Bridge damage identification through moving vehicle laboratory experiments

Chul-Woo Kim[1], Mitsuo Kawatani[1], and Tatsuki Fujimoto[1]

[1] Kobe University, Kobe, Japan

ABSTRACT: This paper investigates feasibility of the damage identification method derived from the bridge-vehicle interactive system through a moving vehicle laboratory experiment of a scaled steel bridge model. In the experiment some cuts are applied to the bridge model for damages. Observations through the study demonstrate that locations and severities of damage are detectable using the proposed method.

1 INTRODUCTION

The potential economic and life-safety implications of early diagnosis investigation in structures have motivated a considerable amount of research into structural health monitoring. Structures in many engineering fields are examined through periodic monitoring with the intention of minimizing the safety risk on the one hand and lowering maintenance costs to the greatest extent on the other hand by carrying out rehabilitation at appropriate times (Wenzel & Pichler (2005)). For countries located in earthquake-prone regions, after earthquakes, structural health monitoring is useful for rapid condition screening. It is also intended to provide, very rapidly, reliable information related to structural integrity.

Usually, to define damage for bridge structures is difficult differently from other structures such as automobiles, aerial vehicles, etc. This is one reason why most precedent studies particularly addressing bridge health monitoring has specifically examined global change of modal properties and quantities of bridge structures. The fundamental concept of this technology is that a change in physical properties, such as reduced stiffness resulting from damage, will change these modal properties detectably (Rizos et al.(1990), Salawu (1997), Shifrin & Ruotolo (1999)).

For bridges with a long span, wind-induced vibrations are important dynamic sources. Even seismic records have been used for investigating system identification of a cable-stayed bridge (Siringoringo & Fujino (2006)). On the other hand, for short span bridges, which are insensitive (or sometimes impassive) to the wind load, normal traffic excitations are important dynamic sources. However, traffic-induced vibration is a kind of nonstationary process that strengthens with decreasing span length. This is one reason why most studies of the bridge health

monitoring (BHM) using modal identification have specifically addressed longer span bridges despite of short span bridges' great contribution to entire bridge structures.

This study is an attempt to use excitations from a specified vehicle such as an inspection car as a dynamic source for BHM because a moving vehicle can provide benefits such as ready excitement of the bridge. This paper investigates feasibility of the damage identification method derived from the bridge-vehicle interactive system (Kim and Kawatani (2008)) through a moving vehicle laboratory experiment of a scaled steel bridge model.

2 DAMAGE IDENTIFICATION METHODOLOGY

2.1 Equations of motion for bridge-vehicle interactive system

The compact matrix formation of the interactive system is obtainable as (Kim et al. (2005))

$$\begin{bmatrix} \mathbf{M}_{br} & \mathbf{0} \\ \mathbf{0} & \mathbf{M}_{vv} \end{bmatrix} \begin{Bmatrix} \ddot{\mathbf{q}}_r(t) \\ \ddot{\mathbf{q}}_v(t) \end{Bmatrix} + \begin{bmatrix} \mathbf{C}_{br} + \mathbf{C}_{cvb}(t) & \mathbf{C}_{bv}(t) \\ \mathbf{C}_{vb}(t) & \mathbf{C}_{vv} \end{bmatrix} \begin{Bmatrix} \dot{\mathbf{q}}_r(t) \\ \dot{\mathbf{q}}_v(t) \end{Bmatrix} + \begin{bmatrix} \mathbf{K}_{br} + \mathbf{K}_{cvb}(t) & \mathbf{K}_{bv}(t) \\ \mathbf{K}_{vb}(t) & \mathbf{K}_{vv} \end{bmatrix} \begin{Bmatrix} \mathbf{q}_r(t) \\ \mathbf{q}_v(t) \end{Bmatrix} = \begin{Bmatrix} \mathbf{f}_{bb}(t) \\ \mathbf{f}_{vv}(t) \end{Bmatrix} \tag{1}$$

