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Structural performance evaluation of a steel-plate girder bridge using ambient acceleration measurements

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Abstract. The load carrying capacity of a bridge needs to be properly assessed to operate the bridge safely and maintain it efficiently. For the evaluation of load carrying capacity considering the current state of a bridge, static and quasi-static loading tests with weight-controlled heavy trucks have been conventionally utilized. In these tests, the deflection (or strain) of the structural members loaded by the controlled vehicles are measured and analyzed. Using the measured data, deflection (or strain) correction factor and impact correction factor are calculated. These correction factors are used in the enhancement of the load carrying capacity of a bridge, reflecting the real state of a bridge. However, full or partial control of the traffic during the tests and difficulties during the installment of displacement transducers or strain gauges may cause not only inconvenience to the traffic but also the increase of the logistics cost and time. To overcome these difficulties, an alternative method is proposed using an excited response part of full measured ambient acceleration data by ordinary traffic on a bridge without traffic control. Based on the modal properties extracted from the ambient vibration data, the initial finite element (FE) model of a bridge can be updated to represent the current real state of a bridge. Using the updated FE model, the deflection of a bridge akin to the real value can be easily obtained without measuring the real deflection. Impact factors are obtained from pseudo-deflection, which is obtained by double-integration of the acceleration data with removal of the linear components on the acceleration data. For validation, a series of tests were carried out on a steel plate-girder bridge of an expressway in Korea in four different seasons, and the evaluated load carrying capacities of the bridge by the proposed method are compared with the result obtained by the conventional load test method.

Keywords: load carrying capacity; ambient vibration test; modal identification; model updating; deflection correction factor; impact factor; steel plate girder bridge.

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1. Introduction

Recently, structural identification techiniques have been frequently applied for the structural integrity assessment of civil infrastructures (Jaishi, *et al*. 2003, Ren, *et al*. 2004, Shama, *et al*. 2001). In the case of a bridge, the load carrying capacity is one of the widely used structural integrity index for bridge rating and maintenance. Many studies have been performed to improve the evaluation procedures of load carrying capacities of bridges (Faulkner, *et al.* 1996, Wolek, *et al.* 1996, Catbas, *et al.* 2003). Conventionally, static and quasi-static load tests with weight-controlled heavy trucks have been commonly used to evaluate the load carrying capacity of a bridge. However, the traffic has to be controlled fully or partially for the conventional load tests, which may draw complaints from the public and increase logistical cost and time. Moreover, the sensor instrumentation may be very difficult or impossible when the target bridge is located across a deep gorge or a river. In this study, an alternative method is proposed for the evaluation of the load carrying capacity of a bridge under ordinary traffic condition. This method uses a simpler measurement system, which uses the ambient vibration tests, to reduce the shortcomings of the conventional load tests. The method includes the estimations of the deflection correction factor and the impact correction factor by ambient acceleration measurements. The deflection correction factor can be estimated by (1) measuring the ambient accelerations, (2) extracting the modal properties, (3) enhancing the initial FE model, and (4) estimating the deflection correction factor, and the impact correction factor can also be estimated by (1) extracting the excited response part by a single vehicle among the full measured ambient acceleration data, (2) removing the linear shifting components of the extracted acceleration data, (3) integrating the acceleration data twice, and (4) estimating the impact factor and the impact correction factor. Using these two correction factors, the load carrying capacity of a bridge can be evaluated with consideration of the current state. Extensive field tests were carried out on a steel plate-girder bridge (Samseung Bridge) of an expressway in Korea in different seasons. The load carrying capacities of the bridge evaluated by the proposed method according to the seasonal change were compared with the results obtained by the conventional method.

2. Load carrying capacity of a bridge

2.1. Basic theory

The load carrying capacity of a bridge (P) is commonly evaluated by combining the design live load (P_r), rating factor (RF), deflection (or stress) correction factor (K_δ (or K_ε)), impact correction factor (K_i), and correction factors for traffic volume and pavement roughness (K_t, K_r) as (MOCT 2005).

$$P = P_r \times RF \times K_\delta(\text{or } K_\varepsilon) \times K_i \times K_t \times K_r \tag{1}$$

where P_r is the given design value; RF is determined by structural analysis using the initial FE model of a bridge; and K_t and K_r are empirically estimated by structural engineers. On the other hand, the two correction factors, K_δ (or K_ε) and K_i, are generally evaluated by load tests on a bridge. Static load tests have been traditionally carried out to obtain K_δ (or K_ε) by using loaded trucks, and vehicle running tests have been carried out to estimate K_i. For stable estimation of these two correction factors, loaded trucks are run through the guided way to excite the members of concern sufficiently. However, such tests obstruct traffic and draw the complaints of passers-by.

2.1.1. Basic load carrying capacity

In Korea, all bridges are classified into three classes considering their location, surroundings, and importance. The design live load (DB-24) of bridges in the first class, usually on an expressway, is specified as 432 kN in the Korean bridge design specification (MOCT 2005), which is approximately 20% larger than the design live load (HS-20) in the ASSHTO design specification (AASHTO 1997). And *RF*, which is the ratio of the live load resistance to design live load, including the dynamic effect expressed as the impact factor *i*, can be evaluated by either allowable stress design (ASD) or ultimate strength design (USD) concepts (MOCT 2005) as

$$RF^{ASD} = \frac{\sigma_a - \sigma_d}{\sigma_l(1 + i_{code})}, \quad RF^{USD} = \frac{\phi M_n - \gamma_d M_d}{\gamma_d M_d(1 + i_{code})} \tag{2}$$

where σ_a is the allowable stress; σ_d and σ_l are the stresses under design dead and live loads; ϕ is the strength reduction factor; M_n is the nominal moment strength; M_d and M_l are the moments under design dead and live loads; γ_l and γ_d are live load and dead load factors; and i_{code} is the impact factor determined by the Korean design specification (MOCT 2005) as

$$i_{code} = \frac{15}{40 + L} \leq 0.3 \tag{3}$$

where L is the effective length of a bridge in meter. The ASD concept may be applied for steel members, while the USD concept may be applied for concrete members.

The basic load carrying capacity can be obtained by combining the design live load and the rating factor. However, this value may not represent the current state of a bridge because of the differences between the numerical model of the structure and the real structure, and therefore, correction factors are necessary to consider the current bridge state.

