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Evaluation of natural vibration characteristics of a steel strutted-beam rigid frame bridge

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ABSTRACT: In order to gain a basic knowledge for establishing rational seismic strengthening procedures for an existing steel strutted-beam rigid frame bridge, a field vibration test was conducted to investigate the natural vibration characteristics of a real bridge. In this study, to measure the vibration accelerations of the bridge, 45 high-sensitive servo-type vibration meters were used together with a wireless LAN system. The bridge response was monitored under the vibration caused by passing unregulated traffic. Moreover, to investigate the damage level of the bridge, the actual natural vibration frequencies and modes of the bridge were compared with the numerical results obtained from a 3D-FE analysis. The results obtained from this study were as follows: 1) the natural vibration frequencies and modes of the bridge can be appropriately evaluated by conducting an ambient vibration test; and 2) it was confirmed that the natural vibration frequencies and modes obtained from the ambient vibration test lie between the numerical analysis results obtained assuming pinned and roller supports at the ends of the superstructure.

1 INTRODUCTION

After the Great Hanshin Earthquake that occurred in 1995, the strengthening and/or retrofitting of the many existing bridges of the national highway have almost been finished in Japan. However, a few types of bridge such as arch, truss, and rigid frame bridges have not yet been strengthened due to an inadequate treatment in the guideline for rehabilitation, because the dynamic responses of these bridges are very complicated and the axial forces of the members were varied alternatively in a strong earthquake. In recent years, an effort has been made to replace the existing members with vibration-control dampers and/or braces for reducing vibration amplitude and for absorbing the vibration energy of the bridges in an earthquake (Nonaka et al. 2003, and Chen et al. 2011). However, it may not be possible to strengthen all types of bridges because of high construction costs.

From this point of view, in order to gain a basic knowledge for the establishment of a rational seismic strengthening procedure for an existing steel strutted-beam rigid frame bridge, a field vibration test was conducted to investigate the natural vibration characteristics of a real bridge. Furthermore, to evaluate the damage level of the bridge, real natural vibration frequencies and modes of the bridge were compared with the numerical results obtained from a 3D-FE analysis

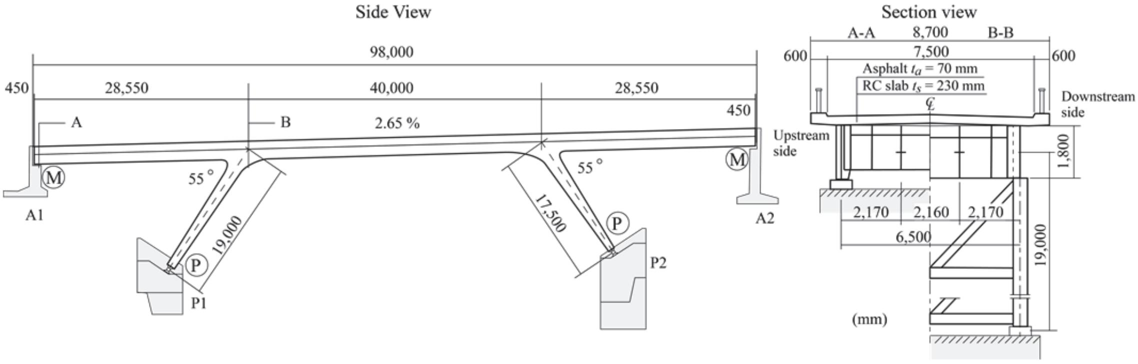


Figure 1. General view of the Miyama bridge.

(a) General view

(b) Close-up of steel pier

Photo 1. View of Miyama bridge.

with the geometric dimensions according to construction drawings. Here, the numerical analysis was performed using ABAQUS (2010).

