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DYNAMIC ASSESSMENT OF THE REDUCTION OF RIGIDITY OF AGED RC BRIDGES

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SUMMARY

This paper describes the experimental results and analytical results about the dynamic mechanical properties of two RC bridges: Unoki Bridge and Otobou Bridge, that had the age of 38 years and 53 years, respectively. These bridges had been damaged and deteriorated during their service period. Vibration tests by using vibration-exciting machine, impact load of falling steel ball, traffic vehicle and test vehicle were conducted on these bridges in order to assess the structural condition. The dynamic responses were measured by accelerometers and displacement meters placed on the deck of these bridges. Finite element analysis by single beam model and grillage model was executed. The theoretical natural frequency could be considered as the results of no occurrence of structural damage and material deterioration in these bridges. By comparing the results from experiment with the results from analysis, the reduction of rigidity of each bridge is deduced. It can be said that the monitoring techniques used in this study are suitable for the assessment of the structural condition of aged RC bridges.

INTRODUCTION

According to the annual statistics on roads in Japan (1994), 29 percent of all the road bridges are reinforced concrete bridges. For RC bridges, either or both of the two main components of RC bridges, concrete and reinforcement, can be affected by the environment to which they are exposed [Kong F. K, 1983]. Deterioration of concrete may be caused by a variety of physical and chemical processes, such as attack by acids, carbonation, or chloride penetration. This deterioration is not only harmful to concrete itself, but also leads to a more serious problem, that is corrosion of steel reinforcement. Steel in concrete is protected against corrosion by a thin oxide film on its surface that is formed due to the high alkaline environment in concrete. Penetration of chlorides and carbonation reduce the concrete alkalinity and cause the thin oxide film to lose its function, the corrosion of steel bars being accelerated eventually. That will lead to a reduction in sectional area of steel bars and ultimate disruption of concrete due to spalling of concrete cover.

On the other hand, load-carrying capacities of a lot of aged motorway bridges used in Japan are lower than that of bridges designed by current specification. With the increase of traffic density than expected and the enlargement of vehicle, maintenance and rehabilitation of reinforced concrete bridges, especially those which have been used for decades, become a very important issue for civil engineers.

As the results of material deterioration caused by chemical attacks, distress due to changing load requirements and natural forces, there would be a reduction in stiffness of RC bridges. A good understanding of the significance of defects to the structural integrity and future service is imperative. Conventional method for the assessment of aged RC bridges relies on visual inspections and material tests (destructive and nondestructive) that depend on the position at which the tests are conducted. There are many limitations for this method and a good representation of the global structural condition is usually time-consuming and expensive.

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The global testing of RC bridge can be achieved by indirect methods related to measurement of dynamic characteristics [Salawu and Williams, 1995]. In this study, dynamic load testing in which the vibration was generated by vibration-exciting machine, traffic vehicle, test vehicle, or falling steel ball were conducted on two RC bridge: Unoki Bridge and Otobou Bridge. Dynamic responses, such as natural frequency, damping coefficient of the bridge, were measured and calculated. The experimental results and analyzed results obtained under the condition that there were no structural damage and no material deterioration by finite element method of two models: single beam model and grillage model, are presented in this paper. By comparing the natural frequency from theoretical analysis with that from measurement, the reduction of rigidity can be deduced.

DESCRIPTION OF THE BRIDGES TESTED

The elevation and cross-section of Unoki Bridge, Otobou Bridge are shown in Figures 1a, 1b and 2, respectively. The basic information about the two RC bridges is given in Table 1.