2.1.2. Correction factors by conventional load tests

Two correction factors, the deflection (or strain) correction factor (K_δ(or K_ε)) and the impact correction factor (K_i), are introduced to describe the current state of a bridge. The deflection (or strain) correction factor can be evaluated by the conventional load test as

$$K_\delta = \frac{\delta_{calculated}^{initial\ FEM}}{\delta_{measured}} \left(\text{or } K_\delta = \frac{\varepsilon_{calculated}^{initial\ FEM}}{\varepsilon_{measured}}\right) \tag{4}$$

where $\delta_{calculated}^{initial\ FEM}$ (or $\varepsilon_{calculated}^{initial\ FEM}$) is the static deflection (or strain) calculated using the initial FE model of a bridge, and $\delta_{measured}$ (or $\varepsilon_{measured}$) is the measured static deflection (or strain) from the static or quasi-static load test using loaded trucks moving at a very low speed. Since deflection can be more easily and accurately than strain in the field, engineers are encouraged to measure and utilize the deflection information especially for the cases of small to mid size bridges which cross the land.

Vehicle running tests are also carried out at higher speeds to obtain dynamic characteristics such as impact factors, natural frequencies, mode shapes and damping values. However, mode shapes and damping values have not been used often for the evaluation of the load carrying capacity of a bridge. The main purpose of the vehicle running tests is to calculate the impact factor with consideration of the dynamic amplification effects of the passing vehicles in a real bridge condition. The impact factor is related with the vehicle mass, bridge roughness and bridge-vehicle interaction; hence, it is very difficult to obtain a consistent value from vehicle running tests. Therefore, the maximum value is usually taken as the

Fig. 1 Loaded truck for the conventional tests

impact factor for conservative estimation. The impact factor is evaluated as

$$i_{measured} = \frac{\delta_{dynamic}}{\delta_{static}} - 1 \tag{5}$$

where $\delta_{dynamic}$ and δ_{static} are the maximum values of dynamic responses and static components, respectively, as shown in Fig. 2. The static response due to a moving vehicle is usually determined by removing the dynamic components from the measured data by low pass filtering or auto regressive operation.

Then impact correction factor, K_i, is calculated as

$$K_i = \frac{1 + i_{code}}{1 + i_{measured}} \tag{6}$$

where i_{code} is the impact factor obtained from the Korean design specification as described in Eq. (3). In Eq. (2), $(1 + i_{code})$ is used to consider the dynamic effect, but this i_{code} is determined not by the current

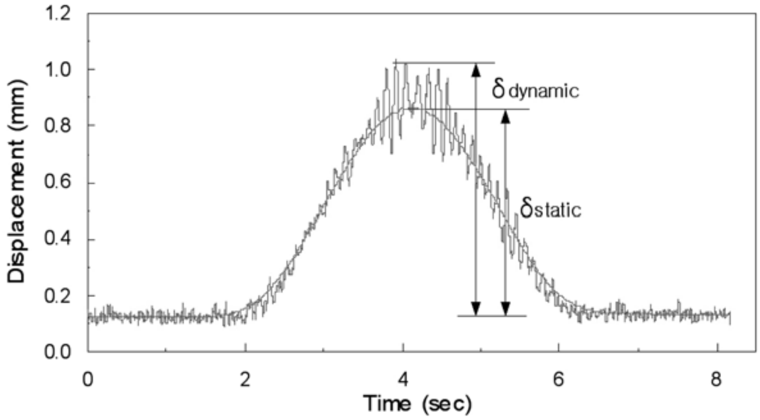


Fig. 2 Dynamic and pseudo-static deflections

bridge condition but only by the effective span length, which is determined by the bridge type and span length. Therefore, it is necessary to substitute this code-based value to the measured value to include the real bridge state by using the impact correction factor. The correction factors for traffic volume and pavement roughness (K_t, K_r) are taken as 1.0 in this study.

2.1.3. Correction factors by proposed method

An ambient vibration test (AVT) approach is proposed for estimating the deflection and impact correction factors of a bridge. Fig. 3 shows the procedures of the proposed method via ambient vibration tests. This AVT approach can be carried out under ordinary-passing traffic conditions, and requires simpler equipment for measuring acceleration. The stochastic subspace identification method (Peeters and De Roeck 1999, Yi and Yun 2004) is used to extract the modal parameters, such as natural frequencies and mode shapes, from the measured ambient acceleration data. Based on the extracted modal parameters, the initial FE model is updated by using the downhill simplex method (Nelder and Mead 1964). Downhill simplex method is realized by iteration of a customized FE program, SAP2000, and it has so simple algorithm that it is effective in reducing the time consumed for this iterative work. Deflection correction factor (K_δ) is obtained by using the deflection calculated from the initial and updated FE models by adding specific loads on them as

$$K_\delta^{proposed} = \frac{\delta_{calculated}^{initial\ FEM}}{\delta_{calculated}^{updated\ FEM}} \tag{7}$$

In the proposed estimation of K_δ, specific loads do not need to be measured from ordinary traffic, since K_δ can be easily obtained just by using any arbitrary truck loading (i.e. truck loading specified by the design specification) on initial and updated FE models, whereas in the conventional method, it is essential to measure the weights of the loaded trucks to add the same loads on initial FE model. The proposed method uses dynamic data for the correction, so this type correction can be interpreted as a dynamic model correction approach, while the conventional method uses static deflection data and can be interpreted as a static model correction approach. From the updated model, deflection akin to the real value by the conventional loading test can be calculated, if the FE model is modified reasonably enough to approximate the current condition of a bridge.

