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A STRUCTURAL DAMAGE DETECTION PROPOSAL USING TRAFFIC-INDUCED BRIDGE VIBRATION ANALYSIS AND GA OPTIMIZATION

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ABSTRACT

This research is intended to establish a bridge damage detection approach employing only direct analyses of train-induced bridge vibration by means of introducing soft computing methods. In this approach, different from identifying structural damages using inverse analyses, the possible damage patterns of the bridge are assumed at first. Then, the running train-induced bridge vibration under a certain damage pattern is calculated employing a developed analysis procedure. When the calculated response is identical to the recorded one, this damage pattern will be the solution. However, owing to the large number of damage patterns, it is difficult to identify the exact solution. Therefore in this approach, the optimization method of Genetic Algorithm is applied to identify the damage pattern including the damage locations and degrees, in which the difference between the calculated results and the recorded responses is defined as the object function. The basic concepts and feasibility investigations of the proposed approach are introduced in this paper.

Keywords: Damage Detection, Train-Bridge Interaction Analysis, Soft Computing, Health Monitoring

1. INTRODUCTION

In the maintenance scheme of railway viaducts nowadays in Japan, effective health monitoring and diagnosis processes become essential because of the huge number of the structures. Currently, the overall health condition of the Shinkansen viaducts is mainly examined by visual inspections, which demand a large number of technicians and also a considerable cost. It is already reported (Doebling et al. 1998; Alvandi and Cremona 2006) that the dynamic characteristics are possible to be used to identify the structural conditions. In fact, impact tests have been adopted to investigate the integrity of the bridge structure in Shinkansen system since 1991. However, the impact tests not only demand enormous manpower and cost, but also have the deficiency that it cannot be carried

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out during the operational hours. If the train-induced vibration data can be effectively used for the health monitoring process, it will be an economical and convenient way.

Some researches on structural identification and bridge health monitoring using traffic-induced vibration data have been initiated recently, which mostly need inverse analysis (Kim and Kawatani 2008) to identify the structural damage. The numerical errors caused by the inverse analysis can bring considerable difficulties in practical applications with the increasing structural members. In this study, a structural identification approach using train-induced vibration data of the Shinkansen viaducts is developed, employing only direct analyses by means of introducing soft computing methods. In this approach, different from identifying the structural damages using inverse analyses, the possible damage patterns of the bridge are assumed according to theoretical and empirical considerations at first. Then, the running train-induced dynamic responses of the bridge under a certain damage pattern are calculated employing a train-bridge interaction analysis procedure established by the authors. If the calculated responses are identical to the recorded ones, this damage pattern will be the solution. However, owing to the large number of damage patterns, it will demand enormous computational work to identify the exact solution. Therefore in this approach, an effective optimization method is applied to identify the damage pattern including the damage locations and degrees. In this paper, the basic concept and feasibility investigations of this approach are represented using real train and simple bridge models.

2. CONCEPTS OF THE PROPOSED DAMAGE IDENTIFICATION APPROACH

To identify the structural characteristics of the bridge with traffic-induced vibration data, a currently conventional way is to perform the inverse analysis of vehicle-bridge interaction (Kim and Kawatani 2008). However, such an approach can encounter considerable difficulties in actual applications because of numerical errors caused by the inverse analysis due to the large number of members. In recent years, the applications to structural identification of soft computing methods including Genetic Algorithm (GA) (Perry et al. 2006; Koh and Perry 2009) and Neural Networks (NN) (Yun and Bahng 2000) are indicating remarkable progress. Therefore in this research, a structural identification approach is developed employing only direct analyses of train-induced bridge vibration by means of introducing soft computing methods to avoid the numerical error problems encountered in the inverse analysis.