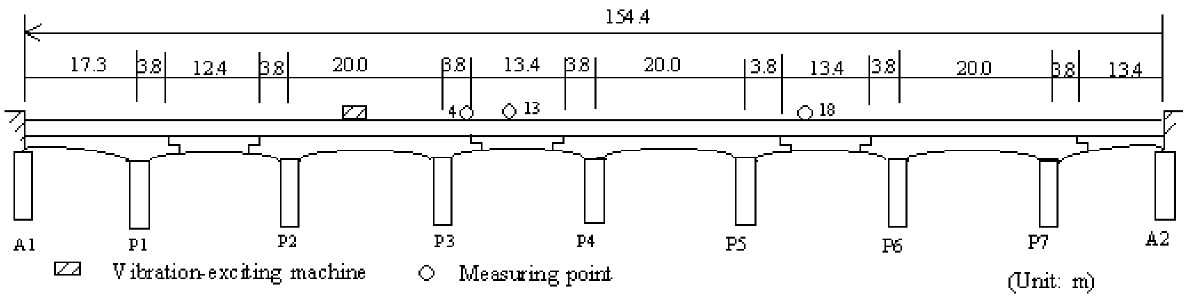


Figure 1a: Elevation of Unoki Bridge

Table 1: Basic information of Unoki Bridge, and Otobou Bridge

Bridge Name		Unoki Bridge	Otobou Bridge
Location		Kaeda, Miyazaki City, Japan	Miyakonojo City, Miyazaki Pref., Japan
Length		154.4 m (6@20.0+2@17.2)	137.5 m (5@21.0 +2@16.0)
Width		6.0 m	5.5 m
Completed Time		1938	1956
Age at testing time		53 years	38 years
Structure form	Main girder	RC cantilever 3 girders, T section	Simply support 3 girders, T section
	Slab	RC, 20 cm thick	RC, 29 cm thick
Guide fence		RC, height = 80 cm	RC, height = 70 cm

