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Structural safety assessment with dynamic characteristics of prestressed concrete structures

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ABSTRACT

Prestressed concrete (PC) structures are advantageous in terms of durability. However, even they deteriorate as they age. In order to enhance the assessment technology for PC bridges, affiliation of authors is jointly conducting research utilizing demolished PC bridges. This paper describes the result of dynamic measurement of an existing PC T-section girder bridge which had been used for more than 40 years. In addition, numerical analysis was conducted using FE model. In the test, ambient vibration and forced vibration induced by heavy truck or weight were measured. Comparing the higher modes, frequencies of damaged span were lower than those of fair span. In the FE analysis, dynamic characteristics were calculated with damaged span model. From the result, it was found that the effect of the fixation of supports because of deteriorated bridge shoes was much larger than that of deterioration of girders in lower modes.

Keywords. Demolished PC bridges, Assessment technology, Dynamic measurement, Nondestructive testing, Numerical analysis

INTRODUCTION

Since prestressed concrete (PC) structure can control cracking by the use of prestressing, it is advantageous in respect of durability. However, even these PC structures deteriorate throughout in-service years. Deterioration has been seen in some PC bridges at stages earlier than their service life due to defective grout filling or other reasons. Therefore, effective understanding of remaining strength of a deteriorated PC structures would be the challenge in the maintenance of PC bridges.

Based on the above-mentioned background, Public Works Research Institute and Japan Prestressed Concrete Contractors Association has been jointly conducting clinical studies by using demolished bridges to enhance the evaluation technology of PC bridges. Measurement of the vibration characteristics of a PC bridge before its demolition was performed as part of the clinical study. Since vibration measurement has the advantage that it is a nondestructive testing and simple approach, whether it can be applicable in understanding the remaining strength of the deteriorated existing PC bridge is verified.

This paper reports the results of the vibration measurement and numerical analysis with FE model conducted on the existing PC T-section girder bridge that had been used for 40 years or more.

TARGET PC BRIDGE

The outline of the PC bridge for which the vibration measurement was performed is shown in **Table 1**, and the photo is shown in **Figure 1**.

Table 1. Target PC bridge

Bridge type	PC road bridge
System	B.B.R.V system, Post-tensioning T-shape simple girder (5 spans)
Bridge length	140.5 m, (27.3 m for each span)
Width	Overall width: 8.8 m, effective width: 8.0 m
Skew angle	90 deq.
Design strength	Precast beam: 40 N/mm^2 Cast-in-place: 30 N/mm^2

Note) Data notations followed those described on the as-built drawing

Figure 1. PC bridge

The structural type is a five-span simple PC T-section girder bridge. It was constructed in 1967 and served for 40 years or more. Since this PC bridge is constructed in the Sea of Japan shore, and the deterioration was severe due to the environment influenced by chloride, repairs had been performed more than once as damages of concrete or corrosion of steel were identified. However, because of confirmation of re-deterioration due to corrosion and breakage of PC wires as shown in **Figure 2**, rebuilding of the bridge was planned, and its service was terminated in September, 2010.

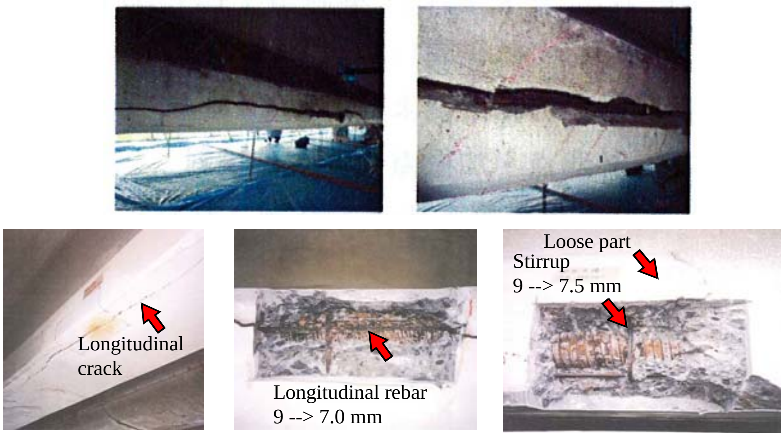


Figure 2. Deterioration of PC bridge

DYNAMIC MEASUREMENT

For the PC bridge, structure and geometry of each span are identical. However, comparison of vibration characteristics by performing vibration measurement for each span was conducted since the degradation varies greatly among spans. In addition, the demolition operation of the bridge had already been started at the time of measurement, and the pavements of the section from the Span 3 to the Span 5 had already been removed. In addition, the deck slab in the Span 3 had already been cut. Therefore, the vibration measurement was conducted for all spans with the exclusion of the Span 3.