In the actual railway viaducts, the possible damage patterns of the structures are comprehended by the bridge engineers based on theoretical and empirical facts. Therefore in this approach, the damage patterns of the bridge members are assumed in advance and used as the input information. Then, the train-induced dynamic responses of the bridge under a certain damage pattern are calculated by a developed analytical procedure. In the assumed damage patterns, the one identical or nearest to the actual damage condition will give the most similar dynamic responses to the recorded ones, through which the exact solution can be identified. To make this approach applicable to actual structures with enormous possible damage patterns, the soft computing methods of NN and GA are introduced and applied as follows.

a) In this proposed approach, the traffic-induced bridge responses used to compare with measured ones will be simulated by a developed vehicle-bridge interaction analysis computer program. However, for a large-scale structure even one time of such an analysis will demand considerable computational capacities, which will leads to the infeasibility in an actual identification process that needs a great number of interaction analyses. Therefore in this research, the NN techniques (Kartam et al. 1997) are planned to be used to simulate the running train-induced bridge response, which can shorten the computational time to an acceptable degree in actual applications. To establish such a NN tool, it is impossible to use measured results to carry out the supervised learning process, thus the results from the train-bridge interaction analysis program have to be used as the sample data. Not to mention, adequate accuracy for such an analytical program is indispensable in the actual applications. In this paper, owing to usage of a simple bridge model that demand only small computational capacities, the establishment of the NN tool is ignored.

b) The calculated train-induced bridge responses under certain damage patterns described above are then used for the identification process. However, even only the possible damage patterns based on engineering facts are assumed, the number can still be considerable large and difficult to identify. This is a typical combinatorial optimization problem and can be solved by some metaheuristic search algorithms. In this research, the GA technique (Goldberg 1989) is adopted to find the exact damage pattern. In the GA algorithm, the damage patterns are set as the population and the difference between the calculated results and the recorded ones is defined as the object function. It is obvious that the proper definition of the object function can be a determinative factor for the identification results in actual applications.

3. SIMPLE GIRDER BRIDGE AND 15-DOF TRAIN MODELS

In this paper, a real train and a simple bridge models are used to examine the feasibility of the proposed approach. Figure 1 shows a train car modeled as 3D sprung-mass system. The variants employed in the car model are shown in Table 1. The definition of dimensions of the car is shown in Table 2. The notations of the train properties are indicated in Table 3. One car of the 15-DOF train model formulized above is used for the analysis. The velocity of the train is assumed as 60 km/h. The rail surface roughness is considered and measured data are used.

A typical simply-supported steel girder bridge in Shinkansen lines, as shown in Figure 2, is adopted for the feasibility evaluation of the proposed approach. The properties of the bridge are also indicated in the figure. In general, such a steel girder bridge has the fundamental frequency as about $175/l$ based on the field investigation (Matsuura 1976). In this analysis, as shown in Figure 2, the girder model is uniformly dived into 10 beam elements.