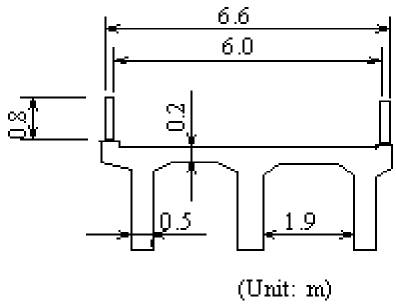


Figure 1b: Cross section of Unoki Bridge

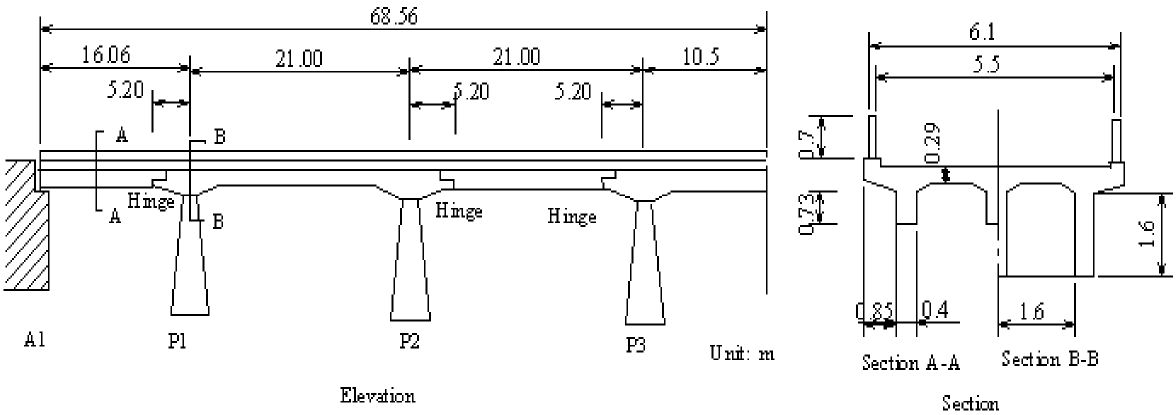


Figure 2: Elevation and cross section of Otobou Bridge

RESULTS ON UNOKI BRIDGE

Outline of experiment

The vibration test on Unoki Bridge was conducted in two steps: vibration caused by falling the back wheel of a truck in full-load state on the deck and caused by vibration-exciting machine. Result from the former step, which was a preliminary test for the second step, shown that the natural frequency of first mode was 4.41 Hz and the damping coefficient was 3 percent. This result was used to the selection of the vibration force generated by the vibration-exciting machine. This machine was a rotary disk attached with some pieces of eccentric counter weight and could generate vertical or horizontal vibration. The vibration force could be adjusted by increasing or decreasing the numbers of counter weight. Dynamic responses of the bridge, including acceleration and deformation, were measured by accelerometers and strain gage type displacement meter. Positions where the vibration-exciting machine and accelerometers were placed are illustrated in Figure 1. To get the natural frequency of the bridge, variation of vibration frequency of the machine at initial period was 0.2 Hz, then the variation was 0.1 Hz in a relative narrow range from 3.5 to 8 Hz. After the guide fence of the bridge was removed, a comparison test to measure the dynamic responses was also conducted for the understanding of the contribution of guide fence to the rigidity of the bridge.

Theoretical analysis

In order to assess the reduction of rigidity, experimental results were compared with the results obtained from theoretical analysis. Single beam model was used for the analysis. The discretization of the bridge is illustrated in Figure 3. There are 91 elements and 106 nodes. Because the bridge had been constructed 53 years ago, there almost no design materials of the bridge remained. Information of support conditions of the bridge was not clear, so two cases: Case 1 and Case 2 (Figure 4) were considered.

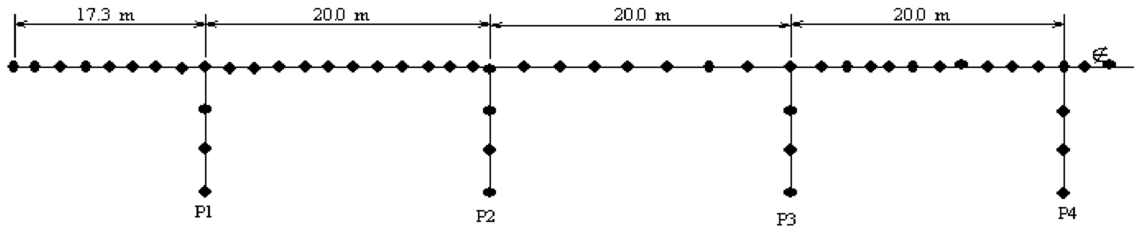


Figure 3: Discretization of Unoki Bridge by single beam model

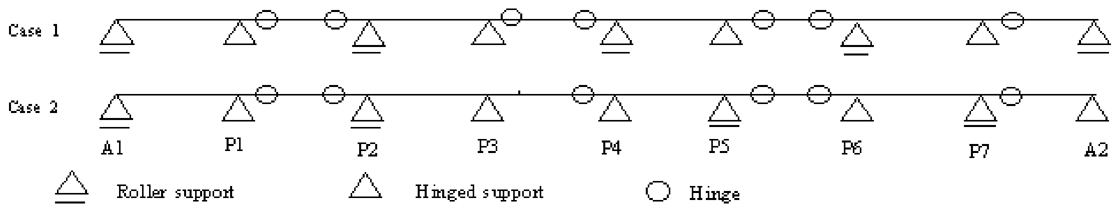


Figure 4: Support condition of Unoki Bridge

Results and discussions

Two instances of the response curves of acceleration and deformation at an equivalent constant vibration force of 29.4 KN in vertical vibration (measuring point 13 and 18 in Figure 1) are illustrated in Figure 5. From response curves, the natural frequency and damping coefficient of the bridge are calculated and are given in Table 2.