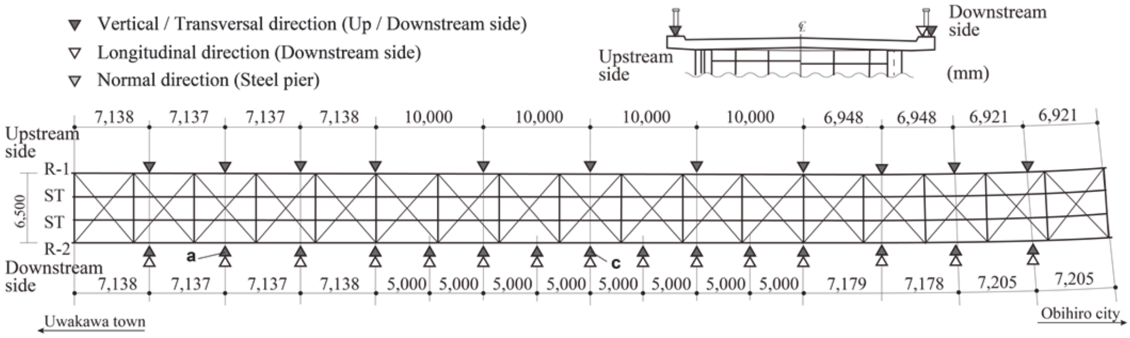
2 BRIDGE DESCRIPTION

The Miyama bridge, located in the southern part of Hokkaido, Japan, was constructed in 1979. The type of bridge is a steel strutted-beam (π-shaped) rigid frame bridge as shown in Fig. 1. The bridge was slightly curved between the P2 pier and the A2 abutment with a longitudinal slope of 2.7 %. Both ends of the steel piers and of the superstructure are a pinned and a roller support, respectively. Photo 1 shows a view of the Miyama bridge.

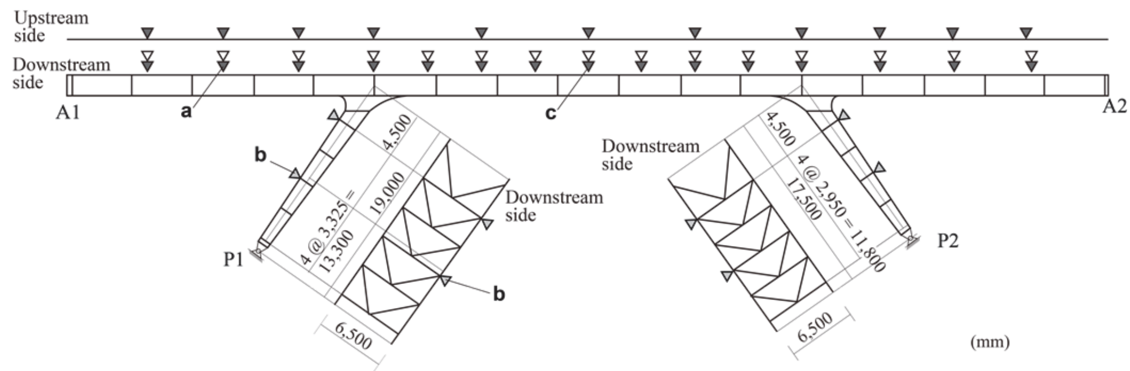
3 OUTLINE OF EXPERIMENT AND EXPERIMENTAL RESULTS

3.1 Measuring system

In total 45 high-sensitive servo-type vibration meters were located on the sides of the slab and the downstream side of the piers, respectively, to accurately evaluate the natural vibration frequencies and modes of the bridge as shown in Fig. 2. Here, two kinds of field test were conducted to specify the vertical and transversal vibration modes. The transversal vibration modes were measured by changing the sensitivity direction of the vibration meters from the



(a) Plan view



(b) Side view

Figure 2. Locations of measuring points.

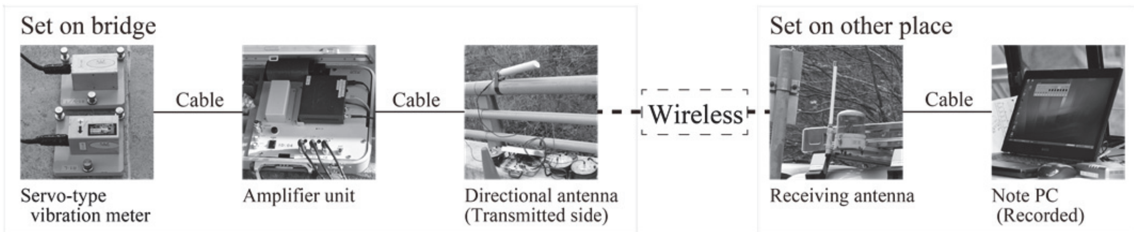


Figure 3. Wireless LAN system.

vertical to the transversal direction as shown in Fig. 2. The bridge response was monitored under the ambient vibration after the unregulated passing of traffic by means of a wireless LAN system. The measured data were recorded using a notebook, in which sampling time for measuring the time histories of the accelerations was set as 5 ms (200 Hz). Figure 3 shows the wireless LAN system applied in this study.