The vibration test was performed by vehicle drop method, weight drop method, and ambient vibration method as shown in **Table 2**.

Table 2. Methods of excitation

	Vehicle drop method	Weight drop method	Ambient vibration method
Excitation method	4 rear wheels of a vehicle of 111 kN weight are dropped from a step of 130 mm high to provide forced excitation to the bridge.	A weight of 0.245 kN mass is dropped by free fall from a height of 1.0 m to provide forced excitation to the bridge. The shock excitation force is about 30 kN.	The ambient vibration during a calm period when there is no traffic on the adjacent temporary bridge is measured. * Because microtremor exists around the bridge due to natural and artificial causes, the bridge is continuously subject to random and micro vibration.
Advantages	No special excitation equipment other than testing vehicle is required. Relatively large excitation force can be obtained.	When a small size weight is used, vibration can be applied by human power. Relatively constant excitation force can be applied.	No excitation needs to be applied, and measurement can be performed easily even for a bridge which is currently in-service.
Disadvantages	The weight of vehicle sometimes cannot be negligible as an added mass. Also, the characteristic vibration of the vehicle (e.g. 3 Hz for a damp truck) can sometimes dominate as an excitation force.	Depending upon the mass of weight, there can be cases that adequate excitation force cannot be obtained.	A vibration measurement equipment with high precision is necessary. It may sometimes be influenced by noise, etc. and/or the process method because of small amplitude.

In the vibration measurement, the output signal obtained from servo-type accelerometer placed on the bridge surface was digitally converted with 1 kHz of sampling frequency after adjustment, then it was imported into a personal computer for a FFT analysis. Excitations were applied at 1/2 and 1/4 points of the span on the G3 girder, and measurements were performed at points that divide the G1 and G5 girders equally into 8 segments. (Shown in **Figure 3**) The measurement condition is shown in **Figure 4**.

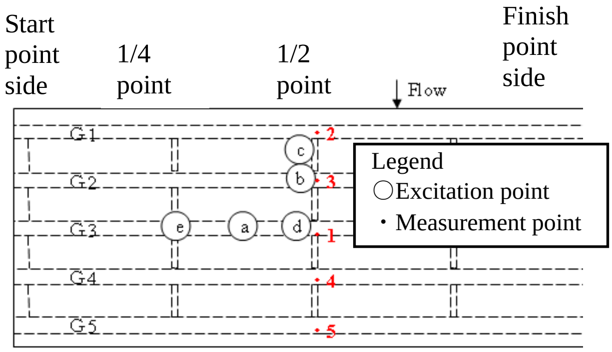


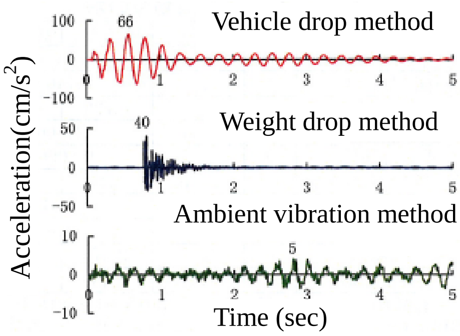
Figure 3. Excitation point and the measurement point

Figure 4. Excitation methods

RESULT OF DYNAMIC MEASUREMENT

Examples of accelerograms obtained in the vibration test performed by using vehicle drop method, weight drop method, and ambient vibration method are shown in **Figure 5**. The Figure shows results obtained in the Span 1 with excitation point located at 1/4 point of the span and measurement at 1/2 point of the G3 girder span. The oscillatory waveform shown in **Figure 5** confirmed peaks 4 ~ 5 times in a second by the vehicle drop method and the ambient vibration method. Vibrations with shorter cycles were confirmed in the weight drop method.

In **Figure 5**, the maximum acceleration measured in the vehicle drop method indicated the largest value (66 cm/s^2), followed by those in the weight drop method (40 cm/s^2) and the ambient vibration method (5 cm/s^2), in descending order. The spectrum graph obtained by FFT analysis based on the waveform data in **Figure 5** is shown in **Figure 6**.



