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〈논 문〉

Instrumentation and Structural Health Monitoring of Bridges
교량구조물의 헬스모니터링을 위한 진동계측

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ABSTRACT

As bridge design is advancing toward the performance-based design, it becomes increasingly important to monitor and re-evaluate the long-term structural performance of bridges. Such information is essential in developing performance criteria for design. In this research, sensor systems for long-term structural performance monitoring have been installed on two highway bridges. Preliminary vibration measurement and data analysis have been performed on these instrumented bridges. On one bridge, ambient vibration data have been collected, based on which natural frequencies and mode shapes have been extracted using various methods and compared with those obtained by the preliminary finite element analysis. On the other bridge, braking and bumping vibration tests have been carried out using a water truck in addition to ambient vibration tests. Natural frequencies and mode shapes have been derived and the results by the breaking and bumping vibration tests have been compared. For the development of a three dimensional baseline finite element model, the new methodology using a neural network is proposed. The proposed one have been verified and applied to develop the baseline model of the bridge.

1. Introduction

Baseline model development of a civil infrastructure is essential for structural health monitoring which can play an important role in securing system integrity, minimizing maintenance cost, and maintaining longevity of highway bridges. Structural health monitoring and baseline model updating have a close correlation and are required periodically, especially after damaging earthquakes because of the degradation of a structure due to aging or environmental loads. Since structural parameters are affected by the environmental condition such as temperature and exciting load level, the establishment of a database and a statistical approach through long-term structural health monitoring are also in demand.[1~3]

While there are many technologies involved in the structural health monitoring, they can be

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broadly categorized as local or global approaches. The first category is designed to provide information about a relatively small region of a structure by utilizing local measurements.[4] The second category uses measurements from a dispersed set of sensors to obtain global information about the condition of the structural system.[5,6] The two approaches are complementary to each other, with the optimum choice of approach highly dependent on the scope of the problem at hand and the nature of sensor and structural system.

Global structural health monitoring technology consists of two aspects: (1) instrumentation of bridges with sensors such as accelerometers and strain gauges and more importantly, (2) methodologies for obtaining meaningful information concerning the structural health conditions, if any, from the measured data. This paper is focused on the baseline model development of highway bridges, which are instrumented for global structural health monitoring. Monitoring systems including accelerometers, strain gauges, pressure sensors, and displacement sensors have been installed on two highway bridges in Orange County, California, USA: the Jamboree Road Overcrossing and the West Street On-Ramp. Both are three-span concrete box-girder bridges. And baseline model has been developed using the vibration data obtained by preliminary vibration tests.

2. Instrumentation of Two Bridges

2.1 Description of Bridges

The Jamboree Road Overcrossing shown in Fig. 1 is a typical three-span continuous cast-in-place prestressed post-tension box-girder bridge. The total length of the bridge is 110.9 m, in which the lengths of spans are 35.5, 46.1, and 30.3 m. The bridge is supported on two monolithic single columns and sliding bearings on both abutments. The sliding bearings allow creep, shrinkage, and thermal expansion or contraction. It is noted that the columns are not on the centerline of the super-structure as shown and the bridge has a skewed abutment 4. The Eastern Transportation Corridor is one of the three design/build/operate toll roads in southern California. All of the bridges in the Corridor are designed based on the two-level seismic performance design criteria, which greatly influence the structural proportion between the super- and sub-structures. In general, the sub-structures become stiffer and larger than those designed by the current Caltrans criteria.[7,8] The consequence of this change has not been evaluated yet. It will be of great interest to monitor and evaluate the long-term structural performance of such bridges under not only seismic but also service loads, and to compare their performance with that of the bridges designed by the current Caltrans approach. During the proposal stage of this project, the Jamboree Road Overcrossing was one of a few bridges that were scheduled for completion of construction at the end of the proposed project, thus providing excellent opportunity for embedding strain sensors in concrete and a pressure sensor in the abutment during the construction. However, the corridor construction was finished before the funding was approved and contract executed by Caltrans. As a result, after the completion of construction, only accelerometers and a displacement sensor were installed on the Jamboree Road Overcrossing.

The West Street On-Ramp shown in Fig. 2 is a three-span continuous and curved cast-in-place prestressed post-tension box-girder bridge. The total length of the bridge is 151.3 m, in which the lengths of spans are 45.8, 60.1, and 45.4 m. The bridge is supported by two fixed columns and sliding bearings on both abutments. The bridge is curved highly with a 12 % super-elevation.

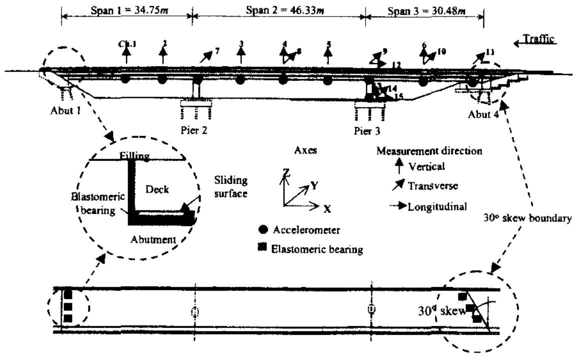


Fig. 1 Jamboree road overcrossing and accelerometer arrangement

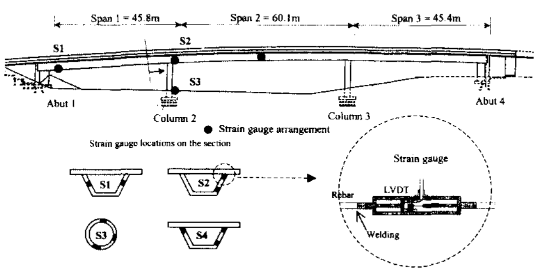


Fig. 2 Strain gauge arrangement at west street on-ramp

2.2 Monitoring System

Monitoring systems involve not only accelerometers, but also strain gauges. Strain measurement as well as acceleration for long-term structural performance monitoring is supposed to provide verifications of current design/analysis and suggest their improvement for retrofit and future construction. The sensors were installed at both super- and sub-structures. Instrumentation at sub-structures is important to measure the responses of sub-structures as well as triggering ground motion during earthquakes, since the columns turned out to be one of the most vulnerable structural components of highway bridges under damaging earthquakes, California in particular.[3]