3.2 Evaluation of the dynamic characteristics of the bridge

Many experimental techniques have been established for evaluating dynamic characteristics (natural vibration frequencies, modes, and damping coefficients) of existing bridges. In this study, the vibration characteristics of the bridge were investigated by using the Fourier spectrum of acceleration response at each measuring point. The vibration modes were obtained based on the following procedures (Komuro et al. 2009):

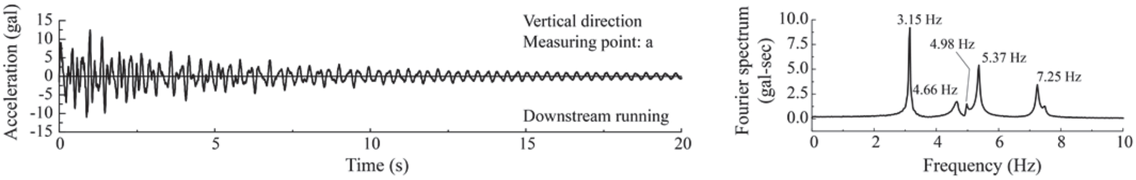
1. The Fourier spectrum of acceleration time history at each measuring point after vehicles passing was obtained;
2. The predominant natural vibration frequencies were identified inspecting the Fourier spectrum at the marked point;
3. The time histories of harmonic vibration with the predominant frequency were obtained at all measuring points numerically using Fourier and phase spectra;
4. The acceleration amplitudes at each one-forth period of the harmonic vibration were plotted at all measuring points; and
5. The uncoupled natural vibration mode was deterimed confirming no significant difference between the modes at every one-forth period.

The damping coefficient for each natural vibration was also determined based on the following procedures:

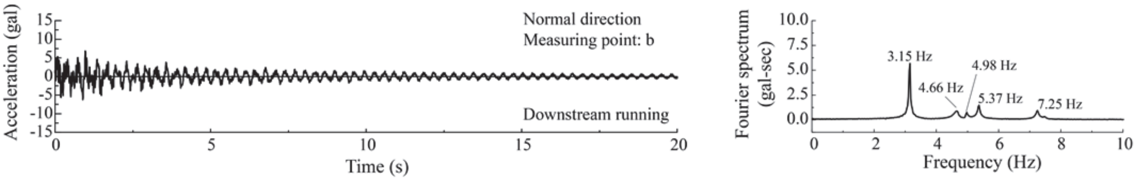
1. For each natural vibration, the time history of the damped harmonic vibration of acceleration was numerically made by using Fourier and phase spectra with a band frequency of 0.05 Hz at the natural vibration frequency; and
2. The damping coefficient for a specified natural vibration of acceleration was evaluated using the maximum amplitude at each period and assuming damped free vibration.

3.3 *Experimental results*

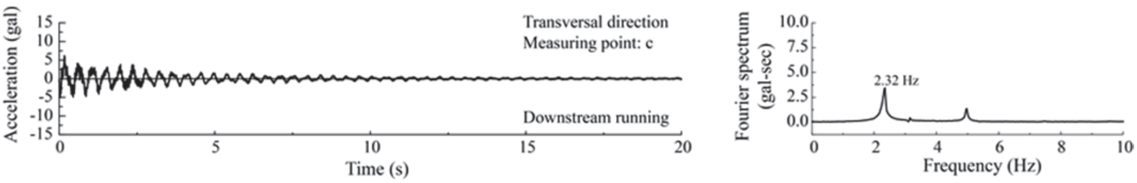
Figure 4 shows an example of acceleration response and its Fourier spectrum at points a, b, and c (see, Fig. 2). The maximum amplitudes of the acceleration response for the superstructure and the steel piers were distributed in the region from about 5 to 13 gal. The maximum values of the



(a) Point: a (vertical direction)



(b) Point: b (normal direction)



(c) Point: c (transverse direction)

Figure 4. Acceleration response and its Fourier spectrum.

Table 1. Comparison of natural vibration frequencies between experimental and numerical results.

