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Sensitivity of Mechanical Behaviors in Damaged Bridges and Its Application to SHM

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ABSTRACT: Structural health monitoring (SHM) using information technology and sensors is increasingly being used for providing a better estimate of structural performance characteristics rather than traditional methods. Firstly experiments were carried out to assess the sensitivity of the dynamic behavior of artificially damaged model bridges for the purposes of damage assessment. A method of damage assessment of bridges based on these behaviors is then discussed in detail. Finally, on the basis of the results, a possible application to a structural health monitoring system for existing bridges is also proposed. As a result, considering the damage assessment method based on the changes of the natural frequency, we could estimate the damage location of the bridge model by using the IF-THEN-ELSE rules

1 INTRODUCTION

Visual inspection is the most common method for assessing a bridges condition. However, in the inspection method, the assessment result is subjective. Structural health monitoring (SHM) using information technology and sensors is increasingly being used for providing a better estimate of structural performance characteristics rather than traditional methods [1]. Because the mechanical behavior of bridges with degrees of various damages is not clear, it is very difficult to exactly assess the mode and degree of damage for existing bridges. Firstly experiments were carried out to assess the sensitivity of the dynamic behavior of artificially damaged model bridges for the purposes of damage assessment. A method of damage assessment of bridges based on these behaviors is then discussed in detail. Finally, on the basis of the results, a possible application to a structural health monitoring system for existing bridges is also proposed.

2 BRIDGE MODEL TEST AND ANALYSIS

2.1 Outline

The effects of damage level on mechanical behavior were analyzed from the experimental results using the bridge model tests and a model simulation [2]. Then, on the basis of these results, damage levels and the accuracy of assessments were examined and the effectiveness of utilizing mechanical behavior for damage assessment was considered. However, it was not possible to examine all damage patterns because there are many kinds of structural conditions and damages in existing bridges. Therefore, the model was made as a simply supported girder bridge with three main girders which is the minimum required to obtain the load distribution characteristics perpendicular to the bridge span. Moreover, in this experiment, the reduction of stiffness of the main girder in bending was chosen as being representative of a common causes of damage in real bridges (e.g. cross section deficiency due to the corrosion in a steel bridge, looseness of joints, stiffness reduction due to cracks in concrete bridges, the deterioration of the concrete, etc).

2.2 Model girder

The details of the bridge model are shown in Figure 1. In order to approximate the general natural frequency (within 10Hz) of a real bridge, ready-made H type steels (H-100×50×5×7.5mm (SS400)) were utilized as main girders. The method of damage (stiffness reduction) in main girder is shown in Table 1(a). Six main girders were prepared with 20% and 40% stiffness reduction at L/2 or L/4 or overall. The stiffness reduction ratios were chosen because model calculations and experimental results for a simple beam show mechanical behavior starts changing at a 20% reduction ratio, for example 20~30% stiffness reduction occurs due to corrosion in real bridges, loss of the function of the shear connector of a composite girder causes 50% or more stiffness reduction. However, this hardly ever happens, so a 40% stiffness reduction was chosen. It is possible to obtain the mechanical behavior trends by using 2 step reduction ratios. Moreover, the stiffness of crossbeams was determined so that the ratio of crossbeam stiffness to main girder stiffness was close to that of a real bridge. For this purpose, T-beams were used to model load distribution crossbeams and sway bracing as shown in Table 1(b). A total of three types of load distribution crossbeam were prepared with differing stiffness.

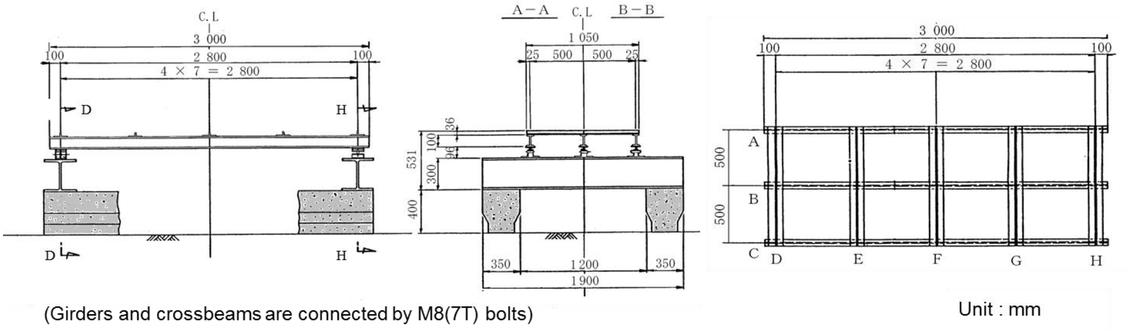


Figure 1. Details of the bridge model

Table 1. Specification of main girders and crossbeams

(a) MAIN GIRDER

No.	Damage location (cm)	Correspondence damage in real bridge	Ratio to stiffness of No.1
1	No damage	–	1.00
2	30 220 30	Main Girder Corrosion, composite girder shear connector damage, RC girder cracking	0.79
3	Overall stiffness reduction		0.60
4	119 42 119	Girder damage such as local corrosion, joint damage, etc	0.79
5	Local (L/2) stiffness reduction		0.60
6	89 42 189	Girder damage such as local corrosion, joint damage, etc	0.79
7	Local (L/4) stiffness reduction		0.60