Uni-axial, bi-axial, and tri-axial force-balance servo-type accelerometers were permanently installed on both bridges. The frequency response functions of the accelerometers indicate that the accelerometers can measure dynamic signals in a frequency range from 0.5 Hz through 30 Hz with very good accuracy. The locations of accelerometers at Jamboree Road Overcrossing are shown in Fig. 1, and the accelerometers are placed along the centerline of the bottom of the girder to minimize the torsional effect of the bridge box girder. Strain gauges, which are micro-displacement sensors, were permanently embedded in concrete members of the West Street On-Ramp. The strain gauges were completely welded to dummy reinforcing bars, and were embedded to measure the dynamic strains induced by bending moments. The locations of strain gauges are shown in Fig. 2. In order to measure the displacement of the super-structure with respect to the abutment due to shortening, creep, shrinkage, as well as seismic excitations, a displacement sensor was installed at an abutment for each bridge. To measure the soil pressures at the abutment during earthquakes, a soil pressure sensor was installed on the back wall of the abutment at the West Street On-Ramp. Data recorders were installed at both bridges. They can be triggered either manually or by earthquake ground motion. The tri-axial accelerometer at the bottom of column 3 was set up for triggering, and the triggering acceleration for each direction is 0.002 g. In the West Street On-Ramp, the factory voltage of electric power around the bridges is transformed into domestic voltage by a transformer, and then the electrical power is used for the monitoring system. Given the complexity of today's networked systems there is a high probability that power failure caused by earthquakes may result in data corruption or equipment failure. The uninterruptible power supply (UPS) system can provide excellent protection against most power disruptions, even for those caused by lightning. An UPS unit was installed at the West Street On-Ramp, which can provide power backup for 10 minutes. In the

Jamboree Road Overcrossing, no regular power electrical power supply is available. For this reason, a solar power supply system was installed at the bridge site. Batteries are charged by the solar panels during daytime and provide power to the monitoring system at nights.

3. Data Processing Methods

Processing of vibration data measured at a structure is usually associated with the extraction of modal parameters including natural frequencies and mode shapes. The loads are normally unknown, and thus the modal parameter extraction has to be based on the responses only. Several methods for extracting modal parameters without requiring information about input loads have been studied. The peak picking (PP), the random decrement (RD), and the frequency domain decomposition (FDD) methods are most widely used methods based on frequency domain analysis. And the Ibrahim time domain (ITD) and the eigen-system realization algorithm (ERA) methods are based on time domain analysis.[22,23] The major advantage of the frequency domain-based methods compared to the time domain-based ones is their user-friendliness. They are fast, simple to use, and give the user a feeling of the data he or she is dealing with. On the other hand, the most important advantage of the time domain-based method is that they can consider the nonlinear responses of structures. In this study, the frequency domain-based methods have been used to derive modal parameters of measurements.

The simplest of modal parameter extraction methods is the peak *peaking method* which has been used for a long time and is sometimes referred to as peak amplitude method.[12,13] This is a method which works adequately for structures whose frequency response functions (FRF) exhibit well-separated modes. It is not suitable for so lightly-damped structures because accurate measurements around resonance are difficult to obtain, and is not suitable for so heavily-damped structures either because the measurements around resonance are strongly influenced by more than one mode. Although all these appear to limit the application of the method, it should be noted that it is highly useful in obtaining initial estimates of the required parameters, thereby speeding up the analysis using more general methods such as the curve fitting method, the random decrement method, and the frequency domain decomposition method. It must be noted that the estimates of the modal parameters using the peak picking method depend heavily on the accuracy of the peak value which, unfortunately, are difficult to measure with good accuracy. Most of the errors in measurements are concentrated around the resonance region and special attention is needed for lightly-damped structures whose peak values may rely entirely on the validity of a single data point in the FRF spectrum. Errors may also arise because the modes are not well separated. Even with clearly separated modes, it is often found that the neighboring modes do contribute a noticeable amount to the total response at the resonance of the mode being analyzed.

In this study, the random decrement (RD) technique,[17,18] sometimes called *Randomdec*, is used to reduce the effect of vibration noises on the estimation of modal parameters from the vibration measurement data. The fundamental concept of the random decrement technique is based on the fact that the random response of a structure is composed of a deterministic part and a random part under white noise excitation. By averaging sufficient sample responses, the random part associated with the random excitation will average out, leaving the deterministic part. It has been shown that the remaining deterministic part is a *free decay response*, from which the modal parameters and the damping characteristics can be easily extracted. The random response is decomposed into segments in a way such that each segment starts with the same initial or triggering conditions. The randomdec signature is defined as the average of these segments.

The peak peaking (PP) method using power spectral density (PSD) is widely used in practice. The PP method gives reasonable estimates of natural frequencies and mode shapes if the modes are well separated. Otherwise, it is very difficult or impossible to use this method. The *frequency domain decomposition* (FDD)[14~16] method, which utilizes the singular value decomposition of the PSD matrix and was originally developed for extracting operational deflection shapes in mechanical vibrating systems, may be used to separate close modes, thus obtaining better estimates. The FDD technique is an extension of the PP technique. The PP approach is based on simple signal processing using the Discrete Fourier Transform (DFT), and is using the fact that well separated modes can be estimated directly from the FRF matrix at peaks. In the FDD technique, the PSD matrix is first formed from the measured data using a simple signal processing by DFT. However, instead of using the PSD matrix directly like FRF in the PP approach, the PSD matrix is decomposed at each frequency line using the Singular Value Decomposition (SVD). By doing so, the PSD matrix is decomposed into a set of auto spectral density functions, each corresponding to a single degree of freedom (SDOF) system. This is exactly true in the case where the loading is white noise, the structure is lightly damped, and the mode shapes of close modes are geometrically orthogonal. If these assumptions are not satisfied, the decomposition into SDOF systems becomes an approximation, but still the results are significantly more accurate than those of the classical approaches. The singular vectors in the SVD are used as estimates of the mode shape vectors, and the natural frequencies are estimated by taking each individual SDOF auto spectral density function back to the time domain by inverse DFT. The frequency and damping are simply estimated from the crossing times and the logarithmic decrement of the corresponding SDOF auto correlation function.