Table 2: Natural frequency and damping coefficient of Unoki Bridge

Vibration force	9.8 KN		29.4 KN		49.0 KN	
Mode No.	Natural frequency	Damping coefficient	Natural frequency	Damping coefficient	Natural frequency	Damping coefficient
1	4.30 Hz	2.36 %	4.25 Hz	3.35 %	4.20 Hz	3.43 %
2	4.60 Hz	--	4.50 Hz	--	4.40 Hz	--
3	4.90 Hz	--	4.70 Hz	--	4.70 Hz	--

From Table 2, it can be found that with the increase of vibration force, the natural frequency decreases. This can be considered as the influence of non-linear properties of reinforced concrete. The damping coefficient is obtained from the response curves by half-power method (Clough 1975). It should be pointed out that the damping coefficient given in Table 2 is not an exact value because the natural frequencies from mode 1 to mode 3 were too close that it was difficult to recognize influence of mode 2 on the damping coefficient in the case of mode 1.
Figure 6 shows the response curves of acceleration at the vibration force of 29.4 KN in vertical vibration (Point 4 in Figure 1) when the guide fence of the bridge was removed. The natural frequency obtained from the curves is about 3.6 Hz. The ratio of the natural frequency with and without guide fence is around 1.9, because frequency is proportional to $\sqrt{EIg/Wl^3}$, we can say that the guide fence has a contribution to the rigidity (EI) of Unoki Bridge from 30 to 40 percent. Experimental results on elastic modulus of concrete core specimen bored from Unoki Bridge shown the average elastic modulus was about 21,000 N/mm^2 [Nakazawa 1995]. Because the average compressive strength of concrete cores was 30 N/mm^2, which meant that the concrete should have a elastic modulus of 26000 N/mm^2, the decrease of elastic modulus was caused by the deterioration of concrete during the past service period. As the deformation mode from theoretical analysis and experiment almost has the same shape, the theoretical deformation mode is used to calculate the damping coefficient by modal analysis method. Table 3 and Table 4 list the analytical results on natural frequency and damping coefficient for the two cases in Figure 4 by single beam model, respectively.

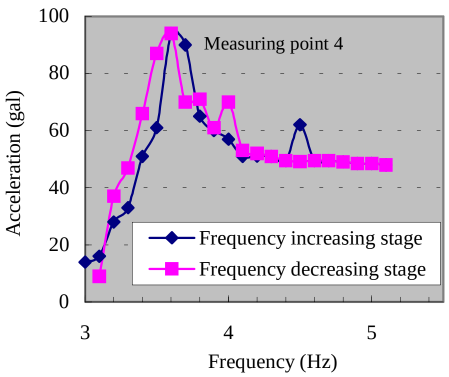


Figure 6: Responses of acceleration without guide fence

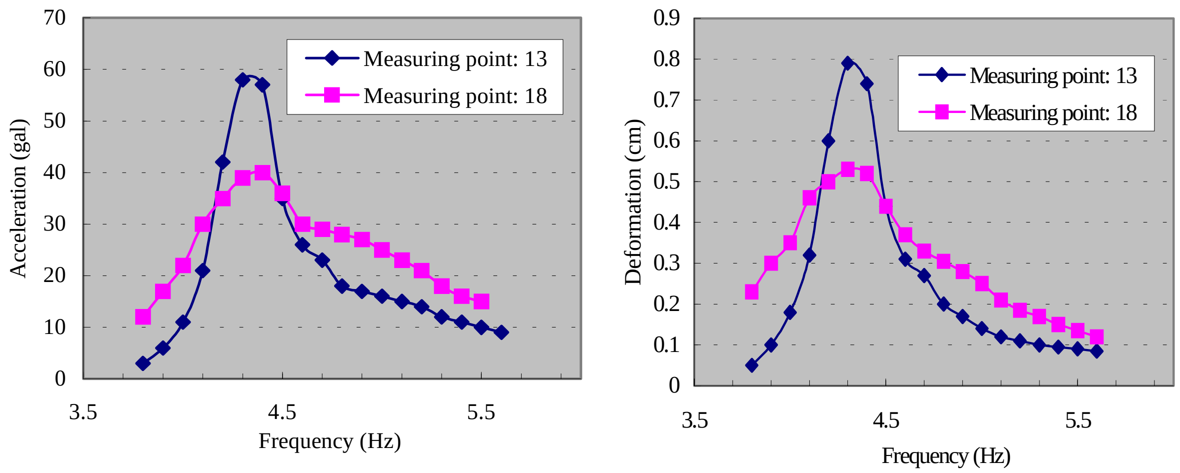


Figure 5: Responses of acceleration and deformation at an equivalent vibration force of 29.4 KN