(b) CROSSBEAM

No.	Description	Ratio to stiffness of main girder
1	Corresponding to a load distribution crossbeam of a real bridge	0.14
2	Corresponding to sway bracing of a real bridge	0.066
3	Near-zero stiffness	2.8×10^{-4}

First, dynamic tests were carried out with 7 types of main girder including the undamaged type. The prescribed stiffness of each girder was checked from their resonant frequencies. Experiments were carried out on 34 combinations of 7 main girders and 3 crossbeams (see Table 2). These 34 cases can be classified into 4 series as follows.

Table 2. Combinations in the experiments

Series No.	Exp. No.	Main girder			Crossbeam				
		A	B	C	D	E	F	G	H
1	1	1	1	1	2		1		2
	2	1	2	1	2		1		2
	3	1	3	1	2		1		2
	4	1	5	1	2		1		2
	5	1	7	1	2		1		2
	6	1	1	2	2		1		2
	7	1	1	3	2		1		2
	8	1	1	4	2		1		2
	9	1	1	5	2		1		2
	10	1	1	6	2		1		2
	11	1	1	7	2		1		2
	12	2	1	2	2		1		2
	13	2	2	2	2		1		2
	14	2	1	3	2		1		2
	15	2	3	2	2		1		2
2	1	1	1	1	2		2		2
	3	1	3	1	2		2		2
	4	1	5	1	2		2		2
	5	1	7	1	2		2		2
	7	1	1	3	2		2		2
	9	1	1	5	2		2		2
	11	1	1	7	2		2		2
	14	3	1	3	2		2		2
	16	3	3	3	2		2		2
3	17	1	1	1	2	2	2	2	2
	18	1	1	1	2	3	2	3	2
	19	1	1	1	3	3	2	3	3
	20	1	1	1	3	3	3	3	3
	21	1	1	1	1		1		1
	22	1	1	1	2		1		2
	23	1	1	1	2		2		2
4	A	1	1	1	2		1		2
	B	1	1	1	2		1		2

Note : ・For the girder symbols shown in the table, see Fig.1.
・The numbers shown in the table indicate the girder numbers (see Table.1).
・A blank space means the absence of a crossbeam.

(1) Series 1 (15 cases)

The influence of the stiffness reduction of the main girders on the mechanical behavior was examined, without changing the crossbeam.

(2) Series 2 (9 cases)

In this series, the crossbeam in the center of the span (girder F) was similar to sway bracing for a real bridge with approximately half the stiffness of crossbeam No.1 (Table 2). In other words, the influence of the stiffness reduction of the crossbeam on the mechanical behavior of the main girder was examined.

(3) Series 3 (7 cases)

Different combinations of crossbeams were used with undamaged main girders. In other words, the changes in mechanical behavior caused by stiffness reductions in directions perpendicular to the bridge axis were examined.

(4) Series 4 (2 cases)

By removing the movable support and assuming free ends, the influence of the support settlement was examined with undamaged main girders and crossbeams.

2.3 Experiment method

The sketch of the dynamic test is shown in Figure 2(a). In this experiment, in order to apply modal analysis [3][4][5], accelerometers were placed at 11 observation points and single blows were applied at 4 points. In order to raise the precision of the transfer function, the blows were applied 20 times at each point.

2.4 Model calculation

The mechanical behavior was analyzed to check the precision of the analysis model for application to real bridges. The dynamic analytic model as shown in Figure 2(b) is the lumped mass model which divided the main girders into 20 elements and the crossbeams into 8 elements. The natural frequency and mode of vibration were calculated from the transmission matrix for the lattice girder with torsional stiffness.

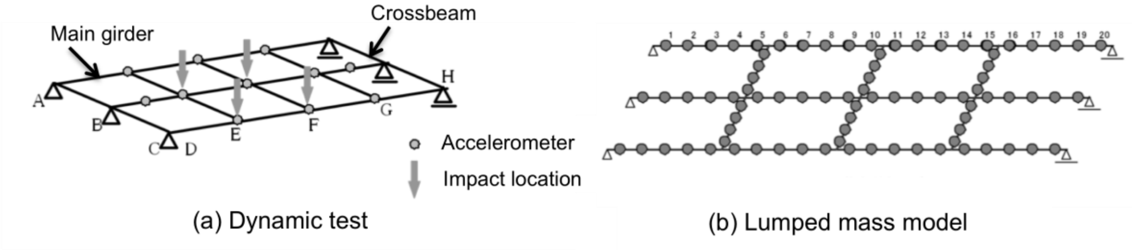


Figure 2. Dynamic test and analytical model

3 DAMAGE ASSESSMENT WITH MECHANICAL BEHAVIORS

3.1 Characteristics of each dynamic behavior

3.1.1 Natural frequency

The target orders and modes of vibration in this study were 5 frequencies of 1-1st, 1-2nd, 1-3rd, 2-1st and 3-1st, as shown in Figure 3. The main conclusions obtained can be summarized as follows:

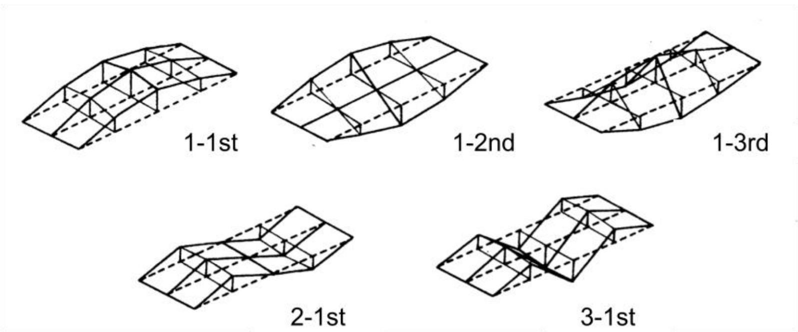


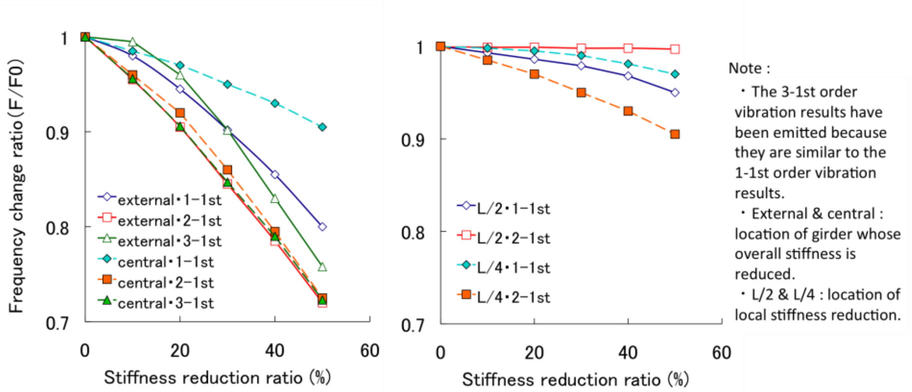
Figure 3. Target orders and modes of vibration

(1) When the overall stiffness reduction occurred in the external girder, the changes of 1-1st, 1-2nd and 1-3rd frequencies were similar. When the overall stiffness reduction occurred in the central girder, the change of the 1-1st frequency was small in comparison with other vibrating orders. These tendencies are in reasonable agreement with the analytical results (see Figure 4(a) and Figure 5(a)).

(2) When the overall stiffness of the main girder was reduced, the change of each frequency was bigger compared to the local stiffness reduction (Refer to Figure 4(a), (b) and Figure 5).

(3) From the experimental results for the frequency changes due to the location of the stiffness reduction, the largest influence for the 1-1st frequency was at the L/2 section, and the most

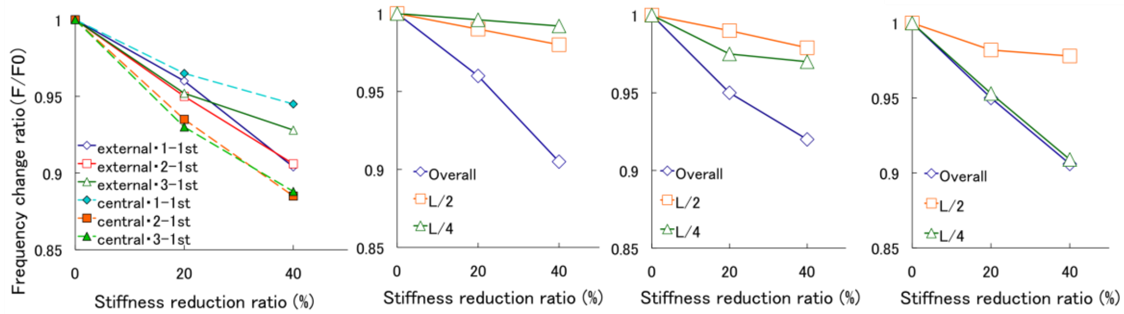
influence for the 2-1st and 3-1st frequency was at the L/4 section. It is thought that each peak and node of the vibration mode and the location are related. Therefore, when the local stiffness reduction occurs at the position for the peak of the vibration mode, a big change of the frequency appears. The experimental results and tendencies for the 1-1st and 2-1st frequencies were in reasonable agreement with the analytical results. However, that of the 3-1st frequency differed from the analytical results. It is thought that the influence of the stiffness reduction of L/2 became small, because the effect of the crossbeam was large in the experiments (see Figure 4(b) and Figure 5(d)).