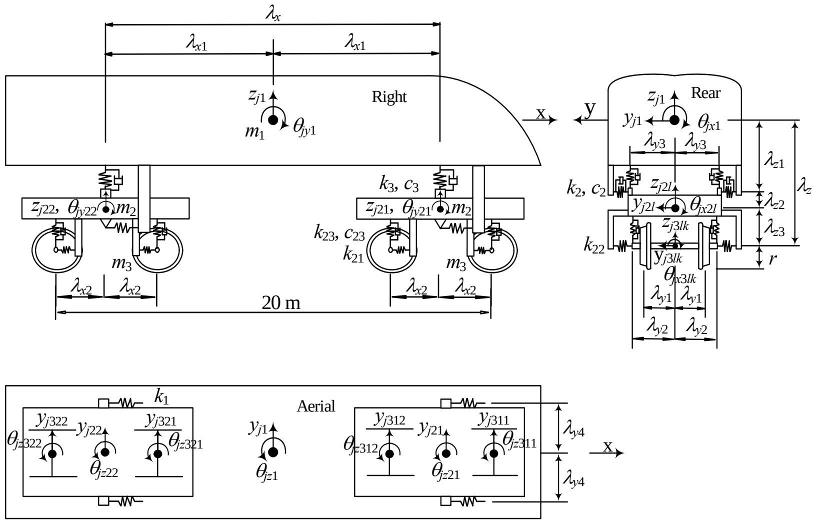

Figure 1: Sprung-mass dynamic train car 3D model

Table 1: Variants employed in train model

Definition (j^{th} car)	Notation
Lateral translation of car body	y_{j1}
Sway of front bogie	y_{j21}
Sway of rear bogie	y_{j22}
Bouncing of car body	z_{j1}
Parallel hop of front bogie	z_{j21}
Parallel hop of rear bogie	z_{j22}
Rolling of car body	θ_{jx1}
Axle tramp of front bogie	θ_{jx21}
Axle tramp of rear bogie	θ_{jx22}
Pitching of car body	θ_{jy1}
Windup of front bogie	θ_{jy21}
Windup of rear bogie	θ_{jy22}
Yawing of car body	θ_{jz1}
Yawing of front bogie	θ_{jz21}
Yawing of rear bogie	θ_{jz22}

Table 2: Definition of dimensions of the train model

1/2 length of car body in x-direction	λ_c	12.50 m
Distance of centers of bogies in x-direction	λ_x	17.50 m
1/2 distance of centers of bogies in x-direction	λ_{x1}	8.75 m
1/2 distance of axes in x-direction	λ_{x2}	1.25 m
1/2 width of track gauge	λ_{y1}	0.70 m
1/2 distance of vertical lower springs in y-direction	λ_{y2}	1.00 m
1/2 distance of vertical upper springs in y-direction	λ_{y3}	1.23 m
1/2 distance of longitudinal upper springs in y-direction	λ_{y4}	1.42 m
Distance from centroid of body to axis in z-direction	λ_z	0.97 m
Distance from centroid of body to lateral upper spring in z-direction	λ_{z1}	0.50 m
Distance from centroid of bogie to lateral upper spring in z-direction	λ_{z2}	0.37 m
Distance from centroid of bogie to lateral lower spring in z-direction	λ_{z3}	0.10 m
Radius of wheel	r	0.43 m

Table 3: Properties of the train model

Definition	Notation	Value
Weight of car body	w_1	321.6 kN
Weight of bogie	w_2	25.9 kN
Weight of wheel	w_3	8.8 kN
Mass moment of inertia of car body	I_{x1}	49.2 kN·s2·m
	I_{y1}	2512.6 kN·s2·m
	I_{z1}	2512.6 kN·s2·m
Mass moment of inertia of bogie	I_{x2}	2.9 kN·s2·m
	I_{y2}	4.1 kN·s2·m
	I_{z2}	4.1 kN·s2·m
Spring constant	k_1	5000 kN/m
	k_2	176.4 kN/m
	k_3	443 kN/m
	k_{21}	17500 kN/m
	k_{22}	4704 kN/m
	k_{23}	1210 kN/m
Damping coefficient	c_2	39.2 kN·s/m
	c_3	21.6 kN·s/m
	c_{23}	19.6 kN·s /m

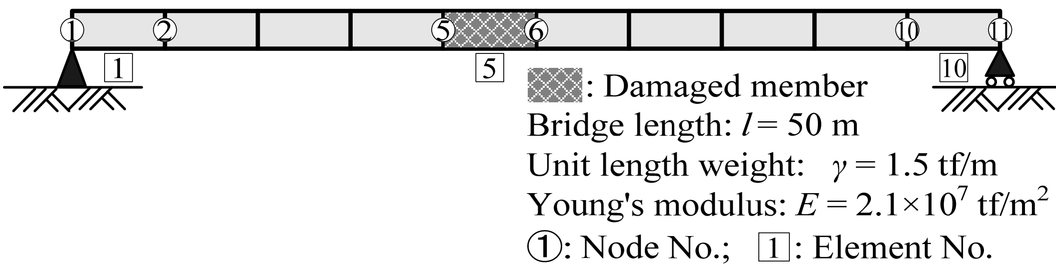

Figure 2: Simple girder bridge model

4. TRAIN-BRIDGE INTERACTION ANALYSIS PROCEDURE

Dynamic responses of the train-bridge interaction system are simulated using a developed computer program based on the formulization process described below. Modal analysis technique is applied to the simultaneous dynamic differential equations of the structure. The Newmark's β step-by-step numerical integration method is applied to solve the dynamic differential equations.