4. Vibration Tests

4.1 Ambient Vibration Test

When dynamic properties of large civil engineering structures such as bridges and buildings are estimated, possibilities to control and measure the loading on structures are rather limited. For a bridge which carries traffic 24 hours a day, the traffic-induced ambient vibration is a dominating signal. That is one of the reasons that ambient vibration tests have gained an increasing interest and popularity during the recent years as a most convenient tool for monitoring large civil engineering structures.[9~11] In comparison to forced vibration techniques, the ambient vibration techniques present the remarkable advantage of not requiring heavy and expensive equipments for exciting the structure. The dynamic properties of the structure can be rather accurately estimated on the basis of the measurements of structural ambient responses. Dynamic property extraction using ambient vibration measurements requires special data processing techniques to deal with relatively small amplitudes of ambient vibrations contaminated with noises without the knowledge of input forces. In this study, the peak picking, the random decrement, and the frequency domain decomposition methods have been used for processing ambient vibration data measured at the Jamboree Road Overcrossing.

The accelerations at the various locations on the bottoms of the box-girder and a column were measured with a sampling frequency of 100 Hz under normal traffic condition. The maximum continuous recording duration is 70 minutes with 1-minute recess every 10 minutes. The typical

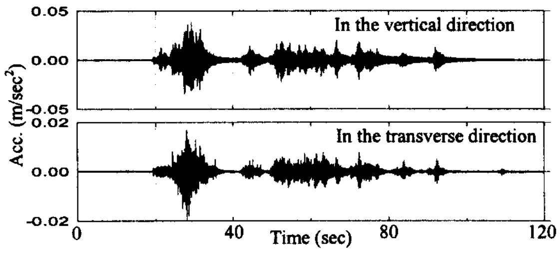


Fig. 3 Typical accelerations in the middle of span 2

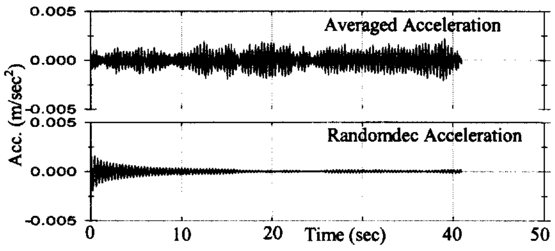


Fig. 4 Signal processed accelerations in the middle of span 2

time histories of accelerations of the super-structure measured in the middle of span 2 in the vertical and in the transverse directions are shown in Fig. 3. A vehicle passes from abutment 4 to abutment 1 with an average vehicle speed of 84.3 km/h(50 mph), and the average time that it takes for a vehicle to completely pass through the bridge is 4.93 sec. The amplitudes of accelerations in the transverse direction are relatively smaller than those in the vertical direction because of the traffic loads mainly induce vibrations in the vertical direction.

Totally, 82 data sets measured in the daytime were processed, in which the time length of each data set is 10 minutes. The peak picking (PP)[12,13] and the frequency domain decomposition (FDD)[14~16] methods were used to extract modal parameters from the averaged vibration data with 25 percent overlapping. The random decrement (RD)[17,18] method was also applied to obtain the parameters from the randomdec vibration data which was derived a prescribed initial condition of positive zero crossing. The total number of sampling data in the averaged and the randomdec data is 4096 (= 40.96 sec). Fig. 4 shows the time histories of the averaged and the randomdec accelerations in the middle of span 2. Randomdec acceleration exhibits its free vibration decay since the data sets have been sampled with the approximately same initial conditions compared to those in the averaged acceleration.

The natural frequencies (five in the vertical and three in the transverse directions) derived analytically and experimentally are shown together

Table 1 Comparison of natural frequencies of jamboree road overcrossing (Hz)

Mode Number	In-plane mode				Out-of-plane mode		
	FE	PP	RD	FDD	FE	PP	FDD
1	2.889	2.954 (0.065)	2.954 (0.065)	2.954 (0.065)	3.126	2.612 (0.514)	2.612 (0.514)
2	3.716	3.979 (0.263)	4.077 (0.361)	3.979 (0.263)	4.535	5.004 (0.469)	5.004 (0.469)
3	4.654	4.660 (0.006)	4.638 (0.016)	4.638 (0.016)	10.509	9.940 (0.569)	9.936 (0.573)
4	5.721	6.250 (0.529)	6.323 (0.602)	6.250 (0.529)			
5	9.400	8.957 (0.443)	8.813 (0.587)	8.911 (0.489)			
6	14.521	12.793(1.728)	12.866(1.655)	12.793(1.728)			

Note: The values in the parentheses are the absolute values of the difference between the analytical and the experimental natural frequencies.

FE = finite element analysis

PP = peak picking method

RD = random decrement method

FDD = frequency decomposition method

in Table 1. In which, in-plane modes are the modes of which displacement fields are in the vertical and in the longitudinal directions, and out-of-plane modes are the modes of which displacement field is in the transverse direction. The analytical finite element analyses will be described later. The difference as shown in the parentheses between the analytical and the experimental natural frequencies suggests the need for updating the initial finite element model using the experimental results. Fig. 5 shows mode shapes corresponding to the natural frequencies. It is observed that the high and the horizontal modes contain more noises than the low and the vertical modes since traffic loads mainly excite vertical vibration and the frequency contents of the loads are relatively low. The measurement on the top of column 3 was also important in the second and the third modes because the longitudinal directions of column 3 were different between the two mode shapes although two modes are very similar in the vertical direction. Observing the mode shapes, those estimated by the FDD method agree better with those from the finite element analysis than those estimated by the PP and the RD methods.

