modal parameters were numerically calculated from a three-dimensional FE model established for the target bridge. Eigenvalue sensitivities were analyzed for seven model-updating parameters which were selected for model adjustment. As a result, two phases were considered to deal with four available modes. From phase-by-phase identification of structural subsystems, good correlations and convergences of natural frequencies between identified FE model and the target bridge were obtained. Based on the updated results, a nondestructive evaluation of test bridge at current time was also performed.

Acknowledgement

This work was supported by a 2006 C&T innovation program (Grant No. 06coretech-B05) funded by Ministry of Construction & Transportation of Korean government. Also, graduate students participated in this study were supported by the 2nd Stage Brain Korea 21 (BK21) program of Korean government.

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