Vibration mode		Frequency (Hz)			Damping coefficient
		Experimental results	Numerical results		
			Roller support	Pinned support	h
		f_e	f_{am}	f_{af}	(%)
1	1st horizontal	2.32-2.34	1.87	2.16	1.16
2	1st flexural	3.13-3.15	3.09	3.24	0.42
3	2nd flexural	4.66-4.76	4.66	3.75	-
4	1st torsional	4.98	5.11	5.15	-
5	3rd flexural	5.32-5.40	5.08	5.53	-
6	2nd torsional	7.25-7.45	6.91	-	-

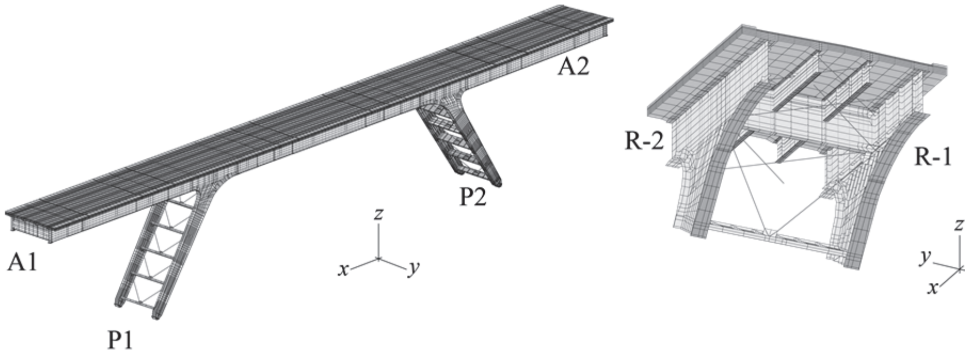


Figure 5. FE model.

Table 2. List of material properties for numerical analysis.

Material	Young's modulus E (GPa)	Density ρ (g/cm^3)	Poisson's ratio ν
Steel	206	7.85	0.30
RC	30	2.50	0.20
Asphalt	9.8	2.30	0.35

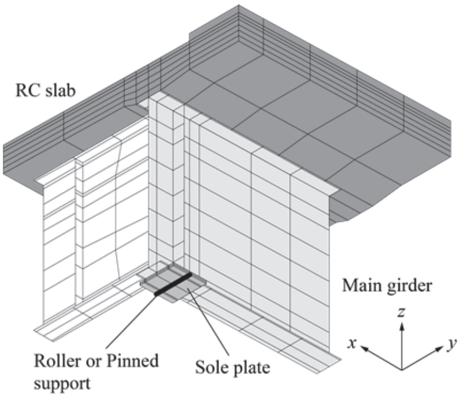
Fourier spectrum are clearly indicated at some frequencies, and the frequencies corresponding to these maximum spectral values are chosen as predominant ones in this study.

The natural vibration frequencies and damping coefficients of the bridge obtained from the field vibration test are summarized in Table 1. From this table, it is observed that the natural vibration frequencies were distributed in a small range. The damping coefficients were only specified for two modes: 1st horizontal; and 1st flexural vibration modes, because the adjoining two predominant frequencies of the other modes were very close to each other.

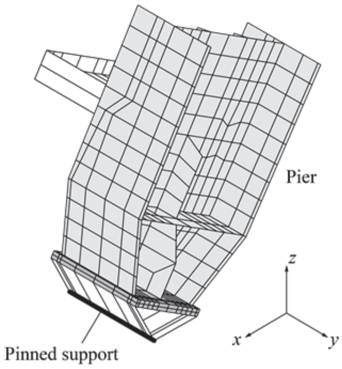
4 FINITE ELEMENT ANALYSIS

In this study, in order to investigate the damage level of the bridge, the natural vibration frequencies and modes of the bridge evaluated using field test data were compared with the numerical results obtained from a 3D-FE analysis for bridge dimensions according to construction drawings.

Figure 5 shows the FE model of the Miyama bridge. In this analysis, girders and steel piers were

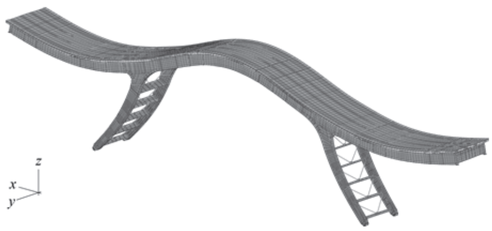


(a) Abutment

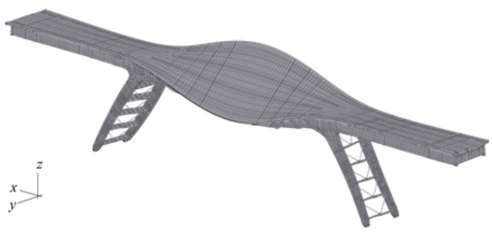


(b) Pier

Figure 6. Boundary conditions.