The impact correction factor can be obtained as

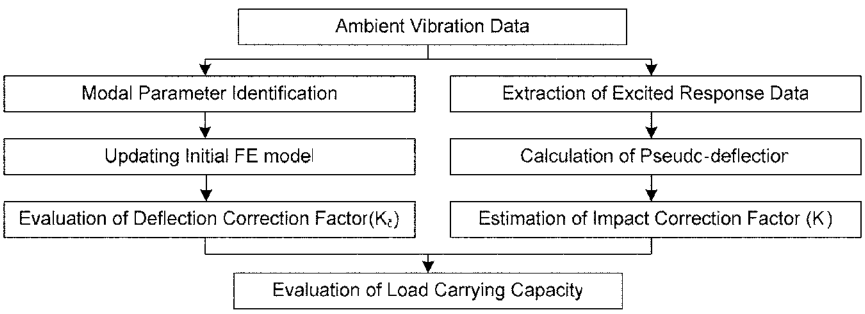
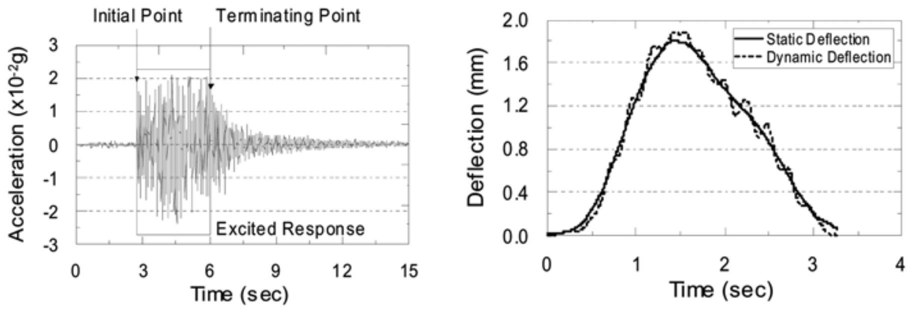


Fig. 3 Flow chart of proposed method



(a) Acceleration by a moving vehicle (b) Estimated pseudo-deflection

Fig. 4 Examples of acceleration and corresponding pseudo-deflection

$$K_i^{proposed} = \frac{1 + i_{code}}{1 + i_{measured}^{ambient\ vib}} \tag{8}$$

where $i_{measured}^{ambient\ vib}$ is the impact factor obtained from the pseudo-deflection generated by double-integration of the excited response part by a single vehicle among the full measured acceleration data. Generally, double-integration may bring two problems: the initial value problem and the low frequency noise amplification problem (Lee and Park 2003). However, what is needed in this study is not the accurate deflection itself but the ratio of dynamic deflection to static component. For stable integration, only the excited part of the measured acceleration is used with removal of any shifting components during the double integration (see Fig. 4), which can reduce the low-frequency noise amplification. For this procedure, a baseline correction was utilized. The performance of the double integration technique can be enhanced by utilizing low-frequency specialized accelerometers such as servo-type sensors (Faulkner, *et al*. 1996) and moiré-fringe type optical fiber sensors (Newport Sensors 2006). An example of extracted acceleration data and the corresponding pseudo-deflection are shown in Fig. 4.

3. Field tests specification

3.1. Description of test bridge

For validation of the proposed method, a series of tests were carried out on an example bridge on the test road of Korea Highway Corporation (KHC). The test road is a 2-lane one-way expressway built in

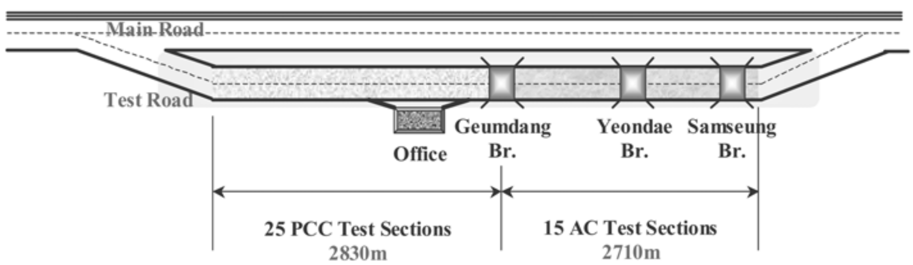


Fig. 5 KHC test road on Jungbu Inland Expressway in Korea

(a) Front View

(b) Bottom View

Fig. 6 Example bridge: Samseung Bridge

parallel to Jungbu Inland Expressway in Korea, as shown in Fig. 5. The total length of the test road is 7.7 km, and there are three bridges along the test road. A series of conventional load tests and ambient vibration tests were carried out in parallel on the Samseung Bridge in four different seasons. Samseung Bridge is a single span, steel plate-girder bridge with a span length of 40 m. It is composed of five main steel girders, floor beams, and concrete slab, as shown in Fig. 6.

3.2. Setup for conventional load tests

A series of conventional load tests were carried out on Samseung Bridge in four different seasons: S4-1, S4-2, and S4-3 in August, 2004; W4-3, and W4-4 in December, 2004; S5-1, S5-2, S5-3, and S5-4 in July, 2005, and W6-1, and W6-2 in February, 2006. Table 1 shows the specification of each test. Three heavy trucks with different weights were used for loading; i.e., 15, 30, 40 tonf, except for the 4th test. Truck weights were gauged before the test, and the vertical load as same as the gauged weight of the each truck was added to calculate the deflection of the initial FE model. For the estimation of the deflection correction factor (K_δ), quasi-static load tests were carried out at the vehicle running speed of 3 km/h. On the other hand, for the estimation of the impact correction factor (K_i), vehicle running tests were carried out at the vehicle speed of 50 km/h, except the 3rd test, which was done at the speed of 60 km/h. For measuring the deflection of the bridge, three, contact(-) type displacement transducers with connecting wires (OU displacement transducers) were installed underneath of the centers of the three main girders. During the 3rd and 4th tests in July 2005 and February 2006, a laser vibrometer (OFV-505 Standard Optic Sensor Head and OFV-5000 Modular Controller, Polytec, Inc.) was also installed at the center of the third girder to validate the

Table 1 Specification of tests carried out on Samseung Bridge

Test	Date	Set	Truck Weight (ton)	Truck Speed (km/h)
1st	2004.8.4	S4-1, S4-2, S4-3	15ton, 30ton, 40ton	3, 15, 30, 50
2nd	2004.12.3	W4-3, W4-4	15ton, 30ton, 40ton	3, 15, 30, 50
3rd	2005.7.6	S5-1, S5-2, S5-3, S5-4	15ton, 30ton, 40ton	3, 60
4th	2006.2.20	W6-1, W6-2	15ton, 40ton	3, 50