Note) The results of measurement performed with excitation point "e" and measurement point "1" in the 1st. span is shown. Numeric values in the figure indicate the maximum acceleration.

Figure 5. Measured wave forms

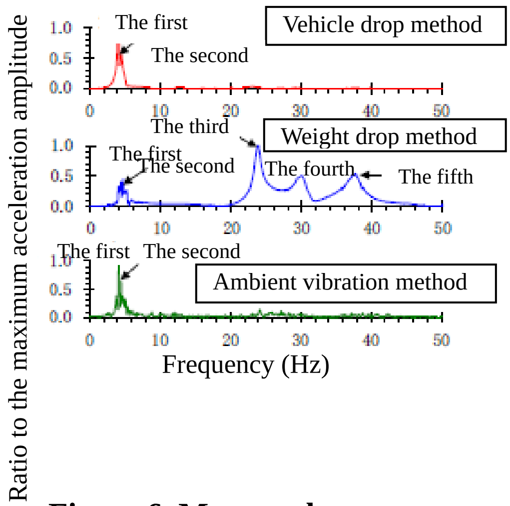


Figure 6. Measured spectra

As shown in **Figure 6**, dominant frequencies have been concentrated in the range of 4 Hz ~ 6 Hz in the vehicle drop method and the ambient vibration method. Additionally, dominant frequencies in the range of 20 Hz ~ 40 Hz were confirmed in the weight drop method. In **Figure 6**, the dominant first frequencies measured in the weight drop method indicated the largest value (4.3 Hz), followed by those in the vehicle drop method (4.2 Hz), and the ambient vibration method (3.8 Hz), in descending order.

Also, for the weight drop method in which the third through fifth dominant frequencies (20 Hz ~ 40 Hz) were confirmed, dominant frequencies of the Span 1, where damage was severe, and the Span 2, where damage was minor, were shown in **Table 3**.

Table 3. Comparison of frequency (Span 1, Span 2)

Mode number	Dominant frequency(Hz)		f1/f2	Mode of vibration
	Span 1	Span 2		
	f1	f2		
First	4.3	4.3	1.00	First bending
Third	23.7	25.2	0.94	Unknown
Fourth	30.2	31.5	0.96	Third bending
Fifth	37.1	40.8	0.91	Unknown

As shown in **Table 3**, while the first dominant frequencies of the Span 1 and the Span 2 were identical, the third through fifth dominant frequencies of the Span 1, where damages was severe, indicated values that were about 4 ~ 9% smaller.

NUMERICAL ANALYSIS

Numerical analysis was performed according to the 3D FE model shown in **Figure 7**. The model was built with the elements and conditions provided in **Table 4**. Additionally, neither the transverse prestressing PC steel nor reinforcing bars are considered in the model shown in **Figure 7**.

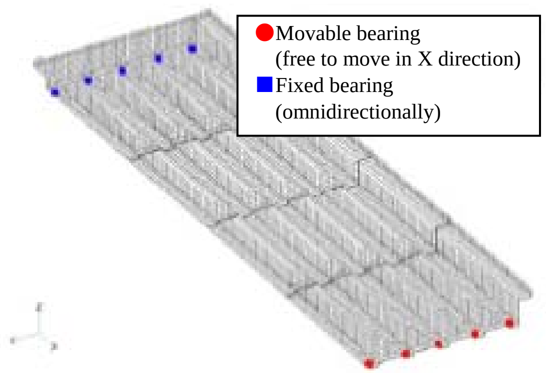


Figure 7. FE model

Table 4. FE model element and terms

Solid element	Main girder, cross beam, wheel guard, asphalt
Rod element	Main cable
Spring element	Bearing

Weight per unit volume /length

Concrete	24.5kN/m^3
Asphalt	22.5kN/m^3
Railing	0.5kN/m

Young's modulus(kN/mm^2)	
Concrete	33.7
PC tendon	204.6
Asphalt	3.0

Next, for the Span 1 where damage was severe, the setting of each member was determined as follows by assuming an analytical cases shown in **Table 5** to model the damage condition of member.