It can be found from Table 3 that when the elastic modulus of 21000 N/mm^2 is taken in the calculation, the theoretical natural frequencies in both cases are similar to the experiments. Whereas when the elastic modulus is 26000 N/mm^2, the experimental results is about 10 percent lower than the theoretical natural frequency. The reduction of rigidity of Unoki Bridge can be deduced as around 10 percent. Usually for a newly built bridge, with the increase of vibration force, the variation of damping coefficient is far smaller than the variation in Unoki Bridge. This can be considered as one of the important characters of aged RC bridges.

Table 3: Theoretical natural frequency (Hz)

Mode No.	Analysis				Experiment
	Ec = 21000 N/mm^2		Ec = 26000 N/mm^2		(f = 49KN)
	Case 1	Case 2	Case 1	Case 2	
1	4.53	4.36	5.04	4.81	4.20
2	4.69	4.68	5.22	5.21	4.40
3	4.93	4.90	5.48	5.44	4.70
4		5.01		5.53	
5	6.53	6.53			

Table 4: Theoretical damping coefficient (%)

Vibration force (KN)		Analysis						Experiment		
		Case 1			Case 2					
		9.8	29.4	49.0	9.8	29.4	49.0	9.8	29.4	49.0
Mode No.	1	0.84	1.45	1.51	0.80	1.40	1.46	2.36	3.35	3.43
	2	2.45	4.06	6.39	2.45	3.90	4.73			
	3	1.81	2.75	4.44	1.91	2.90	5.25			

RESULTS ON OTOBOU BRIDGE

Experimental procedure

Vibration was generated by traffic vehicle, test vehicle, and impact load of falling steel ball, respectively. Strain gage type accelerometers with maximum sensitivity of 2 G utilized to measure the dynamic responses during vibration tests were placed on the deck of the bridge (Figure 7). The dynamic responses were measured at an interval of 5 milli-seconds and lasted for 10 seconds. Test by impact load was conducted by falling steel ball of 160-kg from a height of 50 cm.

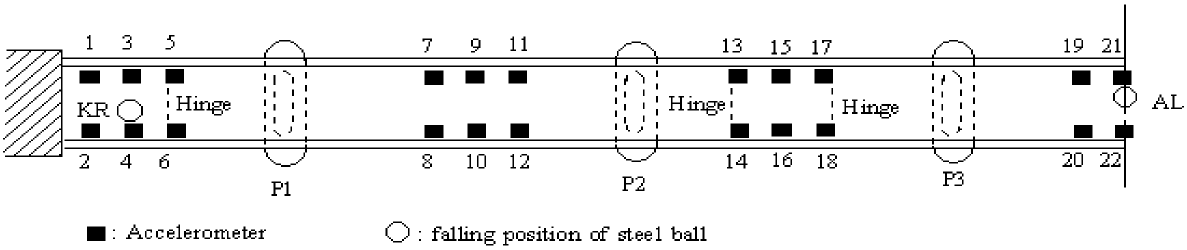


Figure 7: Position of accelerometer and position of falling steel ball

Theoretical Analysis

Finite element method of two models: single beam model and grillage model was used for the theoretical analysis. Both of the models were constituted from bar element and the whole bridge was subdivided into straight forms of members. The external load must be applied at nodes in finite element analysis, because the front axle load and/or the back axle load were not at the nodes so the load in bar element was redistributed to its two nodes. The discretization of Otobou Bridge by grillage model is illustrated in Figure 8 (numerals in it show the element number). There are 222 elements, 177 nodes, and 24 restraints. It was considered that only the deflection was restrained at support nodes and the deflection and torsional angle were compatible at hinge points. The length of each element, cross sectional area, principal moment of inertia, and torsional rigidity are given in Table 5. The discretization of Otobou Bridge by single beam model is illustrated in Figure 9. There are 52 elements, 59 nodes, and 16 restraints. The length of each element, cross sectional area, principal moment of inertia, and torsional rigidity are given in Table 6.