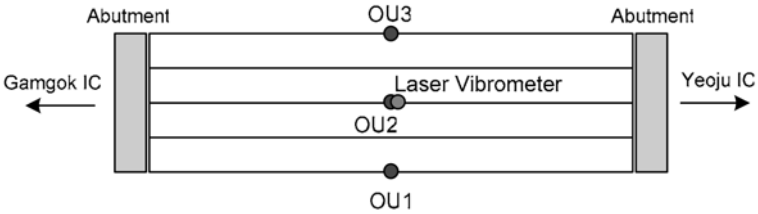
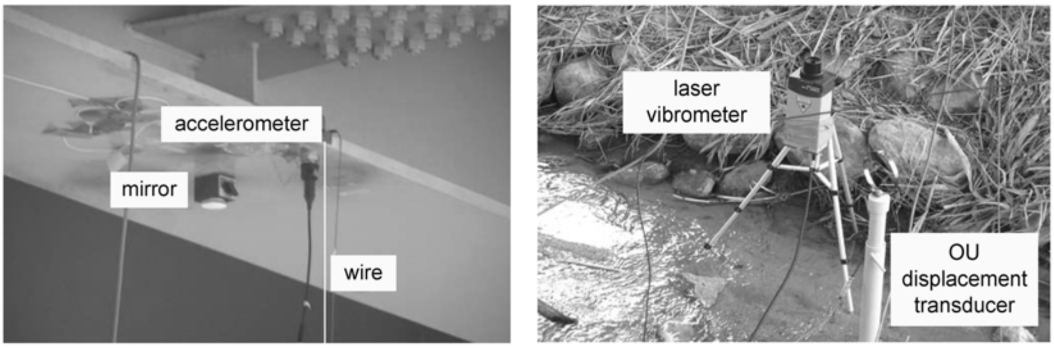


Fig. 7 Locations of installed sensors for load tests



(a) ICP Type Accelerometer, mirror and wires (b) Laser Vibrometer and OU Displacement Transducer

Fig. 8 Sensors used in the tests

performance of the OU displacement transducers. Fig. 7 shows the locations of all the sensors installed at the Samseung Bridge for the conventional load tests, and Fig. 8 is a picture of the installed sensors.

3.3. Setup for ambient vibration tests

For ambient vibration tests, 21 accelerometers were installed on the bridge. Ambient vibrations were measured for 30 minutes at the sampling frequency of 200 Hz after each conventional load test with trucks. Wind and the traffics on the adjacent bridge were the main vibration sources during the ambient vibration tests. After ensuring that the high frequency components of acceleration of over 100 Hz were very small, a low-pass filter with the cut-off frequency of 90 Hz was utilized. Fig. 9 shows the locations of the accelerometers installed at Samseung Bridge for the ambient vibration tests.

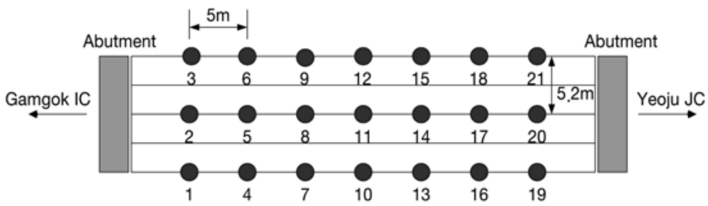


Fig. 9 Locations of installed accelerometers

4. Evaluation of load carrying capacity

4.1. Rating factor by structural analysis

Before analyzing all data from the conventional and ambient vibration tests, rating factor (*RF*) was calculated based on the specification described as Eq. (2). The bridge was modeled by using a 3-D shell and frame elements in SAP2000, as shown in Fig. 10. The design live load (DB-24) is specified as 432 kN in the Korean bridge design specification (MOCT 2005), which is approximately 20% larger than the design live load HS-20 (360 kN) in the AASHTO design specification (AASHTO 1997). *RF*s were calculated with consideration of the material property of each member. The ASD concept was applied for the steel girders, while the USD concept for the concrete slab. In this study, the γ_l and γ_d of the concrete slab are taken as 2.15 and 1.30, respectively, and ϕ is taken as 0.85. The calculated *RFs* are shown in Table 2. The lower flanges of the main girders had the smallest *RF* of 1.40 for the Samseung Bridge. Therefore, *RF* is taken as 1.40 in this study.

4.2. Correction factors from conventional load tests

Deflection correction factors (K_δ) were obtained by using the static deflections obtained from the quasi-static load tests carried out at a very low vehicle speed of 3 km/h. The results obtained from 3 sets

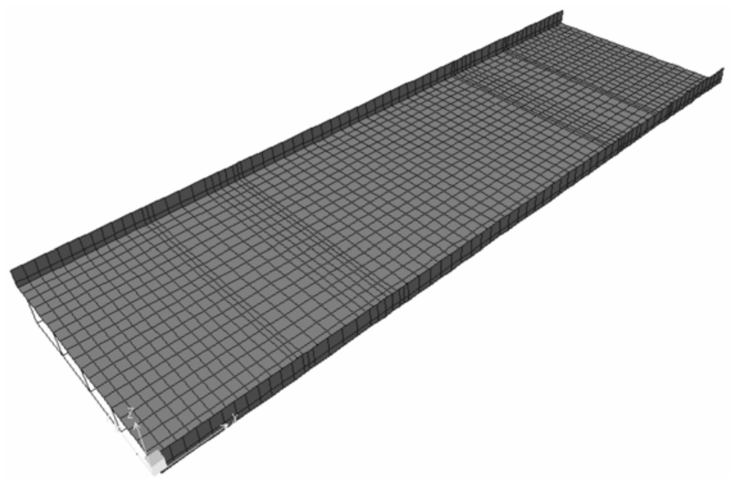


Fig. 10 FE model of Samseung Bridge (SAP2000)

Table 2 Rating factors of Samseung Bridge

Member (USD)		M_n (kN·m)	M_d (kN·m)	M_l (kN·m)	i_{code}	*RF*
Concrete Slab		127.99	4.90	27.44	0.188	1.46
Member (ASD)		σ_a (MPa)	σ_d (MPa)	σ_l (MPa)	i_{code}	RF
Third Girder	Slab	7.94	1.02	2.58	0.188	2.26
	Upper Flange	186.2	102.30	14.19	0.188	4.98
	Lower Flange	186.2	99.13	52.03	0.188	1.40

Table 3 Deflection correction factors (K_δ) obtained from the quasi-static load tests