4.1. Formulization of the train body

The differential equations of the train motion can be obtained based on D'Alembert's Principle as follows.

$$m_1 \ddot{y}_{j1} - \sum_{l=1}^{2}\sum_{m=1}^{2}(-1)^m v_{jylm}(t) = 0 \tag{1}$$

$$m_1\ddot{z}_{j1} + \sum_{l=1}^{2}\sum_{m=1}^{2} v_{jzlm}(t) = 0 \tag{2}$$

$$I_{x1}\ddot{\theta}_{jx1} - \sum_{l=1}^{2}\sum_{m=1}^{2}(-1)^m \lambda_{y3} v_{jzlm}(t) - \sum_{l=1}^{2}\sum_{m=1}^{2}(-1)^m \lambda_{z1} v_{jylm}(t) = 0 \tag{3}$$

$$I_{y1}\ddot{\theta}_{jy1} + \sum_{l=1}^{2}\sum_{m=1}^{2}(-1)^l \lambda_{x1} v_{jzlm}(t) = 0 \tag{4}$$

$$I_{z1}\ddot{\theta}_{jz1} + \sum_{l=1}^{2}\sum_{m=1}^{2}(-1)^{l+m} \lambda_{x1} v_{jylm}(t) + \sum_{l=1}^{2}\sum_{m=1}^{2}(-1)^m \lambda_{y4} v_{jxlm}(t) = 0 \tag{5}$$

In the above equations, the subscript j is the sequence number of the car. The subscripts l and m are described as: l = 1, 2 respectively indicate the front and rear bogies; m = 1, 2 respectively indicate the right and left axles of each train body. $v_{jxlm}(t)$, $v_{jylm}(t)$ $v_{jzlm}(t)$ denotes the forces due to the expansion of the springs and dampers which corresponding directions.

4.2. Formulization of front and rear bogies

The equation of the front and rear bogie body's vibrations is denoted as follows.

$$m_2\ddot{y}_{j2l} + \sum_{m=1}^{2}(-1)^m v_{jylm}(t) - \sum_{k=1}^{2}\sum_{m=1}^{2}(-1)^m v_{jylkm}(t) = 0 \tag{6}$$

$$m_2\ddot{z}_{j2l} - \sum_{m=1}^{2} v_{jzlm}(t) + \sum_{k=1}^{2}\sum_{m=1}^{2} v_{jzlkm}(t) = 0 \tag{7}$$

$$I_{x2}\ddot{\theta}_{jx2l} - \sum_{m=1}^{2}(-1)^m \lambda_{z2} v_{jylm}(t) + \sum_{m=1}^{2}(-1)^m \lambda_{y3} v_{jzlm}(t) - \sum_{k=1}^{2}\sum_{m=1}^{2}(-1)^m \lambda_{z3} v_{jylkm}(t) - \sum_{k=1}^{2}\sum_{m=1}^{2}(-1)^m \lambda_{y2} v_{jzlkm}(t) = 0 \tag{8}$$

$$I_{y2}\ddot{\theta}_{jy2l} - \sum_{k=1}^{2}\sum_{m=1}^{2}(-1)^k \lambda_{z3} v_{jxlkm}(t) + \sum_{k=1}^{2}\sum_{m=1}^{2}(-1)^k \lambda_{x2} v_{jzlkm}(t) = 0 \tag{9}$$

$$I_{z2}\ddot{\theta}_{jz2l} - \sum_{m=1}^{2}(-1)^m \lambda_{y4} v_{jxlm}(t) + \sum_{k=1}^{2}\sum_{m=1}^{2}(-1)^{k+m} \lambda_{y2} v_{jxlkm}(t) + \sum_{k=1}^{2}\sum_{m=1}^{2}(-1)^{k+m} \lambda_{x2} v_{jylkm}(t) = 0 \tag{10}$$