Results and discussions

Measured waves of acceleration responses were analyzed by fast Fourier transform (FFT) to obtain the spectral density. Figures 10, 11 and 12 show the power spectral density of some measuring points obtained in the case of traffic vehicles, test vehicle, and falling steel ball, respectively. Figures 13 and 14 illustrate the vibration mode shape from experiment and the mode shape from theoretical analysis by grillage model, respectively.

Table 7 lists the natural frequency obtained from experiment and from theoretical analysis by two models. It can be found that the analyzed natural frequency by grillage model was a little smaller than that from single beam model, because the effect of cross beam was ignored in the analysis by single beam model.

Considering that the analytical result by grillage model was the natural frequency of no structural damage, rigidity ratio ($\{(c)/(a)\}^2$ in Table 7), which was the square of the ratio of measured natural frequency to theoretical natural frequency, could be used to indicate the reduction of rigidity caused by structural damage. It can be found that the reductions of rigidity were about 3 to 12 percent. The flexural vibration in which the phase of each girder was the same represents the average evaluation of the rigidity of three main girders and the flexural vibration in which the phase of each girder was different represents the rigidity of two side girders.

Table 7: Natural frequency of Otobou Bridge

Mode No.	Analysis		(c) Exp.	$\{(c)/(a)\}^2$
	(a) Grillage	(b) Beam		
1	2.25	2.65	--	--
2	2.33	2.76	2.73	1.37
3	2.42	2.90	3.13	1.67
4	5.73	6.66	--	--
5	6.20	7.32	6.03	0.95
6	6.80	--	6.58	0.94
7	6.82	8.32	--	--
8	7.24	--	7.03	0.94
9	7.81	--	7.33	0.88
10	9.34	--	--	--
11	9.60	10.87	--	--
12	10.32	12.08	10.16	0.97
13	10.39	--	--	--
14	11.51	13.94	--	--
15	11.70	--	11.33	0.94

Table 5: Geometrical properties of element (grillage model)

No.	L (m)	A (m^2)	I_x (m^4)	$J*10^{-3}$ (m^4)
1	1.60	0.964	0.063	0.241
3	2.71	0.946	0.111	0.265
4	2.71	0.892	0.078	0.261
9	1.60	0.793	0.044	0.253
17	1.60	0.782	0.067	0.277
19	1.60	1.009	0.066	0.262
21	2.60	1.033	0.186	0.310
22	2.60	0.979	0.145	0.306
24	2.60	1.207	0.385	0.402
25	2.60	1.153	0.329	0.398
27	1.60	1.828	0.314	0.461
35	1.60	1.668	0.051	0.442
37	2.65	0.946	0.111	0.265
38	2.65	0.892	0.078	0.261
43	1.60	1.683	0.051	0.446
69	1.60	1.024	0.066	0.266
77	1.60	0.798	0.044	0.233

Table 6: Geometrical properties of element (single beam model)

No.	L (m)	A (m^2)	Ix (m^4)	$J*10^{-3}$ (m^4)
1	2.71	2.785	0.301	0.790
5	2.60	3.046	0.518	0.926
6	2.60	3.568	1.099	1.201
9	2.65	2.785	0.301	0.790