Set	$\delta_{calculated}^{initial\ FEM}$	Deflection(mm)			Deflection correction factor (K_δ)		
		15ton	30ton	40ton	15 ton	30 ton	40 ton
S4-1	1.038 mm (15ton)	0.674	1.455	1.777	1.540	1.401	1.554
S4-2		0.627	1.380	1.678	1.656	1.478	1.646
S4-3		0.668	1.464	1.767	1.554	1.393	1.563
W4-3		0.943	1.380	1.881	1.101	1.478	1.468
W4-4	2.039 mm (30ton)	0.945	1.433	1.933	1.098	1.423	1.429
S5-1		0.647	1.280	1.710	1.605	1.593	1.615
S5-2		0.577	1.172	1.544	1.8	1.74	1.789
S5-3		0.572	1.165	1.549	1.814	1.75	1.783
S5-4	2.762 mm (40ton)	0.573	1.181	1.576	1.81	1.727	1.753
W6-1		0.957		1.827	1.085		1.512
W6-2		0.889		1.662	1.168		1.662

Table 4 Impact factors ($i_{measured}$) and impact correction factors (K_i) obtained from the vehicle running tests

Set	i_{code}	Impact Factor ($i_{measured}$)	Impact Correction Factor (K_i)
S4-1		0.107	1.073
S4-2		0.100	1.080
S4-3		0.092	1.088
W4-3		0.069	1.111
W4-4		0.175	1.011
S5-1	0.188	0.305	0.910
S5-2		0.314	0.904
S5-3		0.307	0.909
S5-4		0.196	0.993
W6-1		0.147	1.036
W6-2		0.157	1.027

of tests in different seasons were found in the range of 1.39-1.65 in August, 2004; 1.10-1.48 in December, 2004; 1.59-1.81 in July, 2005; and 1.09-1.66 in February, 2006, as shown in Table 3. For a conservative estimation, the smallest values were selected to calculate the load carrying capacity of each test. Impact factors were estimated from the deflection data obtained when the heaviest truck (40 tonf) was running at a speed of 50 km/h (60 km/h in the 3rd test). From Eq. (3), the design impact factor (i_{code}) was obtained as 0.19. Using these values, the impact correction factor (K_i) was obtained, and the estimated impact factors and impact correction factors are shown in Table 4.

4.3. Results of ambient vibration tests

4.3.1. Deflection correction factors by the proposed method

Deflection correction factors ($K_\delta^{proposed}$) were estimated by the proposed method. To this end, the first 6 modal properties including natural frequencies and mode shapes were extracted from the measured ambient acceleration data? by the stochastic subspace identification method, and they

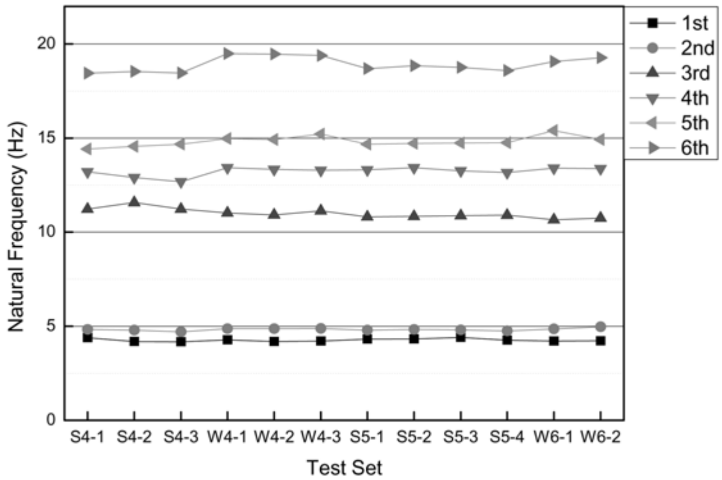


Fig. 11 First 6 natural frequencies extracted by SSI using ambient accelerations

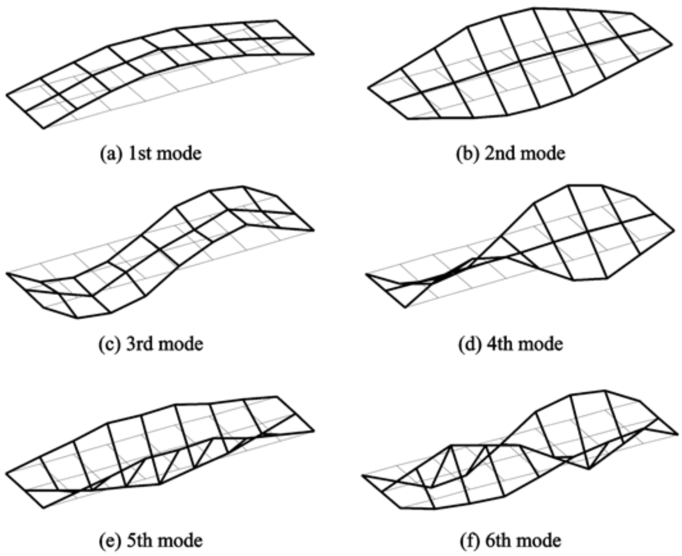


Fig. 12 First 6 mode shapes extracted by SSI using ambient accelerations (S5-2)

are plotted as shown in Fig. 11 and Fig. 12. Fig. 12 shows that the natural frequencies change along with the season, and the natural frequencies obtained in winter are slightly higher than those obtained in summer.

Using the extracted modal properties, the initial FE model was updated. The downhill simplex method was employed as an updating algorithm and SAP2000 was used to calculate the modal properties of the updated FE model, iteratively. The objective function was constructed by using the differences between the measured and estimated natural frequencies, and the constraint equations were considered to limit the differences between the measured and estimated mode shapes as