(a) 1st flexural vibration mode (f_{am} = 3.09 Hz)



(b) 1st torsional vibration mode (f_{am} = 5.11 Hz)

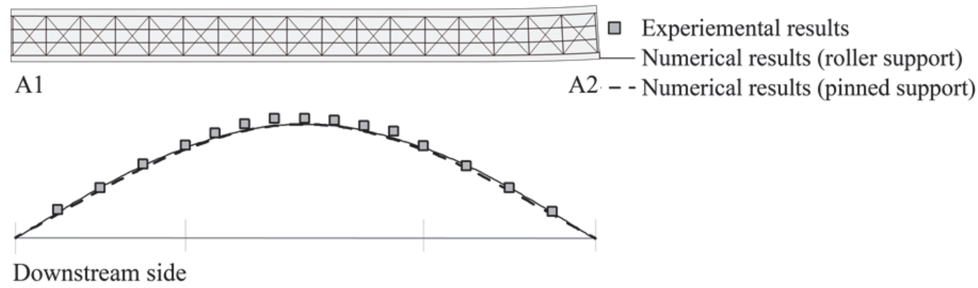
Figure 7. Examples of natural vibration mode.

modeled using shell elements, and shoes and cross beams were modeled using solid and beam elements, respectively. In total, about 80,000 nodal points and 73,000 elements were used for modeling the whole bridge. Two layers of concrete slab and asphalt were modeled using a single layer shell element, which has an axial stiffness and density equivalent to those of the two layers. Table 2 shows the list of material properties of steel, reinforced concrete, and asphalt for the numerical analysis.

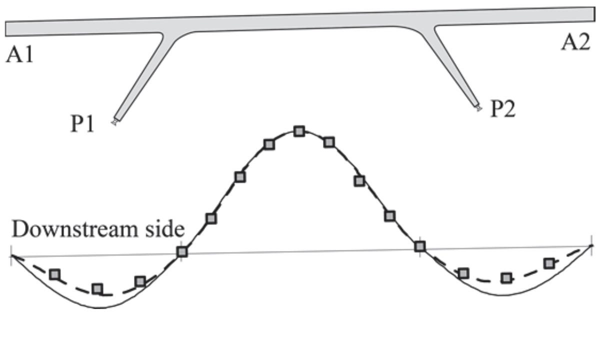
In this study, two kinds of FE analysis were carried out to investigate the effects of the support conditions at abutments A1 and A2 on the natural vibration characteristics: roller support following the design; and pinned support due to ageing deterioration. Piers P1 and P2 were assumed to be pinned irrespective of boundary conditions at the abutments (see, Fig. 6). Figure 7 shows examples of the typical natural vibration modes obtained from numerical analysis in the case of an assumed roller support. It is observed that each mode shape is very clear.

5 COMPARISON BETWEEN EXPERIMENTAL AND ANALYSIS RESULTS

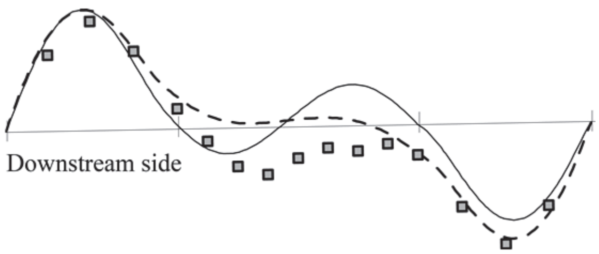
Table 1 lists the comparison of the natural vibration frequencies for three results obtained from the ambient vibration test and for two numerical analyses that assumed a roller support and a pinned support, respectively. Figure 8 also shows a comparison between three results for all vibration modes specified in this study. All vibration modes except the torsional vibration modes were specified using time histories of the acceleration measured along the downstream



(a) 1st horizontal vibration mode (f_e = 2.32-2.34 Hz, f_{am} = 1.87 Hz, f_{af} = 2.16 Hz)



(b) 1st flexural vibration mode (f_e = 3.13-3.15 Hz, f_{am} = 3.09 Hz, f_{af} = 3.24 Hz)



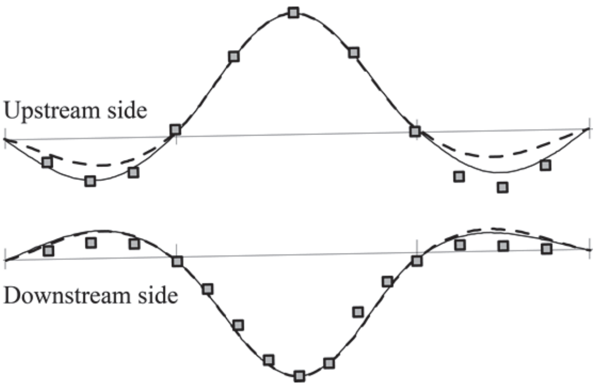
(c) 2nd flexural vibration mode (f_e = 4.66-4.76 Hz, f_{am} = 4.66 Hz, f_{af} = 3.75 Hz)

Figure 8. Comparison of the mode shapes for the numerical results and the experimental results.