Therein, $\mathbf{C}_{cvb}(t) \in \mathrm{R}^{nr \times nr}$ and $\mathbf{K}_{cvb}(t) \in \mathrm{R}^{nr \times nr}$ respectively denote the contribution of the moving vehicle's damping and stiffness to those of the bridge. Furthermore, $\mathbf{C}_{bv}(t) \in \mathrm{R}^{nr \times nv}$ and $\mathbf{K}_{bv}(t) \in \mathrm{R}^{nr \times nv}$ are the coupled damping and stiffness matrices between the bridge and vehicle systems, where *nv* denotes the DOFs of the vehicle model. The respective mass, damping, and stiffness matrices for the vehicle are $\mathbf{M}_{vv} \in \mathrm{R}^{nv \times nv}$, $\mathbf{C}_{vv} \in \mathrm{R}^{nv \times nv}$ and $\mathbf{K}_{vv} \in \mathrm{R}^{nv \times nv}$. $\mathbf{q}_r(t)$ and $\mathbf{q}_v(t)$ denote dynamic responses of bridge and vehicle, respectively. In addition, $\mathbf{f}_{bb}(t) \in \mathrm{R}^{nr}$ and $\mathbf{f}_{vv}(t) \in \mathrm{R}^{nv}$ respectively signify external force vectors of the bridge and vehicle.

A noteworthy point in Eq. (1) is that the system damping and stiffness matrices consist of time-variant coefficients. Consequently, the conventional frequency domain approaches are not directly relevant to identify damage in the bridge-vehicle interactive system, especially for short span bridges which are more easily affected by traffic loading than long span bridges.

2.2 Methodology for damage identification

The concept for the damage identification described in this paper is based on the fact that the stiffness distribution in the structure is induced to change as a result of damage. This change is detectable by measuring dynamic responses under an inspection car whose dynamic wheel loads or dynamic properties are known. To simplify the problem, the mass matrix of a bridge is assumed to be unaffected by damage. The damping matrix is affected by the change of stiffness because this study assumes Rayleigh damping for the bridge structure. Another assumption is that parameters of the intact bridge and vehicle model are estimated initially.

Subtracting linear equations for stiffness of a bridge from Eq. (1) yields Eq. (2) of a pseudo-static formulation to show the change of a bridge structure's stiffness.

$$\mathbf{K}_{br}\mathbf{q}_{rt}(t) = \mathbf{f}(t)\,,\ \mathbf{f}(t) \in \mathrm{R}^{nr} \tag{2}$$

In that equation, the force vector is definable as

$$\mathbf{f}(t) = \mathbf{f}_{b1}(t) - \mathbf{f}_{b2}(t)\,, \tag{3}$$

where $\mathbf{f}_{b1}(t)$ is the distributed dynamic wheel load at each node of an FE model and is directly measurable from the inspection car. $\mathbf{f}_{b2}$ (t) is the contribution of inertia and dissipation forces of the bridge.

The change of stiffness $\mathbf{K}_{br}$ in Eq. (2) provides information about the change of the bridge's health condition. Detection of the change in $\mathbf{K}_{br}$ is the basic concept of the damage identification methodology proposed in this paper. The structural stiffness matrix $\mathbf{K}_{br}$ is obtainable using the assembly operator $\mathbf{L}_e$ as

$$\mathbf{K}_{br} = \sum_{e=1}^{M} \mathbf{L}_e^T \mathbf{K}_{be}^g \mathbf{L}_e \tag{4}$$

Therein, M is the number of elements; $\mathbf{L}_e \in \mathrm{R}^{2nf \times n}$ provides the assembly operator of an element that transforms the element stiffness matrix to a structural stiffness matrix in which nf denotes the number of DOFs at an element node. $\mathbf{K}_{be}^g \in \mathrm{R}^{2nf \times 2nf}$ is an element stiffness matrix in the global coordinate given by Eq. (5).

$$\mathbf{K}_{be}^g = \mathbf{R}_e^T \mathbf{K}_{be}^i \mathbf{R}_e \tag{5}$$

In that equation, $\mathbf{K}_{be}^i \in \mathrm{R}^{2nf \times 2nf}$ is the element stiffness matrix of the intact state. In addition, $\mathbf{R}_e \in \mathrm{R}^{2nf \times 2nf}$ denotes the coordinate transformation matrix.