Table 5 Parameters used in model updating of Samseung Bridge

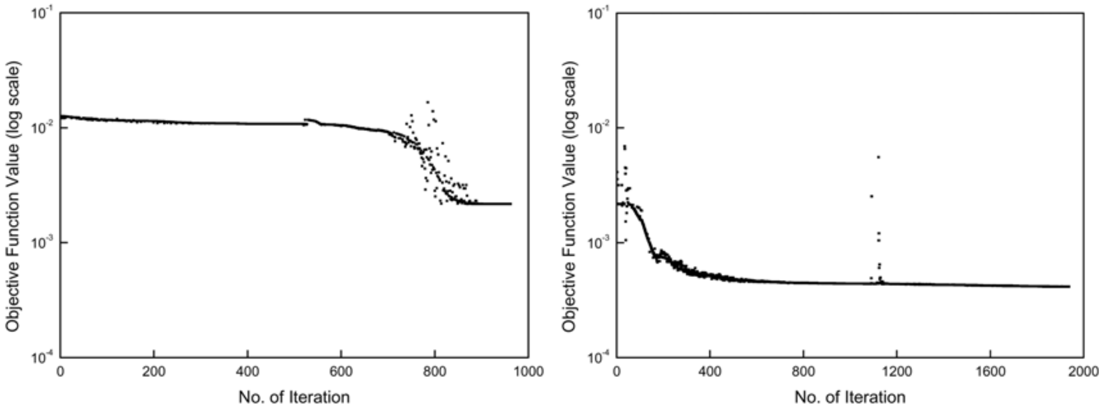
First Step			Second Step		
Members	Updating Parameters	No.	Members	Updating Parameters	No.
Support	Spring Constant (k_{spring})	1	Support	Spring Constant (k_{spring})	2
Concrete Slab	Young's modulus (E)	1	Concrete Slab	Young's modulus (E)	1
Main Girder	Second moment of inertia (I_{yy})	5	Main Girder	Second moment of inertia (I_{yy})	5
	Torsional coefficient (J)	0		Torsional coefficient (J)	5
Floor Beam	Second moment of inertia (I_{yy})	1	Floor Beam	Second moment of inertia (I_{yy})	9
	Torsional coefficient (J)	1		Torsional coefficient (J)	9
	Total	9		Total	31

$$_{\min}J = \sum_{i=1}^{N_m}\left\{w_i\left(\frac{f_i^c - f_i^m}{f_i^m}\right)\right\}^2 \text{ subjected to } \left|\phi_{ji}^c - \phi_{ji}^m\right| \le \varepsilon \tag{9}$$

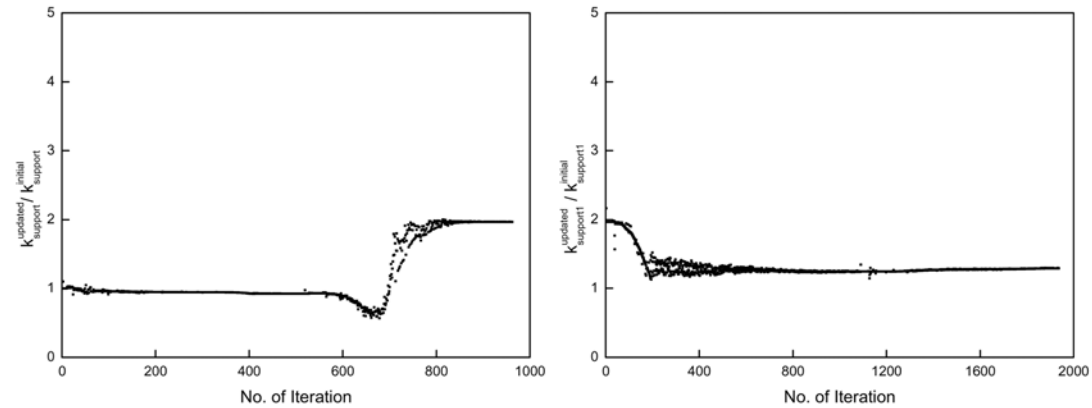
where f_i is the i-th natural frequency; ϕ_{ji} denotes the j-th component of the i-th mode shape φ_i, which is normalized as $\varphi_i\varphi_i = 1$; w_i is the weighting factor for the i-th mode; N_m is the number of modes used for model updating (in this study, $N_m = 6$); and ε is the admissible error bound for the mode shapes, and MAC (modal assurance criterion) was calculated at every computing step to minimize ε, in other words, to make the calculated modal properties agree with the measured ones. The superscripts m and c denote the measured and the calculated data, respectively.

If the number of updating parameters is too larger than the number of input information used for constructing the object function and constraint equations, the possibility of falling in a local minimum increases as the optimizing process proceeds. Therefore, model updating was processed in two steps to reduce the ill-posedness during the updating procedure. At first, the updating parameters were one equivalent spring constant at the supports, Young's modulus of the concrete slab, and the 2nd moments of inertia for five main girders and equivalent 2nd moments of inertia and torsional coefficient for nine floor beams. After the first step of model updating, 31 parameters were selected in the next model updating, as shown in Table 5. They were spring constants at two supports, Young's modulus of the concrete slab, and the 2nd moments of inertia and torsional coefficients for five main girders and nine floor beams. Fig. 13 shows the change of parameters as the updating process goes on at the 1st step (left-hand side) and 2nd step (right-hand side). At the 1st step, objective function and other parameters converge into specific values very slowly. At the 2nd step, in which the number of updating parameters is increased from 9 to 31, the objective function decreases to a smaller value, which means additional updating using properly chosen parameters can make updated model much closer to the real structure. The reason why the convergence rate for the 2nd step is faster than that for the 1st step is the constraint effect is released by increasing the number of updating parameters. After updating the FE model, the natural frequencies of the initial FE model, updated FE model, and measured ones were compared, as shown in Fig. 14, which shows that the natural frequencies of the updated FE model became closer to the measured values than those of the initial FE model.

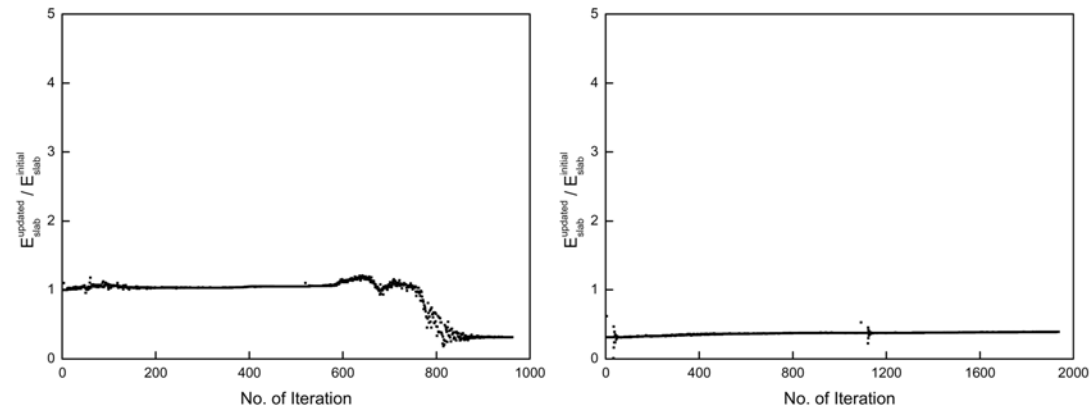
Using the updated FE models based on the modal properties obtained from the ambient vibration tests and by loading the gauged weights of loaded trucks, the deflection of the center of the third girder was obtained just to confirm the feasibility of the updated models. This deflection was compared with the deflections of the initial FE models. Using the deflections of the initial FE model and the updated FE