(a) Comparison in terms of location of girder whose overall stiffness is reduced

(b) Comparison in terms of location of local stiffness reduction

Figure 4. Relationship between the stiffness change ratio and the frequency change ratio (analytical results)



(a) Overall stiffness reduction

(b) Comparison in terms of location of stiffness reduction (1-1st)

(c) Comparison in terms of location of stiffness reduction (2-1st)

(d) Comparison in terms of location of stiffness reduction (3-1st)

Figure 5. Relationship between degree of damage and frequency changes (experimental results)

Figure 5 shows the experimental results of Series No.1 (see 2). In this series, the stiffness of the crossbeam is larger than the others. In order to explain the influence of the crossbeam stiffness reduction, Table 3 shows the comparison with Series No.2 (crossbeam stiffness reduction). This table shows that the influence for the 3-1st frequency does not depend on the damage location (L/2, L/4). When the frequency becomes higher-order such as 3-1st, there is a possibility that experiment results vary widely.

Table 3. Frequency change ratio for cases of local stiffness reduction

Damage location	Series No.	Order of vibration		
		1－1 st	2－1 st	3－1 st
L／2	1	3．2	2．3	2．3
	2	3．8	0．7	3．4
L／4	1	1．6	9．2	3．1
	2	1．1	5．7	3．0

Unit：%

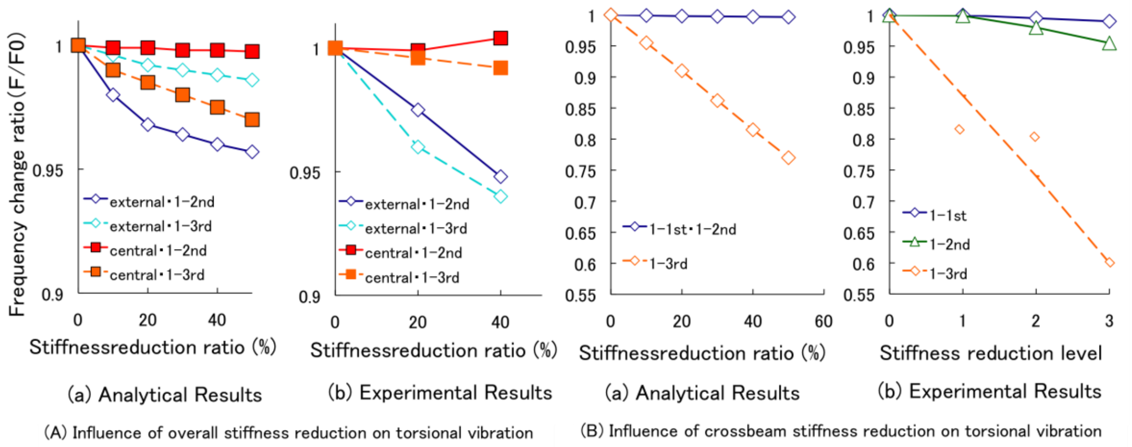


Figure 6. Influence of stiffness reduction on torsional vibration

Figure 6(A) shows the analytical and the experimental results for changes in the torsional vibration when the overall stiffness reduction occurred on the central or the external girder. Moreover, Figure 6(B) shows the analytical and the experimental results of changes of the torsional vibration when the crossbeam stiffness reduction changed (Series No.3). The figures indicate that the higher-order torsional vibration changes when damage occurs which changes the balance of stiffness in a direction perpendicular to the bridge axis, and the crossbeam stiffness reduces. As mentioned above, the tendency of the natural frequencies obtained from the experiments shows relatively good agreement with the analytical results. It has also been found that the results other than the frequencies of higher order vibration such as the 3-1st order vibration show low variability and high repeatability, and the influence of small changes in stiffness can be observed clearly.

3.1.2 Vibration mode and amplitude

Figure 7 shows the changes in the orders of vibration obtained in the experiments. Here, the changes in the orders of vibration in which L/2 of girder B is designated as the reference level (1.0) for the standard vibration mode are represented. The main conclusions obtained can be summarized as follows:

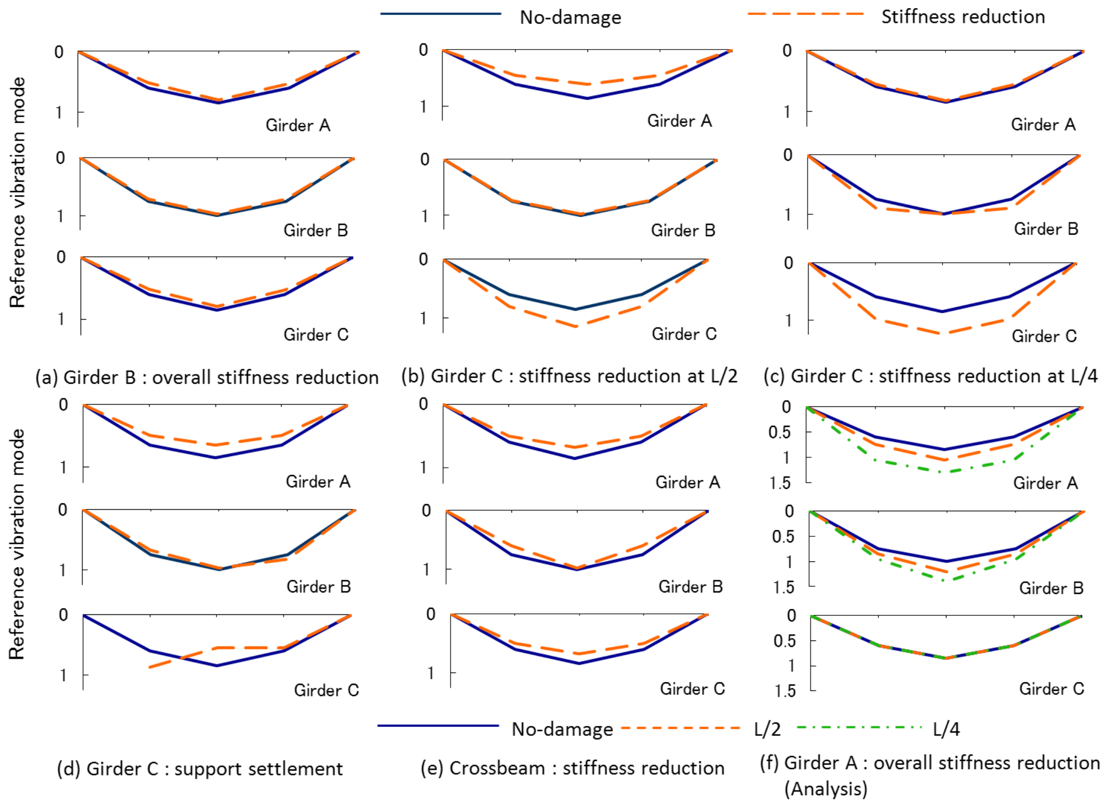


Figure 7. Changes in vibration mode (1-1st) (experimental results)

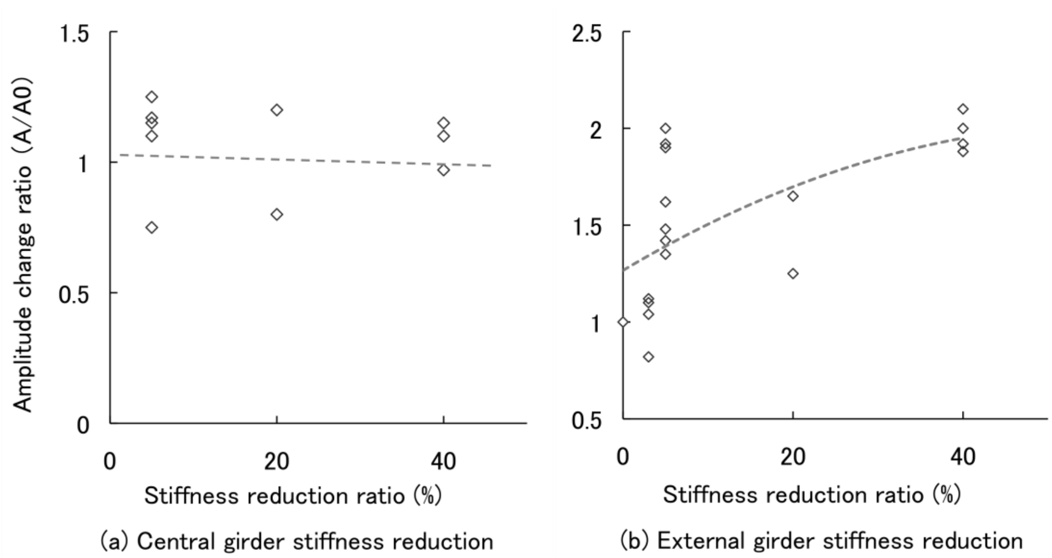


Figure 8. Influence of overall stiffness reduction on amplitude change ratio

(1) When the damage (stiffness reduction) occurs in the central girder (girder B), the vibration mode almost never changes compared with the undamaged condition (see Figure 7(b)).

(2) When the damage occurs in the external girder (girder A and C) even in case of the local damage, big changes appear in the vibration mode compared with the undamaged condition. In addition, the change of the mode of vibration shows a difference due to the stiffness reduction in the L/2 or L/4 section (see Figure 7(b), (c)).

(3) When damage occurs due to the support settlement, the distribution of the bridge axis direction of the mode of vibration changes greatly in the girder whose support is damaged (see Figure 7(d)).

(4) When the damage occurs in the crossbeam, the amplitude of the external girder decreases due to the reduction of the load distribution effect (see Figure 7(e)).

Figure 8 shows the amplitude change ratio of the overall stiffness reduction in the external and central girders. The stiffness reduction of the central girder almost never affects the amplitude change ratio. However, the stiffness reduction of the external girder greatly affects the amplitude change ratio.