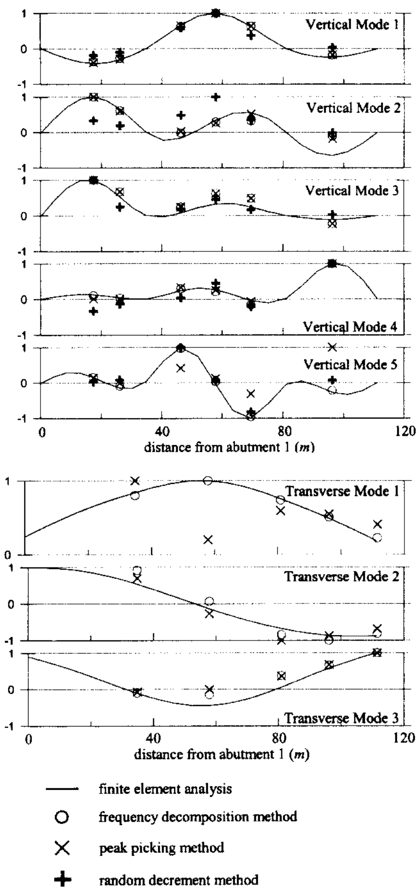


Fig. 5 Comparison of mode shapes

4.2 Forced Vibration Tests

Braking and bumping vibration tests using a water truck were carried out at the final constructional stages of the West Street On-Ramp before opened to traffic. Braking and bumping forces applied by the water truck at the middle point of each span induce bridge vibration in the longitudinal and vertical directions. The water truck weighs approximately 15000 kg when fully loaded. The sampling frequency is 100 Hz. Typical time histories of accelerations of the super-structure are shown in Fig. 6 and 7. The amplitudes of accelerations in the transverse direction are relatively smaller than those in the vertical direction. More importantly, the amplitudes

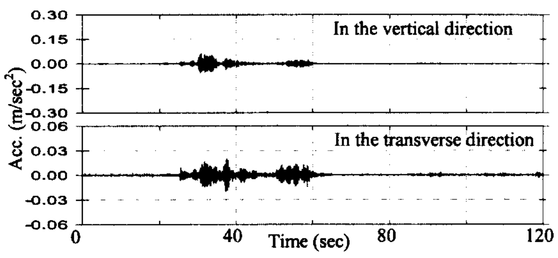


Fig. 6 Typical accelerations by braking tests

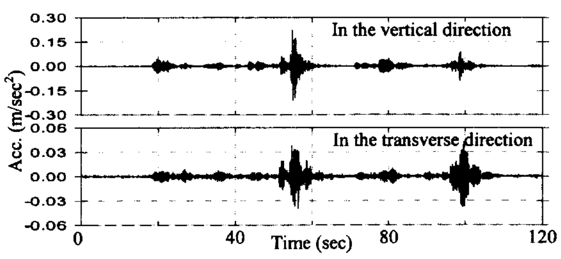


Fig. 7 Typical accelerations by bumping tests

of the accelerations in the bumping vibration tests were greater than those in the braking vibration tests.

Four sets of data from the braking vibration tests and three sets of data from the bumping vibration tests were processed, in which the time length of each data set is 10 minutes and all the data were collected at night with the traffic under the bridge blocked. The FDD method was used to extract modal parameters from the averaged vibration data with 25 percent overlapping. The sampling rate was 0.01 sec and the time length of the averaged data 40.96 sec. The first and the second natural frequencies as shown in Table 2 were estimated in the vertical and transverse directions simultaneously since the vertical and the transverse modes are coupled together. It is observed that the natural frequencies derived from the braking vibration tests are different from those from the bumping vibration tests. The natural frequencies estimated from the bumping vibration tests are maximum 3.5 percent smaller than those from the braking vibration tests, although these tests were performed under the same experimental conditions. This can be explained by the nonlinear characteristics of concrete: the larger deformation corresponds to the smaller elasticity. The magnitudes of the bumping loads were larger than the braking loads and as a result larger bridge responses were induced in the bumping vibration tests than in the braking vibration tests.

Table 2 Comparison of natural frequencies of west street on-ramp (Hz)

Test	Braking Vibration Test		Bumping Vibration Test	
Mode	Vertical	Transverse	Vertical	Transverse
1	2.148	2.148	2.075 (-0.073)	2.124 (-0.024)
2	2.465	2.465	2.441 (-0.024)	2.441 (-0.024)

Note: 'Vertical' and 'Transverse' mean the direction of vibration measurements, and the values in the parentheses are the difference between two test results.

5. Development of Baseline Model

5.1 Preliminary Finite Element Analysis

For preliminary finite element analyses, 3 dimensional (3D) finite element model as shown in Figure 8 was developed for the Jamboree Road Overcrossing. The super- and sub-structures were modeled as 3D frame elements. The cross section area and moment of inertia for each element were calculated from the design drawings and listed in Table 3. Considering that the model is for analyzing the bridge response to operational (traffic) loads, the bridge bearings at the abutments were assumed 2.035×10^7 kg/m as linear longitudinal and transverse springs, and 7.729×10^7 kg/m as linear rotational springs, which were based on FHWA.(19) The natural frequencies and the mode shapes computed using this analytical model are shown in Table 1. Sensitivity analyses of modal parameters with respect to column damages were performed. The damage is defined

Table 3 Structural parameter

Bridge	Element	Area (m^2)	Moments of inertia (m^4)		
Jamboree Road Overcrossing	Deck	5.94	7.63	3.01	59.36
	Column	3.53	2.51	0.72	1.51
West Street On-Ramp	Deck	6.45	4.55	9.80	47.30
	Column	4.21	41	2.83	1.41