(a) Objective function



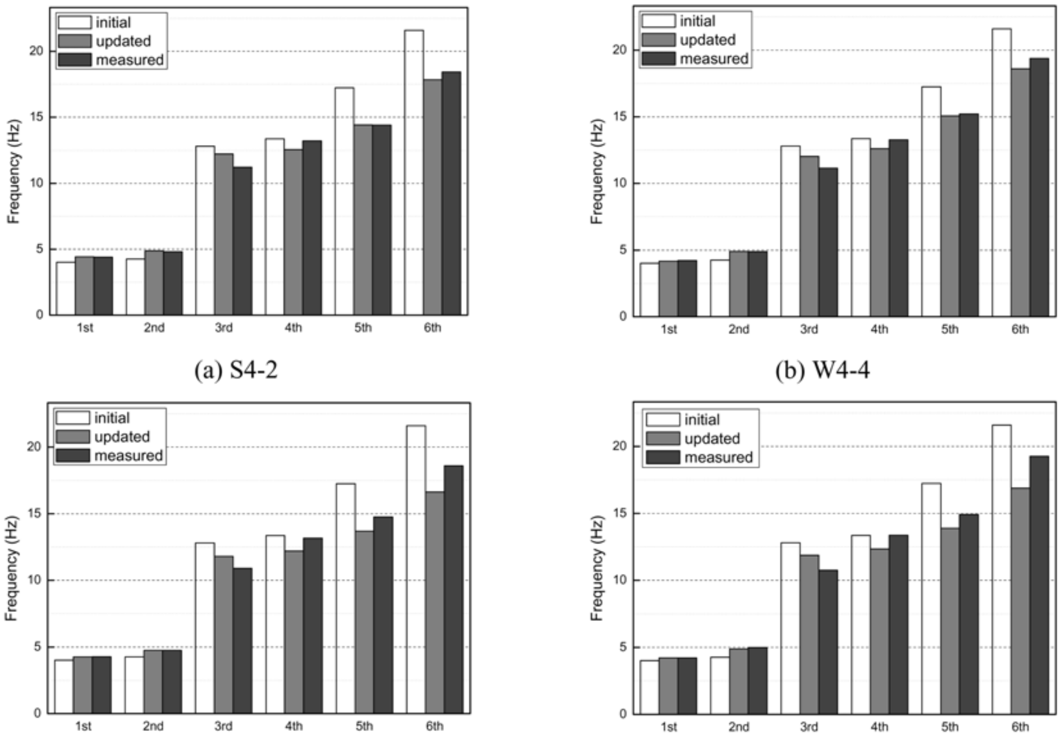
(b) Spring constant at left support



(c) Young's modulus of concrete slab

Fig. 13 Optimization procedures by downhill simplex algorithm (1st + 2nd step)

model, deflection correction factors (K_δ) were estimated and compared with those obtained by the conventional method using OU displacement transducers in Fig. 16. K_δ values by the proposed method show significant difference from those by the conventional method. Since OU displacement transducer



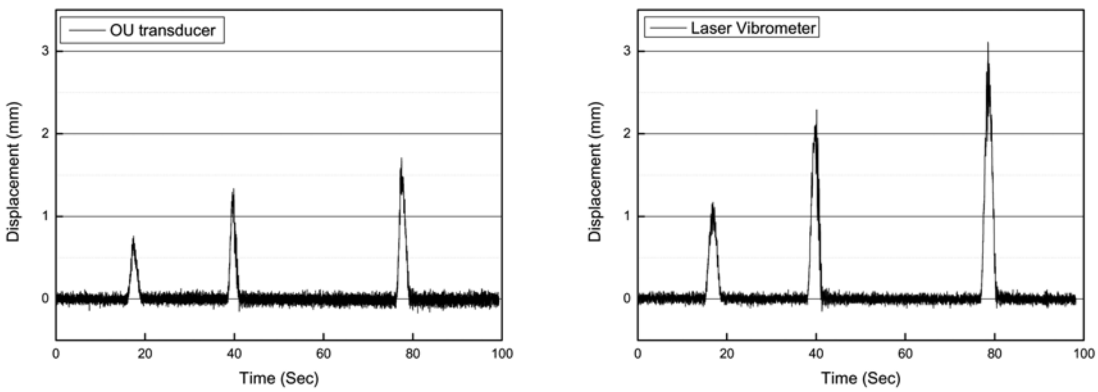
(a) S4-2

(b) W4-4

(c) S5-4

(d) W6-2

Fig. 14 Examples of natural frequencies of updated models



(a) OU displacement transducer

(b) Laser vibrometer

Fig. 15 Comparison of data from OU displacement transducer and laser vibrometer

uses a wire as a medium to transfer the deflection of the girder to the gauge, it is highly likely to indicate a wrong value when it is installed underneath the girder, which is very high up from ground.

To validate the measurements by the OU displacement transducer, a laser vibrometer was additionally installed during the 3rd test in July, 2005, and 4th test in February, 2006. Fig. 15 shows the comparison of data from the OU displacement transducer and laser vibrometer obtained when 3 trucks ran by turns

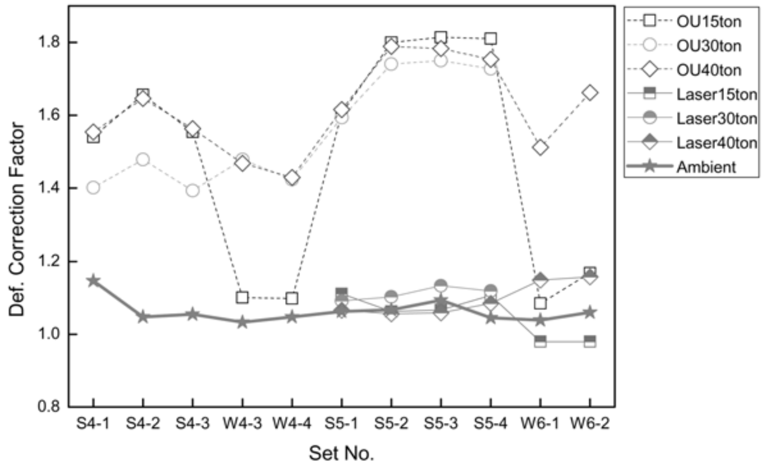


Fig. 16 Deflection correction factors obtained by conventional and proposed methods