3.1.3 Damping coefficient

The primary factors which affect the damping of the vibration are many, and appear as scatter in the experimental results. Figure 9 shows the relationship between the stiffness reduction ratio for all girders and the damping coefficient. If a stiffness reduction of a certain value does not occur, the damping coefficient does not change. When the damping of the vibration changes greatly, it is thought that there is a greater possibility for damage in the bridge.

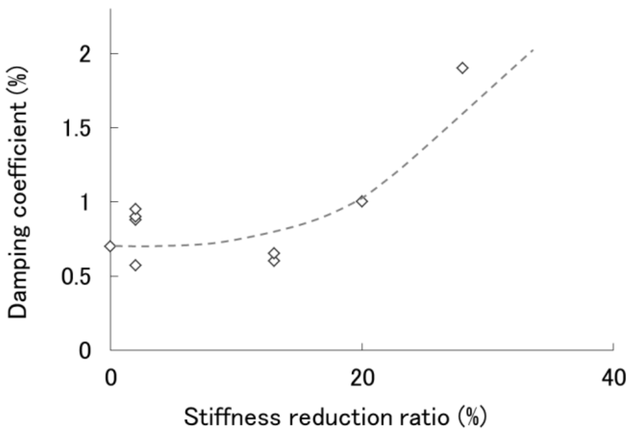


Figure 9. Relationship between stiffness reduction and damping coefficient

3.2 Summary

Table 4 summarizes the general results for dynamic behavior changes. The change in dynamic behavior is generally, sensitive to damage (stiffness reduction) occurring in the external girder.

Table 4. Changes in mechanical behavior due to stiffness reduction

Damage		Dynamic behavior			
Location	Description	Frequency 1-1st, 2-1st, 3-1st	Torsional vibration frequency, 1-2nd, 1-3rd	Mode shape	Damping Coefficient
Central girder	Local stiffness reduction	The 1-1st order reduction ratio is high in case of L/2 damage; the 1-2nd order reduction ratio is high in case of L/4 damage.	The 1-3rd frequency decreased, and the 1-2nd frequency did not change.	Almost no change	Highly variable
	Overall stiffness reduction	The 1-1st order reductio ratio lower than 2-1st and 3-1st order reduction ratios.			Frequency to increase at a reduction ratio of 40%
External girder	Local stiffness reduction	The 1-1st order reduction ratio is high in case of L/2 damage ; the 1-2nd order reduction ratio is high in case of L/4 damage.	Both decreased : the 1-3rd order reduction ratio is higher.	Change at 20% : great influence in case of L/4 damage.	Highly variable
	Overall stiffness reduction	The reduction ratios of three frequencies are the same.		Considerable change	Tendency to increase at a reduction retio of 40%
Crossbeam	Stiffness reduction	Almost no change	Considerable decrease in 1-3rd	Central girder amplitude greater than external girder amplitude.	Highly variable
Support	Settlement	Slight decrease	Slight decrease	Changes in distribution in bridge axis direction.	Highly variable

3.3 Evaluation of usability of mechanical behavior in damage assessment

A damage assessment method is proposed by considering the experimental results. Figure 10(a) shows the mechanical behavior changes caused by damage to the main girder and crossbeam from the experiments. The changes in mechanical behavior resulting from girder and crossbeam damage (stiffness reduction) that have been identified through the model experiments and analysis are shown in a simplified form in Figure 10(a). The characteristics of these changes in mechanical behavior are categorized, compared and illustrated in Figure 10(b). These mechanical behavior characteristics can be summarized as follows:

(1) The frequency is a parameter of medium level sensitivity for assessing the damage among the dynamic behaviors which are considered in these experiments. However, the lower-order frequency accuracy is good. So by measuring the higher-order torsional and bending vibration, there is the possibility of assessing local damage, however it is thought that damage assessment is better resolved through examination of the other dynamic behaviors.

(2) The damping coefficient varies widely, and the change is insensitive to slight damage. However, if a large damping coefficient change appears, serious damage can be detected.

(3) The change of amplitude and mode of vibration is very different in the case of the external girder being damaged and the case of the central girder being damaged. In other words, the mode of vibration does not change in the case of the central girder damage which does not upset the stiffness balance in the direction perpendicular to the bridge axis. However, the mode of vibration changes greatly in the case of external girder damage which upsets the stiffness balance in the direction perpendicular to the bridge axis.

From the results of the experiments above, we have proposed a damage assessment method based on each natural frequency. Firstly, the damage assessment flow chart was drawn with the natural frequencies obtained from the analytical and experimental results and the effectiveness of this damage assessment was considered.