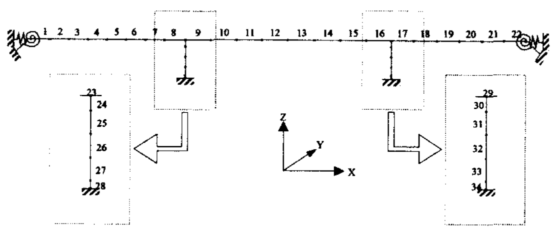


Fig. 8 Finite element model of jamboree road overcrossing

as the decrease of flexural stiffness. Column damages by earthquakes were assumed at both top and bottom sections of columns for longitudinal ground motions, and at only bottom sections of columns for transverse ground motions. The results are shown in Fig. 9, which shows the column damages decrease the lower natural frequencies and increase the rotations of columns in mode shapes. Therefore, the instrumentation of accelerometers in the longitudinal and transverse directions at the top of columns is important. Sensitivity analyses of modal parameters with respect to bearing stiffness were also performed. The bearing stiffness at abutment 4 changed from 10 to 190 percent of the one used in preliminary finite element analysis. The results are shown in Figure 10, and it shows that the modal parameters are very insensitive to the rotational bearing stiffness.

5.2 Neural Network Based System Identification

A model called back-propagation neural network or multi-layer perceptron is one of the main types used in engineering fields including approximation of structural analysis, identification of structural integrity, fault diagnosis, prediction, strategic management, decision making, and structural optimization.[20,21] The back-propagation Neural Network (NN) is used for the system identification in this study. It consists of an input layer, hidden layers, and an output layer. The relationship between input and output of a neural network can be nonlinear as well as linear, and its characteristics are determined by the weights assigned to the connections between the neurons

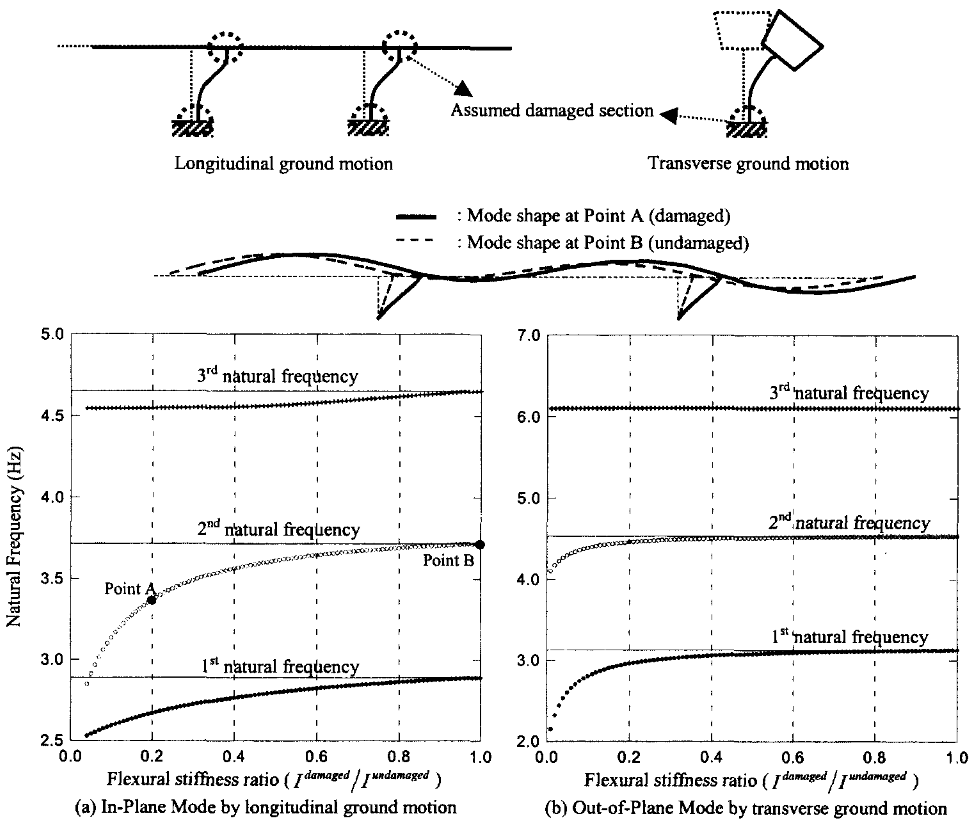
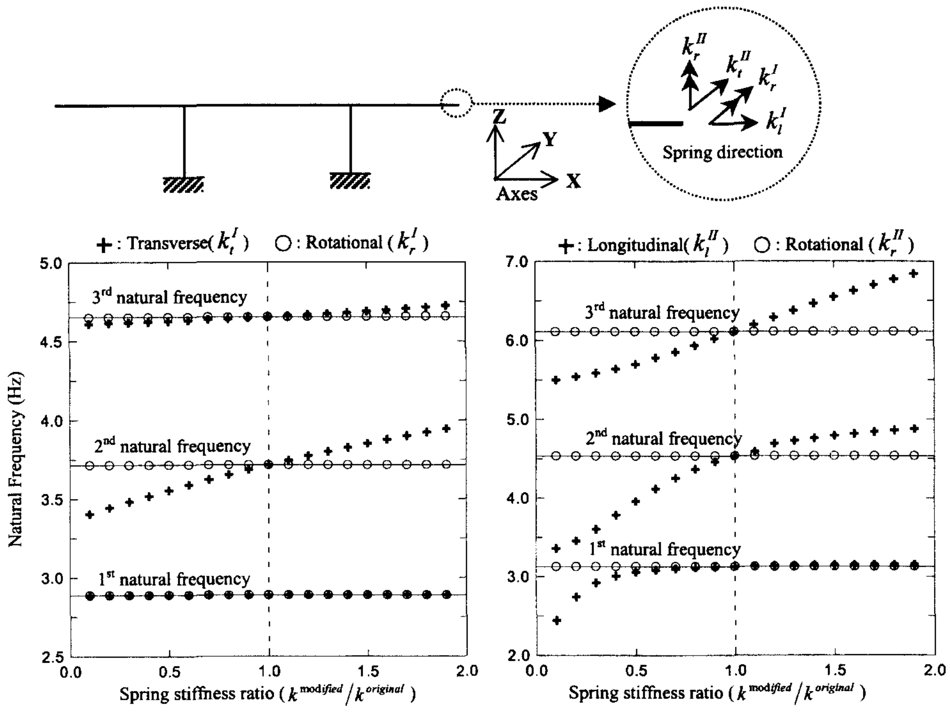


Fig. 9 Sensitivity of natural frequency for column damages



(a) In-plane mode (b) Out-of-plane mode

Fig. 10 Sensitivity of natural frequency for bearing stiffness

in two adjacent layers. Changing these weights will change the input/output relationship of the network. Systematic ways of determining the weights of the network to achieve a desired input/output relationship are referred to training or learning algorithm.