Total number of elements = 222

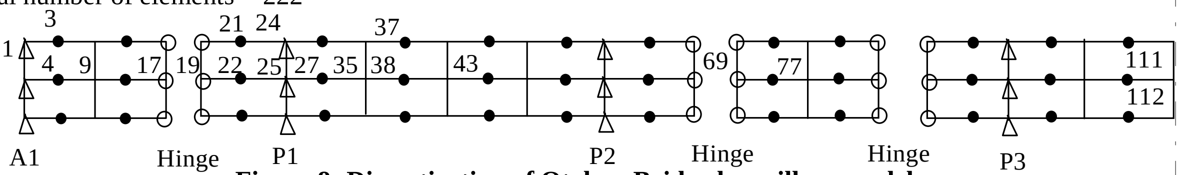


Figure 8: Discretization of Otobou Bridge by grillage model

Total number of elements = 52

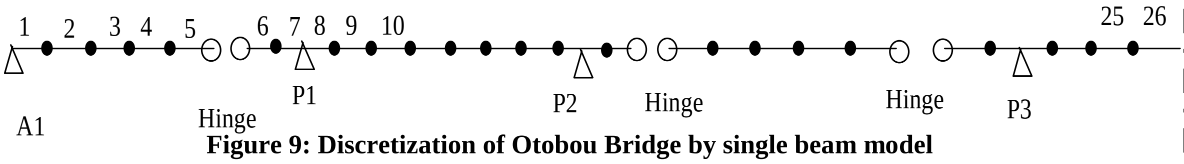
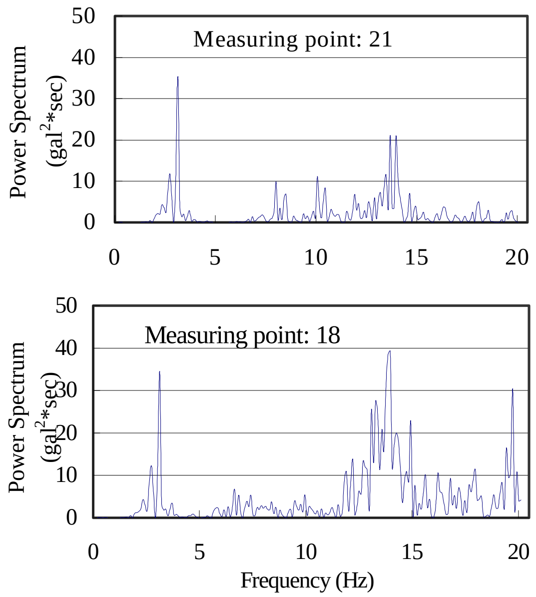


Figure 9: Discretization of Otobou Bridge by single beam model



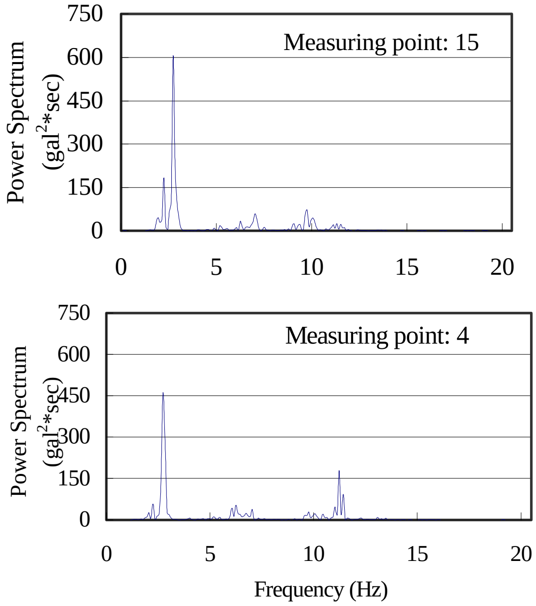


Figure 10: Power Spectral density for traffic vehicles **Figure 11: Power Spectral density for test vehicle**