In the above equations, the subscripts k and m are described as: k = 1, 2 respectively indicate the front and rear axle of rear bogies; m = 1, 2 respectively indicate the right and left axles of each bogie. $v_{jxlkm}(t)$, $v_{jylkm}(t)$, $v_{jzlkm}(t)$ denotes the forces due to the expansion of the springs and dampers which corresponding directions. w_{jylkm}, w_{jzlkm} denotes the dynamic displacement at the contact point of bogie wheel and railway.

4.3. Formulization of the bridge

The dynamic differential equations of the bridge can be derived as follows, based on FEM theories and D'Alembert's Principle.

$$\boldsymbol{M}_b\ddot{\boldsymbol{w}}_b + \boldsymbol{C}_b\dot{\boldsymbol{w}}_b + \boldsymbol{K}_b\boldsymbol{w}_b = \boldsymbol{F}_b \tag{11}$$

where, $\boldsymbol{M}_b$, $\boldsymbol{C}_b$, $\boldsymbol{K}_b$ and $\boldsymbol{w}_b$ respectively denote the bridge mass, damping, stiffness matrices and the nodal displacement vector. Herein, the damping matrix $\boldsymbol{C}_b$ is calculated by Rayleigh damping (Agabein 1971). The external force vector, $\boldsymbol{F}_b$, can be expressed as follows, where, $P_{jlk}(t)$ and $\boldsymbol{\Psi}_{jlk}(t)$ respectively denote the wheel load and the distribution vector, while h is the total car number.

$$\boldsymbol{F_b} = \sum_{j=1}^{h}\sum_{l=1}^{2}\sum_{k=1}^{2}\boldsymbol{\Psi}_{jlk}(t)P_{jlk}(t) \tag{12}$$

Applying the modal analysis technique to the bridge system, the structural displacement vector, $\boldsymbol{w}_b$, can be expressed as follows using eigenvectors $\boldsymbol{\varphi}_i$ and generalized coordinates q_i, where, subscript i is the mode number and n indicates the highest one to be considered.

$$\boldsymbol{w}_b = \sum_{i=1}^{n}\boldsymbol{\varphi}_i q_i = \boldsymbol{\Phi}\cdot\boldsymbol{q} \tag{13}$$

Substitute $\boldsymbol{w}_b$ into the bridge vibration equation will obtain the following equation.

$$\boldsymbol{M}_b\boldsymbol{\Phi}\ddot{\boldsymbol{q}}+\boldsymbol{C}_b\boldsymbol{\Phi}\dot{\boldsymbol{q}}+\boldsymbol{K}_b\boldsymbol{\Phi}\boldsymbol{q}=\boldsymbol{F}_b \tag{14}$$

Multiply $\boldsymbol{\Phi}^T$ to both sides,

$$\boldsymbol{\Phi}^T\boldsymbol{M}_b\boldsymbol{\Phi}\ddot{\boldsymbol{q}}+\boldsymbol{\Phi}^T\boldsymbol{C}_b\boldsymbol{\Phi}\dot{\boldsymbol{q}}+\boldsymbol{\Phi}^T\boldsymbol{K}_b\boldsymbol{\Phi}\boldsymbol{q}=\boldsymbol{\Phi}^T\boldsymbol{F}_b \tag{15}$$

According to the orthogonality of eigenvectors, and sssuming $\boldsymbol{\varphi}_i^T$ $\boldsymbol{F}_b=F_i$, the bridge equation corresponding to each mode can be expressed as follows by generalized coordinates.