In this study, the standard back-propagation algorithm is used for training in order to recognize structural parameters (representing the stiffness and the mass matrices of a structure, and the bearing stiffness at the abutments) from measured natural frequencies and mode shapes. The procedure of the neural network-based identification is summarized as the following steps: (1) the types of input and output patterns are determined based on the variations of the structural parameters and the experiment plan for measuring the modal data; (2) the training and testing patterns are prepared by preliminary finite element analyses; and (3) the neural network architecture is determined, and network is trained using the back-propagation algorithm. The second and third procedures may be iterated, if needed. Finally the structural parameters of the baseline finite element model are estimated by inputting the measured natural frequencies and mode shapes. In the present study, the input pattern consists of the natural frequencies and the mode shapes obtained by finite element analysis of the bridge structure by randomly assuming a correction coefficient for each structural parameter.

$$\textit{Input Pattern Vector} = \{f_1, \cdots, f_l, ((\phi_{1i}, \cdots \phi_{ki}), i=1, \cdots, m)\} \tag{1}$$

where f_i is the ith natural frequency, ϕ_{ki} denotes the kth component of the ith mode shape which is normalized as $\phi_i^T \phi_i = 1$, k is the number of accelerometers installed on a structure, and l and m are the number of natural frequencies and mode shapes considered. The output pattern consists of correction coefficients of structural parameters.

Output Pattern Vector= $\{c_1, \cdots, c_n\}$ (2)

where are correction coefficients of structural parameters. The architecture of the neural network is shown in Fig. 11.

The neural network-based system identification method has several advantages compared with the other methods. Generally the components of the mode shapes may not be fully measured in the field, and the few lower modes may be measured. Accordingly the method for the development of baseline model must be suitable for using the partially and incompletely measured components of the mode shapes. The neural network is very suitable for the case. The neural network is easy to parameterize any properties of bridges, including effective shear area, as the unknowns to be identified. Generally in the case of the sensitivity methods that many of damage identification methods can be categorized as, the sensitivity matrix may be unstable, especially for complex structural systems. On the other hand, the neural network does not require the calculation on sensitivity matrix. Accordingly the method can be applied to the system identification of complex civil structures. The modeling error is the most difficult problem in the system identification of a complex structure. The errors are associated with the kind of finite elements used to model a structure, the number of finite element meshes, boundary conditions, and soil-structure interaction. Fortunately, the effect of the error may be depressed within a reasonable range in the case of the neural network-based approach, if the structure is suitably modeled with fine finite element meshes, because the approach can use directly the finite element analysis for generating training patterns. However, most of the other methods suffer from numerical instability, if the structure is modeled with fine finite element meshes.

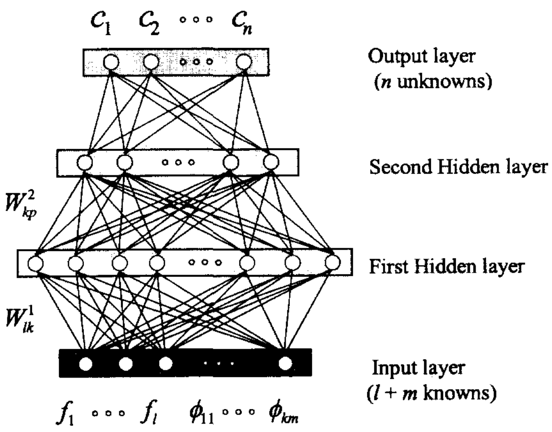


Fig. 11 Back-propagation neural network

5.3 Development 3D Baseline Model Using Ambient Vibration Measurements

The methodology involves the following steps: (I) updating in-plane structural parameters of a 3D initial baseline finite element model through vertical and longitudinal ambient vibration measurements, and then (II) identifying out-of-plane structural parameters of the 3D baseline finite element model by using transverse ambient vibration measurements. The unknown parameters in two steps can be categorized into three and two groups each, which are related with the mass and the stiffness matrices of super- and sub-structures, and bearing stiffness as follow

i) Step I (in-plane mode)

$$\boldsymbol{M}=\{c_1^I A^{super}, c_2^I A^{sub}\} \tag{3a}$$

$$\boldsymbol{K}=\{c_3^I I_Y^{super}, c_4^I I_Y^{sub}\} \tag{3b}$$

$$\boldsymbol{k}=\{c_5^I k_{l1}^{abut}, c_6^I k_{r1}^{abut}, c_7^I k_{r4}^{abut}, c_8^I k_{r4}^{abut}\} \tag{3c}$$

ii) Step II (out-of-plane mode)

$$\boldsymbol{K}=\{c_1^{II} I_Z^{super}, c_2^{II} I_Z^{sub}\} \tag{4a}$$

$$\boldsymbol{k}=\{c_3^{II} k_{l1}^{abut}, c_4^{II} k_{r1}^{abut}, c_5^{II} k_{r4}^{abut}, c_6^{II} k_{r4}^{abut}\} \tag{4b}$$

in which A^{super} and I^{super} are the effective shear area and the moment of inertia of the super-structure; A^{sup} and I^{sup} are those of sub-structures; k_{li}^{abut}, k_{vi}^{abut}, k_{ri}^{abut} and are linear bearing stiffness in the longitudinal, vertical, and rotational directions at abutment i; and c^I and $c^{II}$are correction coefficients at each step for these parameters. The shear areas of the deck are common parameters for both of in-plane and out-of-plane motions of the bridge. In this study, the shear areas have been identified in Step I since in-plane motions are more dominant and accurate than out-of-plane motions in traffic-induced ambient vibration measurements.