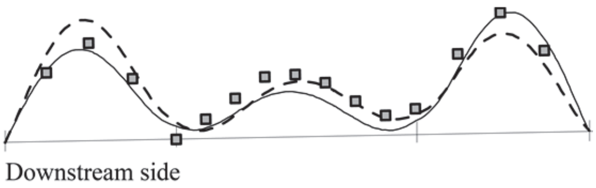
side. The 2nd torsional vibration mode in the case of the pinned support is not discussed in this study.

From Fig. 8a, it was confirmed that, although the natural vibration frequencies obtained from numerical results were smaller than those from the experimental results, the vibration modes were almost the same for both the experimental and the numerical results irrespective of the support conditions of the abutments.

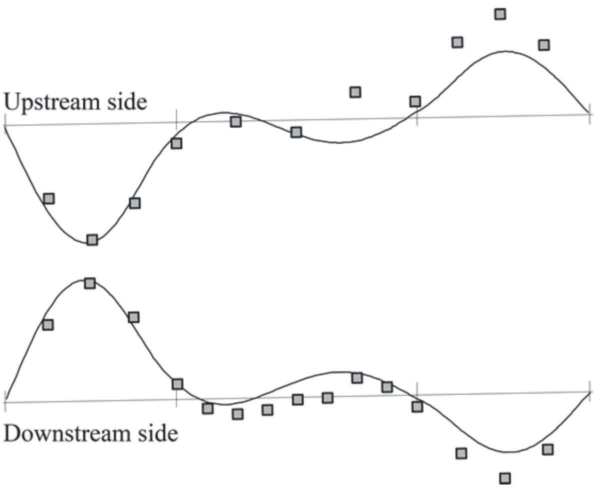
In the case of the 1st flexural vibration mode (see, Fig. 8b), the experimental results are in good agreement with the numerical ones assuming a pinned support condition. The natural frequencies obtained from the experimental results lie between the two numerical results obtained assuming pinned and roller supports. In the case of the 3rd flexural vibration mode (see, Fig 8e), it was confirmed that: 1) the modes obtained from the experimental results were very similar to those obtained from two numerical analyses; and 2) the natural frequencies were



(d) 1st torsional vibration mode (f_e = 4.98 Hz, f_{am} = 5.11 Hz, f_{af} = 5.15 Hz)



(e) 3rd flexural vibration mode (f_e = 5.32-5.40 Hz, f_{am} = 5.08 Hz, f_{af} = 5.53 Hz)



(f) 2nd torsional vibration mode (f_e = 7.25-7.45 Hz, f_{am} = 6.91 Hz)

Figure 8. Continued.

distributed between the results of the two numerical analyses. However, the 2nd flexural vibration mode (see, Fig. 8c) was a little different from them.

In the case of the torsional vibration mode, the mode shape of the experimental results for the 1st torsional vibration (see, Fig. 8d) was in good agreement with that of the numerical results assuming a roller support. However, it was difficult to obtain close agreement with the experimental and numerical results for the 2nd torsional vibration mode, because the amplitudes of the mode in the higher frequency region were very small.

Therefore, it was confirmed that natural vibration frequencies and modes, except for the horizontal vibration mode, obtained from ambient vibration test lie between the numerical analysis results obtained assuming pinned and roller supports.

6 CONCLUSIONS

In order to gain a basic knowledge for establishing rational seismic strengthening procedures for existing steel strutted-beam rigid frame bridge, a field vibration test and a 3D-FE analysis were conducted to investigate the natural vibration characteristics of the existing bridge. The bridge response was monitored under ambient vibration after the passing of unregulated traffic. The results obtained from this study were as follows:

(1) The natural vibration frequencies and modes of the bridge can be appropriately evaluated by conducting an ambient vibration test;

(2) It was found that the natural vibration frequencies and modes obtained from the ambient vibration test lie between numerical analysis results obtained assuming pinned and roller supports at the ends of the superstructure; and

(3) It was confirmed that the bridge is not damaged to the that it needs strengthening.

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