$$M_i\ddot{q}_i + C_i\dot{q}_i + K_iq_i = F_i \tag{16}$$

Based on the above formula, the bridge-train coupled equations can be expressed as below in matrix form, where, $\boldsymbol{M}_q$, $\boldsymbol{C}_q$ and $\boldsymbol{K}_q$ respectively denote the equivalent mass, damping and stiffness matrices of the bridge taking into account the influence of the train, while $\boldsymbol{M}_{qv}$, $\boldsymbol{C}_{qv}$ and $\boldsymbol{K}_{qv}$ respectively indicate the coupled components of the train-bridge system. $\boldsymbol{F}_q^*$ and $\boldsymbol{F}_v^*$ compose the external force vector of the coupled system, which are obtained from $\boldsymbol{F}_q$ and $\boldsymbol{F}_v$ by removing the coupled components to the left side.

$$\begin{bmatrix}\mathbf{M}_q & \mathbf{M}_{qv}\\ Sym. & \mathbf{M}_v\end{bmatrix}\begin{Bmatrix}\ddot{\mathbf{q}}\\ \ddot{\mathbf{w}}_v\end{Bmatrix}+\begin{bmatrix}\mathbf{C}_q & \mathbf{C}_{qv}\\ Sym. & \mathbf{C}_v\end{bmatrix}\begin{Bmatrix}\dot{\mathbf{q}}\\ \dot{\mathbf{w}}_v\end{Bmatrix}+\begin{bmatrix}\mathbf{K}_q & \mathbf{K}_{qv}\\ Sym. & \mathbf{K}_v\end{bmatrix}\begin{Bmatrix}\mathbf{q}\\ \mathbf{w}_v\end{Bmatrix}=\begin{Bmatrix}\mathbf{F}_q^*\\ \mathbf{F}_v^*\end{Bmatrix} \tag{17}$$

5. GA OPTIMIZATION PROCEDURE

In this paper, the simple GA algorithm (Goldberg 1989, Koh and Perry 2009) is used for the identification process due to the simplicity of the structure. The flow-chart of GA is shown in Figure 3, in which the calibrated crossover rate is set as 60 percent, the mutation rate as 10 percent, and the number of initial population as 50. For the bridge structure, the damage degrees of members are treated as discrete values, and each value is encoded by a three binary digits gene string. The definition of the gene strings and the discrete values of damage degrees are given in Table 4, which can detect 0 to 70 percent damage of the structure with an interval of 10 percent. Then, an individual (chromosome) is composed of 10 gene strings corresponding to the 10 finite elements, which is used as the input data for the train-bridge interaction analysis program.

In this analysis, the object function (OBJ) is defined as the difference between the analytical acceleration response and the measurement data, as Equation (18). Where, $f(i)$ is the discrete values of the measurement data and $f^*(i)$ is the analytical results, respectively. Here, i and t respectively indicate the number and the total number of the time steps employed in the interaction analyses. In this paper, the simulated dynamic responses of these damage scenarios are assumed as pseudo-measurement data. Here, only the acceleration response of one node (Node 6) is used considering the simple structure. This means only one sensor is needed on the bridge to record the response. Of course, as mentioned above, for actual complex problems dynamic responses of multiple nodes as well as additional dynamic characteristics may be necessary.

$$OBJ = \frac{1}{t}\sum_{i=1}^{t}\{f(i) - f^*(i)\}^2 \tag{18}$$

The effectiveness of the GA algorithm depends greatly on the methods of crossover and mutation as well as the convergence condition, and calibration is needed for a certain problem. In this paper, two-point crossover is adopted, while the mutation is generated by random numbers. The criterion of convergence condition is determined as 10^{-5}.