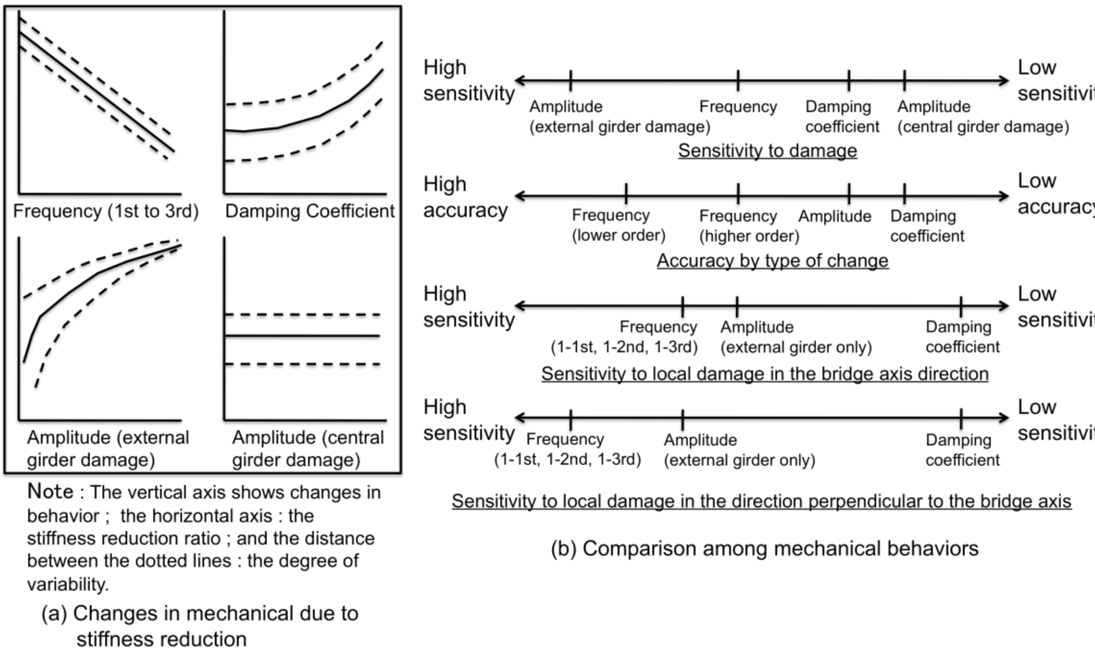


Figure 10. Summarize of the sensitivity of each dynamic behavior with stiffness reduction

The tendencies for each order of frequency which were obtained can be classified into several categories. By using this, a damage location presumption flow chart was drawn up as in Figure 11. The flow chart is based on IF-THEN-ELSE rules. On the way, the damage assessment separates into the direction perpendicular to the bridge axis due to the torsional vibration and the axial direction due to the bending vibration. Finally, the damage locations can be estimated from these results.

After the presumption of the damage location with Figure 11, we consider the probable degree of damage. Firstly, the relationship between the 1-1st order frequency and the stiffness reduction at the location from Figure 11 is estimated from the analysis and the bridge model test. It is assumed that the stiffness reduction and the change of the lower-order natural frequency are linear relations. Next, the degree of damage is estimated from a linear relation. The overall flow chart for this damage assessment method is shown in Figure 12. An example of the damage assessment which was estimated from the model test results with this flow is shown in Table 5.

In the case of damage assessment for real bridges, we have to estimate using analytical results, because it is difficult to make models of each bridge. Table 5 shows that the analytical and experimental results are very close. For this reason, it is thought that the accuracy of the estimated frequencies is very good, and the lower-order frequencies which are easy to match with theoretical values were obtained. There are no data for real bridges in a undamaged condition. Therefore we have to estimate the undamaged data by analysis. However, it is thought that the damage could be estimated from data obtained with damage progress, because this estimation method assumes that the change of the frequency is linear.