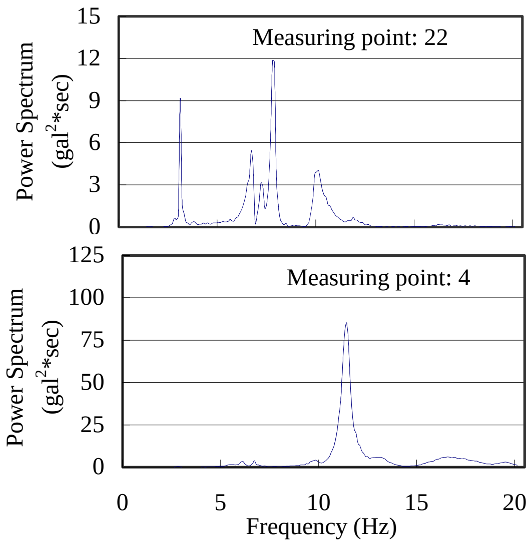


Figure 12: Power spectral density for falling steel ball

From Table 7 it can be found that the reduction of rigidity associated with flexural vibration in which the phase of each girder was the same was about 3 to 5 percent. Whereas the reduction of rigidity associated with flexural vibration in which the phase of each girder was different was about 6 to 12 percent. It can be deduced that damage of the side girders was larger than that of the central girder. This deduction was confirmed by the crack distribution in these girders that there were more cracks in the side girders than in the central girder. There exists no bending restraint for pin connection and roller support, but the support utilized in Otobou Bridge was plate surface bearing support and therefore there were bending restraints in its supports. The practical reductions of rigidity should be higher than the results in Table 7 because of the effect of bending restraint in the supports.

CONCLUSIONS

1) One of the important characters of aged RC bridges is that with the increase of vibration force, the variation of damping coefficient is much larger than the variation of a newly built bridge.
2) The reduction of rigidity in Unoki Bridge is deduced as around 10 percent. For Otobou Bridge, reduction of rigidity associated with flexural vibration with same phase in each girder was about 3 to 5 percent, whereas the reduction of rigidity associated with flexural vibration with different phase in each girder was about 6 to 12 percent. This was because the former represented the average rigidity of three main girders and the latter represented the rigidity of two side girders. It can be deduced that the damage of side girders was larger than that of the central girder.
3) Guide fence in Unoki Bridge has a large contribution to the rigidity of the bridge.

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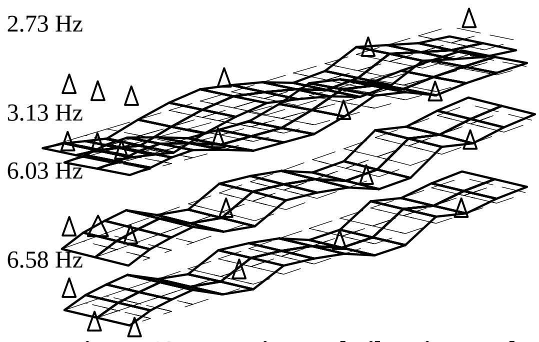

Figure 13: Experimental vibration mode

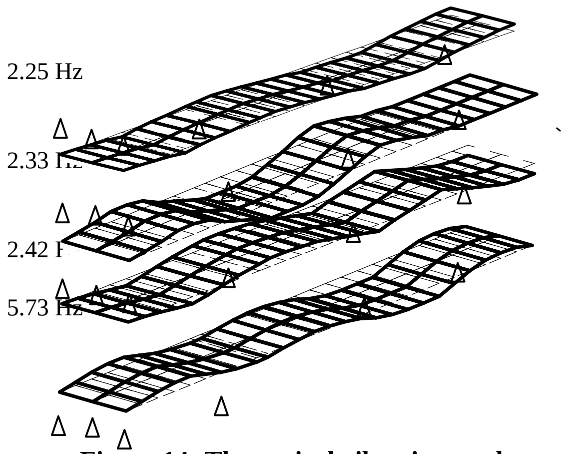

Figure 14: Theoretical vibration mode