Table 4: Definition of gene strings

Gene string	Damage degree (%)
000	0
001	10
010	20
011	30
100	40
101	50
110	60
111	70

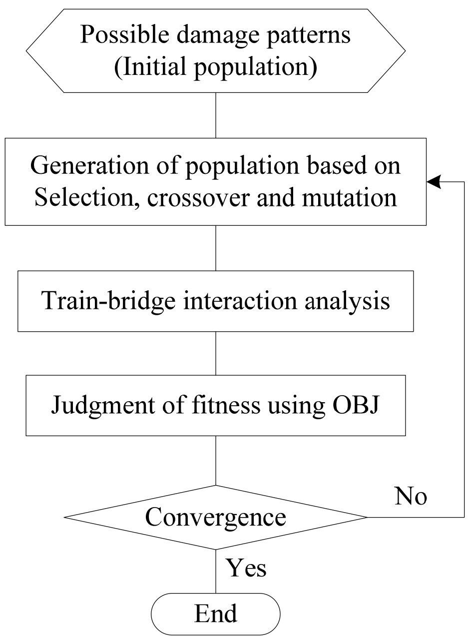

Figure 3: Flow-chart of GA algorithm

Table 5: Identification results by GA

Elem. No.	Case-1		Case-2		Case-3		Case-4		Case-5	
	M	A	M	A	M	A	M	A	M	A
1	0	0	0	0	0	0	0	0	0	0
2	0	0	0	0	0	0	0	0	0	0
3	0	0	0	0	0	0	0	20	0	20
4	0	0	0	0	0	0	0	0	0	0
5	30	30	30	30	30	30	30	0	30	0
6	0	0	0	0	0	0	0	0	0	10
7	0	0	0	0	0	0	0	10	0	0
8	0	0	0	0	0	0	0	0	0	0
9	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0	0
C.G	43		26		26		14		10	

6. DAMAGE SCENARIOS AND IDENTIFICATION RESULTS

To carry out the structural damage identification using GA algorithm, the following 5 cases of damage scenarios are assumed for investigation. In all the 5 cases, only Element No.5 is assumed to have a 30 percent damage expressed by a 30 percent decrease of its bending moment of inertia, while the measurement points are changed for each case. In the current development stage, the simulated dynamic responses of these damage scenarios are assumed as pseudo-measurement data.

For Case-1 to Case-5, the measurement points of each case are respectively set as Nodes No.6, No.2, No.9, No.1 and No.11. In structural identification problem, it is desirable to use a small number of sensors as possible due to not only the costs but also the limit of sensor locations. Thus in this analysis, only one of the Nodes is used as the measurement point. Then, Case-1 expresses the situation that the sensor location is near to the damaged member, while Case-2 and Case-3 indicates a far measurement point from the damage location. Case-4 and Case-5 suppose that the sensor is set on the end, i.e. the support, of the girder, which is the easiest way to install sensors.

The identification results based on the approach described so far are shown in Table 5 for all cases. In the table, M means pseudo-measurement value, A means Analysis result and C.G denotes Convergence Generation, respectively. In Case-1 to Case-3, the damage member is effectively and accurately identified by the developed approach. This result indicates that the damage can be identified even by using only one sensor and a relatively far sensor location, which can lead to considerable economical benefits. On the other hand, the identification process converged with wrong identification results in Case-4 and Case-5. The reason is considered as that the dynamic responses of Node No.1 and Node No.11 which are located on the supports of the bridge are relatively complicated compared with other nodes. For such cases, more sensitive object function is needed to carry out the identification properly.

7. CONCLUSIONS

In this paper, using train-induced vibration response of the bridge, a damage detection approach taking advantage of only direct structural analysis via introducing soft computing methods is proposed and preliminarily established. The basic concept and process of the approach are represented. A train-bridge interaction analysis procedure employing a real bullet train model is formulized and coded to simulate the structural dynamic responses. Then, the feasibility of this approach is examined based on the analytical results using simple structural and train models. This study laid a foundation toward developing a practical structural identification approach to actual bridge structures, although more detailed models and critical discussions are necessary.

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