The change of the element stiffness is obtainable using the element stiffness index (ESI), which is defined as

$$\mu_e = \frac{\mathbf{K}_{be}^d}{\mathbf{K}_{be}^i}, \tag{6}$$

where μ_e is the element stiffness index which normalizes changes in the element stiffness, and $\mathbf{K}_{be}^d$ denotes the element stiffness of the e-th element of a damaged state.

Equation (4) of the structural stiffness matrix for a bridge can be rewritten for a damaged bridge using Eqs. (5) and (6) as

$$\mathbf{K}_{br} = \sum_{e=1}^{M} \mu_e \mathbf{L}_e^T \mathbf{K}_{be}^g \mathbf{L}_e \tag{7}$$

A noteworthy point is that the ESI value is unity for an intact bridge, meaning that the value of unity for μ_e (e = 1, …, M) is the reference value for this study.

Substituting the relation in Eq. (7) into Eq. (2) of the pseudo-static formulation yields

$$\sum_{e=1}^{M} \mu_e \mathbf{h}_e(t) = \mathbf{f}(t), \tag{8}$$

where $\mathbf{h}_e(t) \in \mathrm{R}^{nr}$ is a coefficient vector of the e-th element at time t. It is defined as

$$\mathbf{h}_e(t) = \left(\mathbf{L}_e^T \mathbf{K}_{be}^g \mathbf{L}_e\right) \mathbf{q}_r(t) \tag{9}$$

If $\mathbf{x} \in \mathrm{R}^m$ gives the vector of ESI of a bridge, which is definable as Eq. (10) and $\mathbf{H}(t) \in \mathrm{R}^{nr \times m}$ is a coefficient matrix of a bridge model at time t defined as Eq. (11), then Eq. (8) can be condensed as a matrix formation of Eq. (12).

$$\mathbf{x} = \{\mu_1; \quad \mu_2; \quad \cdots \quad \mu_{m-1}; \quad \mu_m\} \tag{10}$$

$$\mathbf{H}(t) = \{\mathbf{h}_1(t) \quad \mathbf{h}_2(t) \quad \cdots \quad \mathbf{h}_{m-1}(t) \quad \mathbf{h}_m(t)\} \tag{11}$$

$$\mathbf{H}(t)\mathbf{x} = \mathbf{f}(t) \tag{12}$$

If a moving vehicle experiment is carried out and measured data of *mt* samples are available in damage identification, then Eq. (12) can be written simply as

$$\mathbf{A}\mathbf{x} = \mathbf{b}\,, \tag{13}$$

where, $\mathbf{A} = [\mathbf{H}(0);\cdots;\mathbf{H}(mt)]$, and $\mathbf{b} = [\mathbf{f}(0);\cdots;\mathbf{f}(mt)]$.

Equation (13), which shows the formulation in the form of a linear system of equations subtracted from equations of motion for the bridge-vehicle interaction, is used for damage identification of bridges. Usually, inverse problems are often ill-posed. The Tikhonov regularization (Tikhonov and Arsenin, 1977, Hansen (1994)) as a RLS minimization is used to solve Eq.(13).

The health condition of bridge structures is detectable directly from investigating the ESI vector **x**. Determination of the ESI vector can also provide information relating to damage location and severity.

3 MOVING VEHICLE LABORATORY EXPERIMENT

3.1 Experiment setup

Feasibility of the method for damage identification is investigated through a moving vehicle laboratory experiment. A scale bridge model is adopted as an experimental girder. The experiment setup is shown in Figure 1 with geometry of the girder model and a scale roadway surface profile. Details of the structural properties of the girder are also presented in Figure 1.

Two damage scenarios are considered: scenario 1 with damaged section I which has three saw cuts at both left and right sides of web plates between L/4 and L/2 (ELEM. No.2 in Figure 1) of the bridge; and scenario 2 considering both damaged section I and damaged section II. The damaged section II cuts out a part of the web plate between 3L/4 and L (ELEM. No.4 in Figure 1) of the bridge. For damaged section I, the bending rigidity of the member decreases to 89% of the intact state. 23% decrease of bending rigidity of the damaged member is obtained due to the damaged section II.