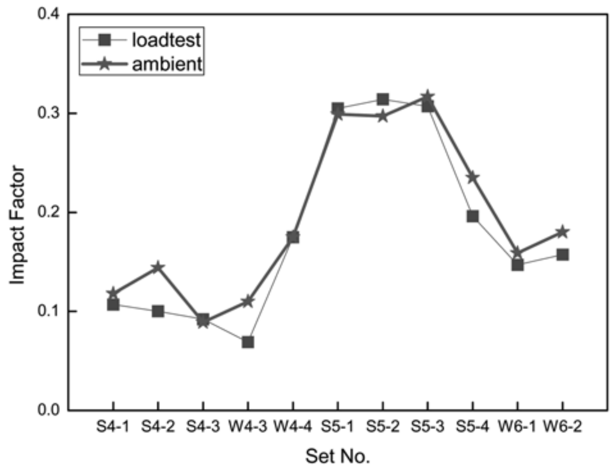


Fig. 17 Impact factors obtained by the conventional and proposed methods

in the 3rd test. It shows that the values of the OU displacement transducer are smaller than those of the laser vibrometer. These larger values made the deflection correction factors estimated by the conventional method using the OU displacement transducer larger than those estimated by the proposed method. Therefore, the K_δ's were re-estimated by using the data from the laser vibrometer, and the results are also shown in Fig. 16. K_δ's by the proposed method are very similar to the values obtained by the conventional method using the laser vibrometer data. The overall results by the proposed method are found to be very consistent, independent of the season.

4.3.2. Impact correction factors by the proposed method

For the estimation of the impact correction factor (K_i), impact factors were estimated using the pseudo-deflections, which were obtained from the acceleration data by double integration. An example case of the measured acceleration and the corresponding pseudo-deflection is shown in Fig. 4. The

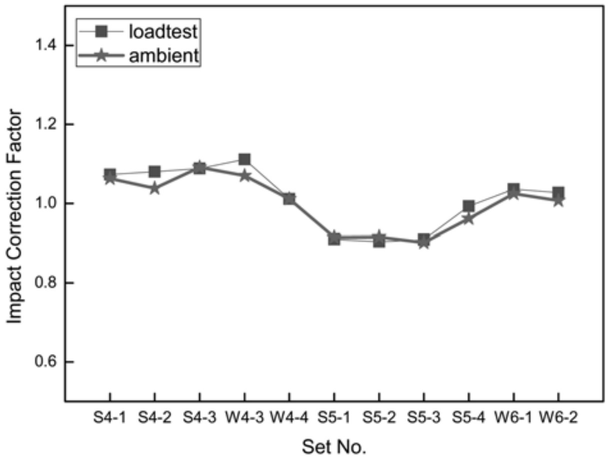


Fig. 18 Impact correction factors obtained by the conventional and proposed methods

Table 6 Load carrying capacities (P) by the conventional (laser vibrometer) and proposed methods (AVTs)

Sets	P_r	RF	Conventional method (laser vibrometer)			Proposed method (AVTs)		
			K_δ	K_i	P	K_δ	K_i	P
S5-1	DB-24	1.40	1.092	0.910	DB-33.8	1.063(2.66)	0.915(0.55)	DB-33.1(2.07)
S5-2			1.063	0.904	DB-32.7	1.067(0.38)	0.916(1.33)	DB-33.2(1.53)
S5-3			1.067	0.909	DB-33.0	1.094(2.53)	0.902(0.77)	DB-33.6(1.82)
S5-4			1.107	0.993	DB-37.4	1.045(5.60)	0.962(3.12)	DB-34.2(8.56)
W6-1			0.980	1.036	DB-34.5	1.039(6.02)	1.025(1.06)	DB-36.2(4.93)
W6-2			0.980	1.027	DB-34.2	1.060(8.16)	1.007(1.95)	DB-36.3(6.14)

[a]Values in parentheses are the relative errors (%) of the results by the proposed method.

impact factors are estimated by using the pseudo-deflections of each test set and compared with impact factors obtained by the conventional method, as shown in Fig. 17, and the impact correction factors estimated by using $i_{measured}^{ambient\ vib}$ ($K_i^{proposed}$) are compared with those estimated by the conventional method (K_i), as shown in Fig. 18. The impact factors and the impact correction factors estimated by the proposed method ($i_{measured}^{ambient\ vib}$ and $K_i^{proposed}$) are very similar to those obtained from the real deflection data ($i_{measured}$ and K_i).

4.3.3. Comparison of load carrying capacities by conventional and proposed method

Using the correction factors obtained above, the load carrying capacities of the bridge were evaluated, as provided in Table 6. The results show that the load carrying capacities evaluated by the proposed method are reasonably close to the values obtained by the conventional method using the laser vibrometer, and are very consistent independent of the test season. The errors in the deflection correction factors, the impact correction factors and the load carrying capacities are in the ranges of 0.5-8.2%, 0.5-3.2%, and 1.5-8.5%, respectively.

5. Conclusions

A new method for evaluation of the load carrying capacity of a bridge is proposed. This method uses ambient vibration tests, so it does not require traffic control during the tests and has simpler installation of sensors than that of the conventional method. First, modal properties, such as natural frequencies and mode shapes, were extracted by using SSI in four test seasons from the ambient acceleration data, and the deflection correction factors were evaluated by using the updated FE model based on the modal properties. The impact correction factors were evaluated by using the pseudo-deflection data obtained by double integration of the excited response part of measured acceleration data. The results of a series of field tests on the target bridge can be summarized as

1) The accuracy of the deflection sensor is very critical in the conventional method. The conventional OU displacement transducer did not provide accurate deflections of the bridge girder, but the laser vibrometer gave good results.
2) The deflection correction factors by the proposed method using the updated FE model were very close to those obtained by the conventional method and the deflection obtained by using the laser vibrometer.
3) The impact correction factors by the proposed method using the pseudo-deflection were sufficiently close to those by the conventional method.
4) The proposed method gave very consistent results for the load carrying capacity regardless of the test season, and the results were reasonably close to those obtained by the conventional method.
5) Using the proposed method, the load carrying capacities of bridges can be efficiently evaluated, even when the bridges are surrounded by harsh condition in the conventional load test.

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