For the verification of the proposed method, the correction coefficients c's due to the scenario were assumed and compared with those obtained by the proposed method. The ranges of the correction coefficients in the process of the neural network were chosen from 0.5 to 1.5 for super- and sub-structures, and from 0.1 to 1.9 for bearing stiffness considering relatively high uncertainties in boundary conditions. Initial finite element model was the same one used for the preliminary analysis. Five natural frequencies and one mode shape as in-plane modal parameters were used, and three natural frequencies and one mode shape as out-of-plane modal parameters were adopted. The assumed locations of accelerometers were the same as those of actual measurements. The results are shown in Table 4 and Fig. 12. Structural parameters except the stiffness of rotational bearing springs at abutments are well identified since the stiffness of rotational bearing springs is very insensitive to the modal parameters. Without the stiffness of rotational bearing springs, the estimation of structural parameters gives more good accuracy with maximum mean error 3.67 percent.

Baseline finite element model was developed using the proposed neural network method and the

Table 4 Neural network output for verification

Correction Coefficient		Target			Output with Rotational Spring			Mean Error (%)	Output without Rotational Spring			Mean Error (%)
		1	2	3	1	2	3		1	2	3	
In-plane mode	c_1^I	1.2	1.2	1.3	1.34	1.22	1.32	6.00	1.19	1.20	1.32	1.00
	c_2^I	0.5	1.3	1.4	0.55	1.26	1.41	3.33	0.51	1.31	1.40	0.67
	c_3^I	0.7	0.7	0.5	0.76	0.71	0.57	4.67	0.70	0.70	0.50	0.00
	c_4^I	0.9	0.5	1.3	0.97	0.56	1.27	5.33	0.90	0.50	1.31	0.33
	c_5^I	1.1	1.6	1.0	1.21	1.56	0.97	6.00	1.09	1.58	1.00	1.00
	c_6^I	1.5	1.4	1.5	1.38	1.05	0.97	33.33				
	c_7^I	0.1	0.6	1.1	0.15	0.72	1.12	6.33	0.15	0.56	1.12	3.67
	c_8^I	0.2	1.8	0.3	0.26	1.80	0.74	16.67				
Out-of-plane mode	c_1^{II}	1.2	1.2	1.3	1.16	1.11	1.35	6.00	1.20	1.20	1.30	0.00
	c_2^{II}	1.1	1.0	1.0	1.08	0.91	0.95	5.33	1.10	1.01	1.00	0.33
	c_3^{II}	0.6	0.5	0.1	0.60	0.49	0.13	1.33	0.59	0.51	0.12	1.33
	c_4^{II}	0.6	1.7	0.3	0.97	0.81	0.8	58.67				
	c_5^{II}	0.8	0.4	0.9	0.81	0.43	0.83	3.67	0.81	0.41	0.90	0.67
	c_6^{II}	0.6	1.5	0.1	0.99	1.07	1.15	62.33				

Note: Mean error is the average of the absolute values of the differences between target and output values.

vibration data obtained by preliminary ambient vibration tests. The training and testing patterns and the number of modal data were the same with verification. The result is shown in Table 5. Even though the natural frequencies of the baseline model showed more good agreements than preliminary model, there were still discrepancies between measured and baseline natural frequencies since the training and testing patterns were prepared using finite element method and there were uncertainties in finite element modeling.

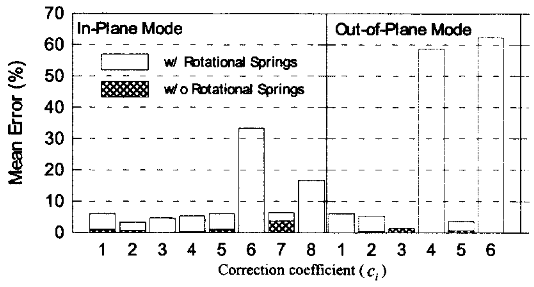


Fig. 12 Effectiveness comparison of rotational spring at abutments

Table 5 Neural network output using measurement data

Index		Correction coefficient (c_i)	Natural frequency (f_i) FDD	FE	Baseline FE
In-plane mode	1	0.93	2.954	2.889 (0.065)	2.947 (0.007)
	2	0.99	3.979	3.716 (0.263)	3.955 (0.024)
	3	0.96	4.638	4.654 (0.016)	4.798 (0.160)
	4	1.00	6.250	5.721 (0.529)	5.831 (0.419)
	5	1.25	8.911	9.400 (0.489)	9.565 (0.654)
	6				
	7	1.24			
	8				
Out-of-plane mode	1	1.00	2.612	3.126 (0.514)	2.633 (0.021)
	2	0.99	5.004	4.535 (0.469)	4.982 (0.022)
	3	0.16	9.936	10.509 (0.573)	9.745 (0.191)
	4				
	5	0.67			
	6				

Note: The values in the parentheses are the absolute value of the difference between the analytical and the experimental natural frequencies.
FDD = Frequency decomposition method
FE = Finite element analysis
Baseline FE = Finite element analysis by baseline model

6. Concluding Remarks

Sensor system including accelerometers, embedded strain gauges, displacement sensors, and pressure sensors were installed on two highway bridges, and preliminary data collection and analysis were performed after the completion of the installation. On the Jamboree Road Overcrossing, traffic-induced ambient vibrations were measured several times. The acceleration amplitudes in the vertical direction were larger than those in the transverse direction. The peak picking method, the random decrement method, and the frequency domain decomposition method were used as to extract modal parameters including natural frequencies and mode shapes from the ambient vibration data. The modal parameters were also compared with those obtained by the preliminary finite element analysis. The results showed that the high and the horizontal modes were more contaminated by noises than the low and the vertical modes, and the frequency domain decomposition method was considered as the best method among the three methods. On the West Street On-Ramp, braking and bumping vibration tests were carried out using a fully loaded water truck. The acceleration responses exhibited similar trends to those in the Jamboree Road Overcrossing. The modal parameters were estimated by the frequency decomposition method. The vertical and the transverse modes appeared to be coupled, and the natural frequencies derived from the braking vibration tests were smaller than those from the bumping vibration tests, because the braking loads are smaller that the bumping loads.