Three different vehicle models are considered during the experiment. The natural frequencies of the vehicle model used in the experiment are respectively 2.93 Hz, 3.03Hz and 3.76 Hz for the bounce motion. Two different speeds of 0.93 m/s and 1.63 m/s are also adopted to investigate the effect of the vehicle speed to damage identification accuracy. The six loading scenarios are considered as

SCN1: Vehicle of 2.93Hz with speed of 0.93m/s;

SCN2: Vehicle of 2.93Hz with speed of 1.63m/s;

SCN3: Vehicle of 3.76Hz with speed of 0.93m/s;

SCN4: Vehicle of 3.76Hz with speed of 1.63m/s;

SCN5: Vehicle of 3.03Hz with speed of 0.93m/s; and

SCN6: Vehicle of 3.03Hz with speed of 1.63m/s.

Three points of 1/4, 1/2, and 3/4 of the span length and two points of front and rear axles of the vehicle are observation points. The sampling rate is 100 Hz.

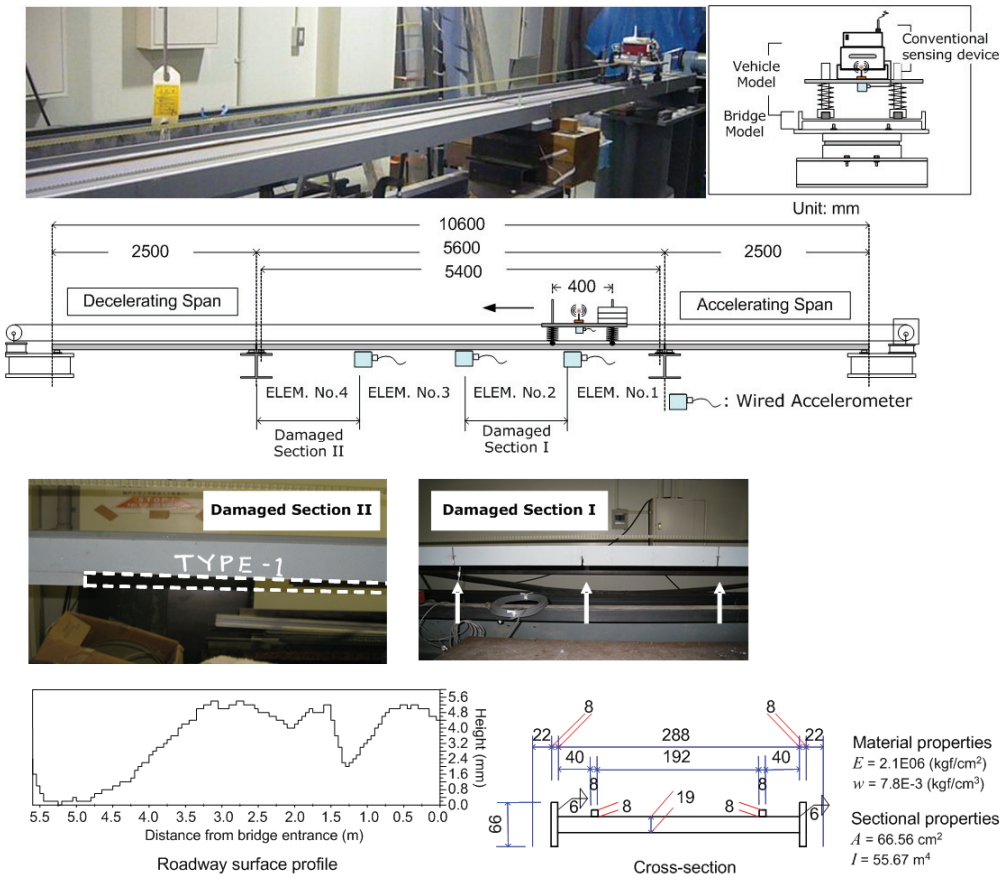

Figure 1. Experiment setup

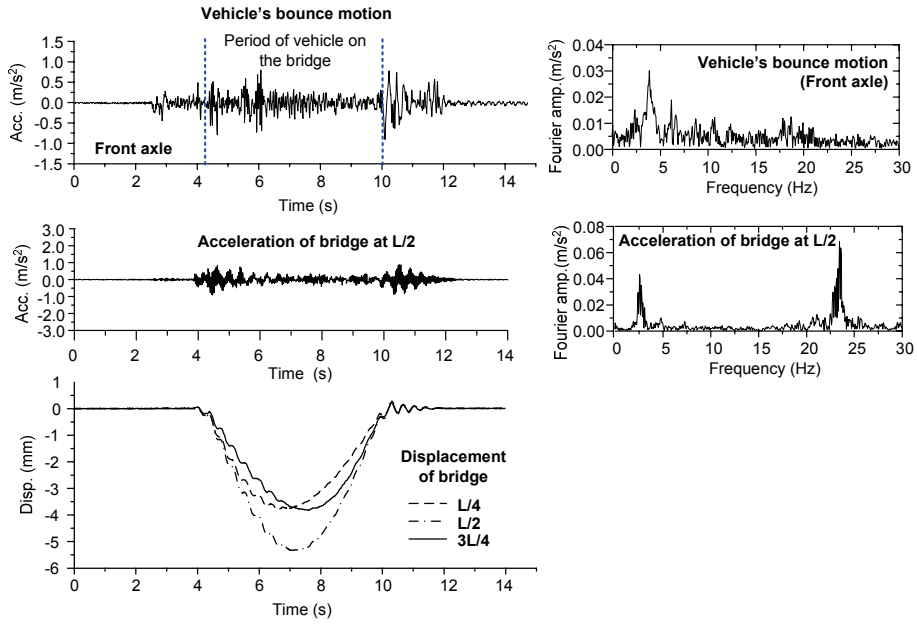

Figure 2. Dynamic responses of intact bridge and vehicle under loading scenario 1 (SCN1).

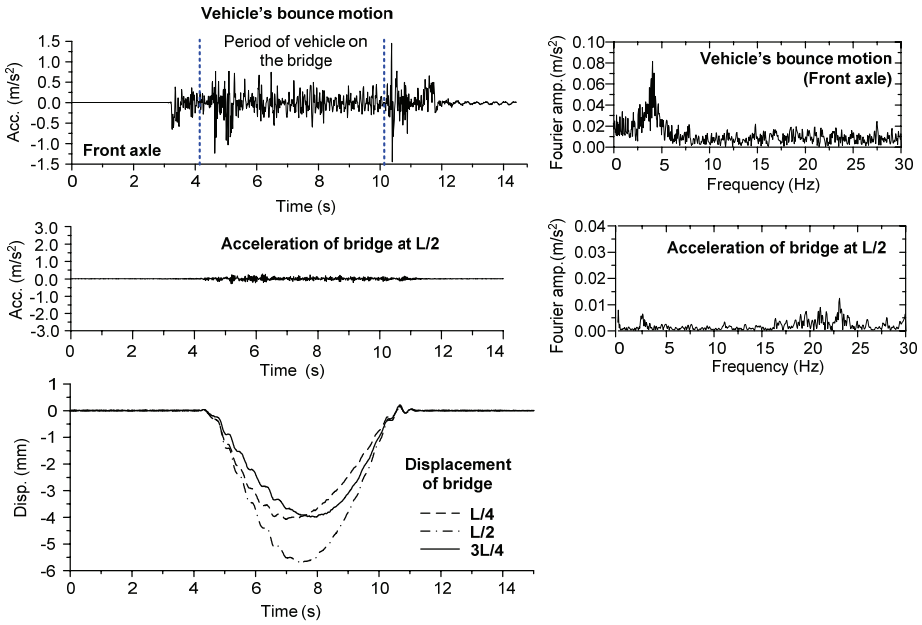

Figure 3. Dynamic responses of bridge and vehicle under damage scenario 1 and loading scenario 1 (SCN1).