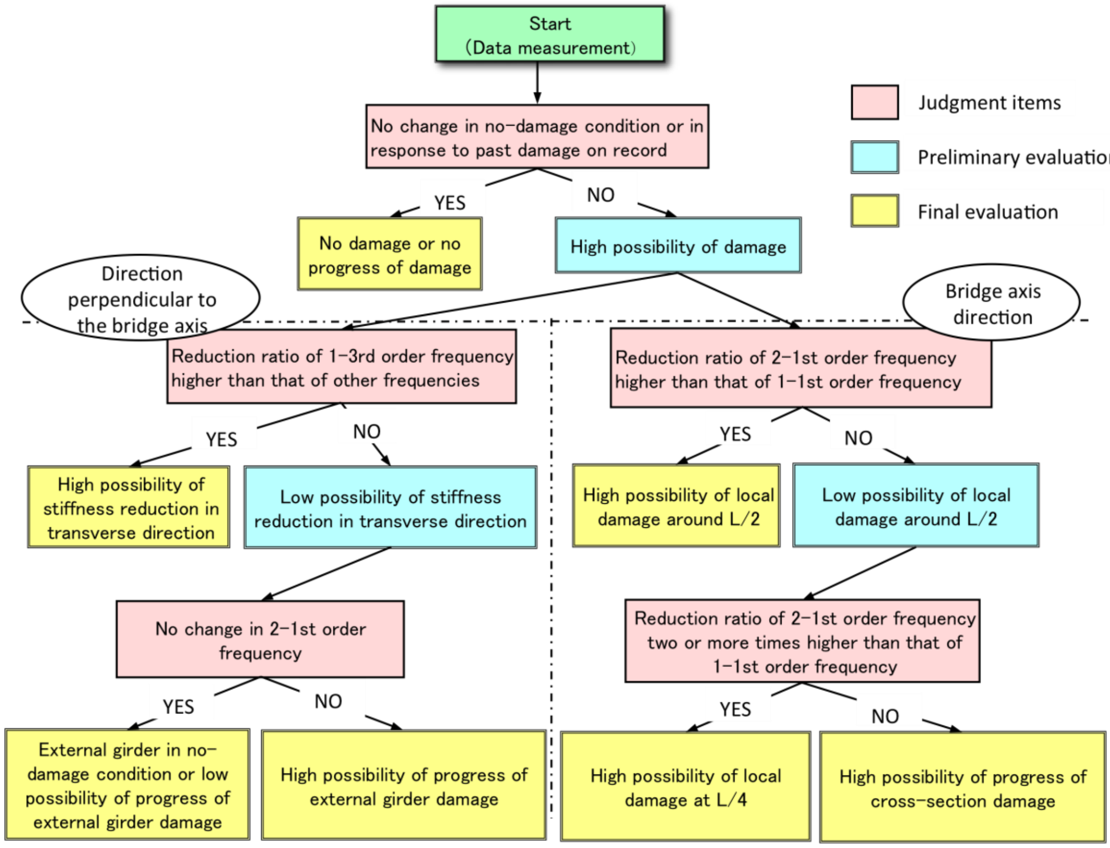


Figure 11. Flow of damage assessment based on natural frequency

Step 1. Estimate the location of damage according to the flowchart of Fig.10.

↓

Step 2. For the damage location identified at above *Step 1*, calculation the relationship between the degree of damage and the change ratio of the 1-1st order vibration frequency and linearly regress the results.

↓

Step 3. Substitute the change ratio of the 1-1st order natural frequency obtained by experiment into the linear regression equation derived in above *Step 2*, and estimate the damage degree.

Figure 12. Flow of damage assessment

Table 5. Estimated stiffness reduction ratio (%)

Exp. No.	Estimated value		Actual reduction ratio	Exp. No.	Estimated value		Actual reduction ratio
	Experiment	Analysis* (model calculation)			Experiment	Analysis* (model calculation)	
1-2	22.3	20.6	20	1-7	43.4	33.2	40
1-3	30.5	25.5	40	1-8	2.4	3.3	3.0
1-4	5.1	6.9	6.0	1-9	5.7	7.2	6.0
1-5	4.5	2.7	6.0	1-10	4.5	2.7	3.0
1-6	23.7	21.5	20	1-11	6.0	5.4	6.0

* : Local stiffness reduction converted to a overall cross-section value.

4 CONCLUSIONS

For the purpose of establishing a rational health diagnosis for bridges, Experimental data and model tests were carried out to examine the effectiveness of the assessment parameters in damage assessments based on the possibility of quantitative assessment of degree of damage. The main conclusions obtained in this study can be summarized as follows:

(1) From the results of analysis and experiments on a model bridge, it is found that the location and the degree of damage can be estimated by combining characteristic changes in mechanical behavior.
(2) It is possible to improve the precision of the estimation of the damage and its location by estimating not only the frequency of lower-order bending vibrations, but also the frequencies of torsional vibrations and higher-order bending vibration. Moreover, the natural frequency can be the most effective parameter because the natural frequency has an almost linear relation with the degree of damage (reduction of stiffness).
(3) Considering the damage assessment method based on the changes of the natural frequency, we could estimate the damage location of the bridge model by using the IF-THEN-ELSE rules, and the assessment results were reasonably accurate.
(4) The changes in the mechanical behaviors in both analytical and bridge tests was relatively well matched.

REFERENCES

[1] M. Ito and T. Katayama,"Vibration Damping of Bridge Structures", Proceedings of the Japan Society of Civil Engineers (JSCE), No. 117, pp.12-21, May 1965.

[2] A. Nishimura, M. Fujii, A. Miyamoto & T. Kagayama, "Sensitivity of Mechanical Behavior of Bridges for Their Damage Assessment", Jour. of Structural Mechanics and Earthquake Engineering, JSCE, No. 380, I-7, pp. 355-364, April 1987.

[3] Hewlett Packerd : 5451C Fourier Analyzer System Operating Manual MODAL ANALYSIS OPERATING & SERVICE MANUAL, 1978.12.

[4] D. J. Ewins, : Modal Testing, Theory and Practice, Research Studies Press, 1984.

[5] X. He, B. Moaveni, and J.P. Conte and A. Elgamal, "Modal Identification Study of Vincent Thomas Bridge Using Simulated Wild-Induced Ambient Vibration Data", Computer-Aided Civil and Infrastructure Engineering, Vol. 23, pp. 373-388, July 2008.