For the development of the 3-D baseline finite element model of Jamboree Road Overcrossing, the new methodology using a neural network was proposed. The proposed one was verified using assumed correction coefficients of the preliminary finite element model, and was applied to develop the baseline model of the bridge. Even though the natural frequencies of the baseline model showed more good agreements than preliminary model, there were still discrepancies between measured and baseline natural frequencies since the training and testing patterns were prepared using finite element method and there were uncertainties in finite element modeling.

With the sensor systems permanently installed on these two highway bridges, their structural performance will be continuously monitored in the future. Methodologies for assessing the structural health conditions, which are based on the monitored data considering environmental conditions and the level of exciting forces, will be developed.

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REFERENCES

(1) Bement, M.T., and Farrar, C.R., 2000, "Issues for the Application of Statistical Models in Damage Detection." Proceedings of the 18th International Modal Analysis Conference, pp. 1392 ~1398.

(2) Peeters, B., and Roeck, G.D., 2000, "One Year Monitoring of the Z24-Bridge: Environmental Influences Versus Damage Events." Proceedings of the 18th International Modal Analysis Conference, 2000, pp. 1570~1576.

(3) Priestly, M.J.N., Seible, F., and Calvi, G.M., 1996, Seismic Design and Retrofit of Bridges, John Wiley & Sons, INC., New York.

(4) Dherty, J.E., 1987, "Nondestructive Evaluation", Chapter 12 in Handbook on Experimental Mechanics, A.S. Kobayashi Edt., Society for Experimental Mechanics, Inc..

(5) Doebling, S.W., Farrar, C.R., Prime, M.B., and Shevitz, D.W., 1998, "A Review of Damage Identification Methods that Examine Changes in Dynamic Properties." Shock and Vibration Digest, 30(2).

(6) Aktan, A.E., Catbas, F.N. Grimmelsman, K.A., and Tsikos, C.J., 2000, "Issues in Infrastructure Health Monitoring for Management." Journal of Engineering Mechanics, ASCE, Vol. 126, No.7, pp. 711~724.

(7) Transportation Corridor Agencies, 1996, Design Memorandum No. 6: Seismic Design Criteria and Commentary for Structures.

(8) California Department of Transportation, 1996, Seismic Design Criteria V.1.1, July.

(9) Abdel-Ghaffer, A.M., and Scanlan, R.H., 1985, "Ambient Vibration Studies of Golden Gate Bridge. I: Suspended Structure." Journal of Engineering Mechanics, ASCE, Vol.111, No. 4, pp. 463~482.

(10) Harik, I.E., Allen, D.L., Street, R.L., Guo, M., Graves, R.C., Harrison, J., and Gawry, M.J., 1997, "Free and Ambient Vibration of Brent-Spence Bridge." Journal of Structural Mechanics, ASCE, Vol. 123, No. 9, pp. 1262~1268.

(11) Feng, M.Q., Kim, J.M., and Xue, H., 1998, "Identification of a Dynamic System Using Ambient Vibration Measurements." Journal of Applied Mechanics, ASME, Vol. 65, No. 4, pp. 1010~1021.

(12) Bendat, J.S., and Piersol, A.G., 1993, Engineering Application of Correlation and Spectral Analysis, John Wiley & Sons, New York, USA.

(13) Ewins, D. J., 1999, Modal Testing: Theory, Practice and Application, Wiley, John & Sons.

(14) Otte, D., Ponseele, P.V.D., and Leuridan, J., 1990, "Operational Shapes Estimation as a

Function of Dynamic Loads," Proceedings of the 8th International Modal Analysis Conference, pp. 413~421.

(15) Brinker, R., Zhang, L., and Andersen, P., 2000, "Modal Identification from Ambient Response Using Frequency Domain Decomposition," Proceedings of 16th International Modal Analysis Conference, San Antonio, Texas, USA, Feb., 6-10, 625~630.

(16) Cole, H.A., 1968, "On-The-Line Analysis of Random Vibrations," AIAA Paper, No. 68~288.

(17) Yang, J.C.S., Chen, J., and Dagalakis, 1985, "Damage Detection in Offshore Structures by the Random Decremental Technique," Journal of Energy Resources Technology, ASME, Vol. 106, pp. 38 ~42.

(18) Asmussen, J.C., and Brincker, R., 1996, "Estimation of Frequency Response Function by Random Decrement," Proceedings of International Modal Analysis Conference, pp. 246~252.

(19) U.S. Federal Highway Administration, 1996, Seismic Design of Bridges Design Example No. 6: Three-Span Continuous CIP Concrete Box Bridge, Report FHWA-SA-97-011, Federal Highway Administration.

(20) Masri, S.F., Smyth, A.W., Chassiakos, A.G., Caughey, T.K., and Hunter, N.F., 2000, "Application of Neural Networks for Detection of Changes in Nonlinear Systems," Journal of Engineering Mechanics, ASCE, Vol. 126, No. 7, pp. 666~676.

(21) Tsou, P., and Shen, M.H.H., 1994, "Structural damage detection and identification using neural networks," AIAA J., 32(1), pp. 176~183.

(22) Ibrahim, S.R. and Mikulcik, E.C., 1977, "A Method for the Direct Identification of Vibration Parameters from the Free Responses," Shock and Vibration Bulletin, No. 47, Part4, pp. 183~198.

(23) Juang, J.N., 1994, Applied System Identification, Prentice Hall, Englewood Cliffs, New Jersey, USA.