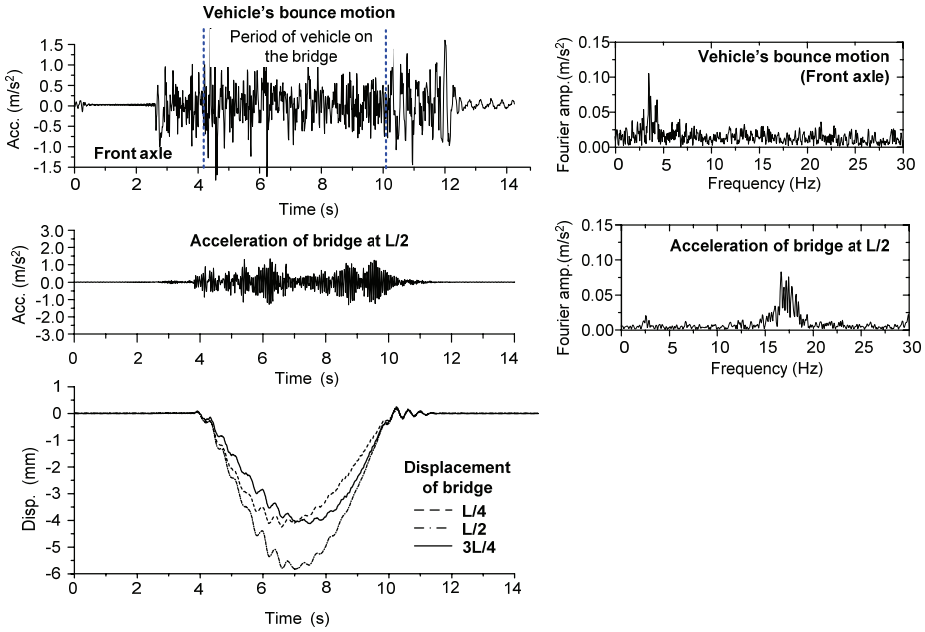

Figure 4. Dynamic responses of bridge and vehicle under damage scenario 2 and loading scenario 1 (SCN1).

3.2 Dynamic responses

Examples of time histories of acceleration at the span center of the bridge and displacements at each observation point as well as the acceleration response at the front axle of the vehicle are shown in Figures 2, 3 and 4 according to damage scenarios. They also give Fourier amplitude spectra before and after damage. They show that the amplitude of acceleration responses drastically decreases due to the damage scenario 1. An interesting point is that dominant

frequencies near 2.5Hz and 23.4Hz are weakened by the damage. For vehicle responses, the acceleration amplitude of the vehicle running on the damaged bridge is greater than that of running on the intact girder. Considering damage scenario 2 which assumes both damaged sections I and II, on the other hand, the acceleration response of the bridge as well as the vehicle tend to increase.

3.3 Damage identification

Damage identification results are summarized in Figures 5 and 6, respectively for damage scenarios 1 and 2. For damage scenario 1, the identification result demonstrates that the damage location at element No.2 is well identified by the proposed method except the loading scenario 3 (SCN3). The error of identified severity of the damage varies up to 5%, and it demonstrates the proposed method can presume the damage severity. For damage scenario 2, suspected damage locations are also well identified. The damage severity of each member is identified within the error of 8% for the damaged section I (ELEM. No.2 in Figure 1) and within the error of 10% for the damage section II (ELEM. No.4 in Figure 1).

Observations from the experimental investigation demonstrate that locations and severities of damage are constantly identified without great variation according to vehicle type and speed.

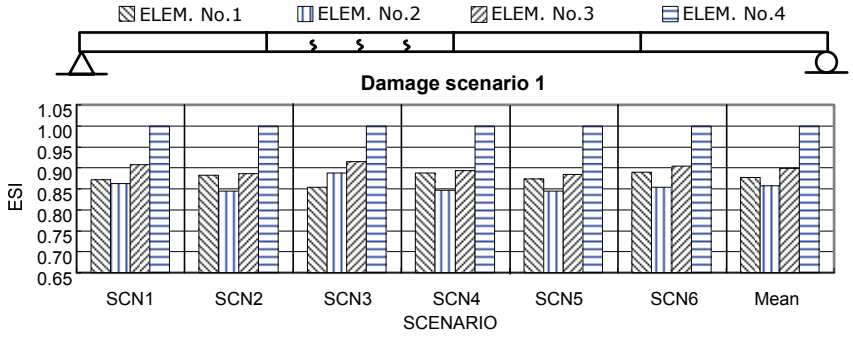

Figure 5. Identified damage location and severity of the bridge under damage scenario 1 with damage at ELEM. No.2.

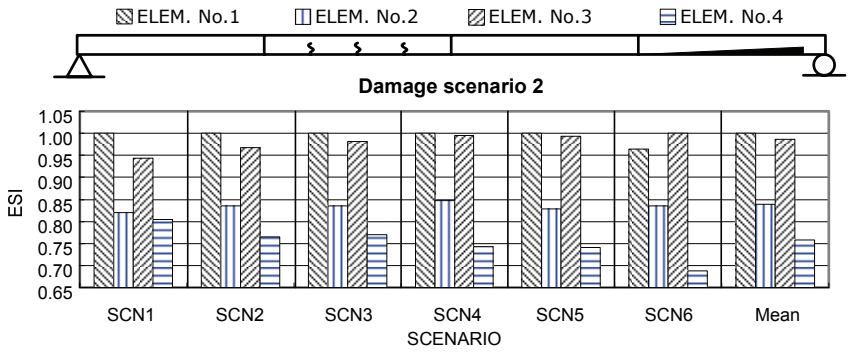

Figure 6. Identified damage location and severity of the bridge with damages at ELEM. No.2 and ELEM.4.

4 CONCLUSIONS

This paper presents a moving vehicle laboratory experiment of a bridge model to investigate feasibility of bridge damage identification using traffic-induced vibration data, which is especially focused on condition assessment of short and medium span bridges because of their great contribution to entire bridge structures. The element stiffness index (ESI), which defines the normalized changes in the element stiffness, is adopted as an indicator of damage.

The experimental study demonstrates that damage location and severity are well detectable using the proposed method. Observations from the experimental investigation also show that locations and severities of damage are constantly identified without great variation according to vehicle type and speed.

The most important and fundamental result obtained through this study is the feasibility of the method for addressing real-world problems. Feasibility investigations for actual bridges will be a strong motivation for the next challenge.

5 ACKNOWLEDGEMENTS

A part of this study is supported by the Japan Society for the Promotion of Science (Grant-in-Aid for Exploratory Research under project No.19656112 and Grant-in-Aid for Scientific Research (C) under project No. 20560443), which is gratefully acknowledged.

6 REFERENCES

Hansen PC. 1994. Regularization tools, A Matlab package for analysis and solution of discrete ill-posed problems. *Numerical Algorithms*, 6: 1-35.

Kim CW, Kawatani M and Kim KB. 2005. Three-dimensional dynamic analysis for bridge-vehicle interaction with roadway roughness. *Computer and Structures*, 83: 1627-1645.

Kim CW and Kawatani M. 2008. Pseudo-static approach for damage identification based on coupling vibration with a moving vehicle. *Structure and Infrastructure Engineering*, 4(5): 371-379.

Rizos PF, Aspragatos N and Dimarogonas AD. 1990. Identification of crack location and magnitude in a cantilever beam from the vibration modes. *J Sound Vibr*, 138: 381-388.

Salawu OS. 1997. Detection of structural damage through changes in frequency: a review. *Eng Struct*, 19: 718-723.

Shifrin EI and Ruotolo R. 1999. Natural frequencies of a beam with an arbitrary number of cracks. *J Sound Vibr*, 222: 409-423.

Siringoringo DM and Fujino Y. 2006. Observed dynamic performance of the Yokohama Bay Bridge from system identification using seismic records. *Structural Control and Health Monitoring*, 13: 226-244.

Tikhonov AV and Arsenin VY. 1977. *Solutions of Ill-posed Problems*, New York: Wiley.

Wenzel H and Pichler D. 2005. *Ambient Vibration Monitoring*, New York: John Wiley & Sons.