Table 5. Damage condition for analysis cases

Case	Damage condition	Modeling of damage
1	Intact state (no damage)	Basic model
2	Partial loss of area of main girder concrete	Deletion of the elements
3	Cracking in main girder concrete	Double node of cracking part
4	Reduced steel due to corrosion	Reduced cross section of PC steel
5	Introduction of tensile force in main cable	Addition of temperature load
6	Deterioration of bearing (fixation)	Change the movable condition to fixed condition

Main girder concrete. Regarding the both sides of lower flanges of G1 and G3 girders where damages were significant, elements were deleted up to the location of reinforcing bars as shown in **Figure 8**. The deletion of elements was applied throughout the entire length of the girder.

Cracking was modeled by assuming that the adjacent solid elements of the double node are discontinuous as shown in **Figure 9**. For the G1 girder, crack was modeled within cover concrete at the position of bottom PC cables. And for the G3 girder, crack was modeled at the position of second row of PC cables and all through the lower flange. The cracks are introduced in the area where C1 cables, which is located in the uppermost position, are aligned horizontally as shown in **Figure 10**.

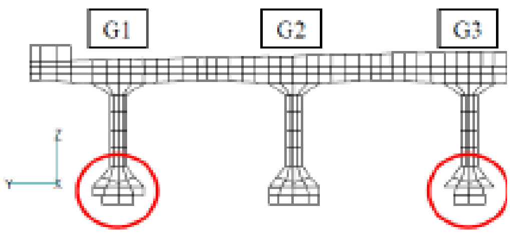
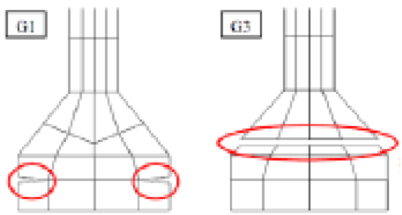


Figure 8. Concrete section of defect model



Note) The double nodes on the identical cordinate are represented separately.

Figure 9. Cracked concrete model

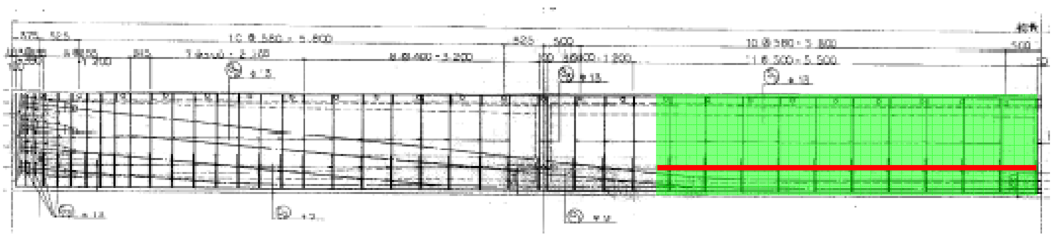


Figure 10. Area of crack

Main cable. To model corrosion and breakage in the main cable, the cross section of the PC steel was reduced for the entire length. The reduction rate was 34.3% for the G1 girder, 12.0% for the G3 girder, and 1.0% for the G2, G4, and G5 girders. Additionally, the prestressing force was modeled by adding PC steel tensile force as temperature load.

Bearing. The deterioration model of bearing was assumed by changing the movable bearing to the fixed bearing.

RESULT OF NUMERICAL ANALYSIS

Result of numerical analysis shown in **Table 6**.

Table 6. Result of numerical analysis

Case	First bending		Second bending f2 (Hz)	Third bending		First torsion t1 (Hz)
	f1 (Hz)	Rate of change from Intact state (%)		f3 (Hz)	Rate of change from Intact state (%)	
1:Intact state	3.51	—	12.60	29.50	—	5.28
2:Partial loss of area	3.45	-1.7%	12.38	29.06	-1.5%	5.25
3:Cracking	3.51	0.0%	12.53	29.16	-1.2%	5.28
4:Reduced steel	3.50	-0.3%	12.58	29.46	-0.1%	5.28
5:Introduction of tensile force	3.51	0.0%	12.61	29.50	0.0%	5.28
6:Fixation of bearing	5.11	+45.6%	12.86	29.44	-0.2%	5.70

Decrease in frequencies were observed due to degradation factors (i.e. partial loss of area, cracking, and reduced cross sectional area of PC steel) compared with those observed in the intact state.

Among those, the model of partial loss of area of concrete indicated the largest decrease in frequency compared with that observed in the intact state. On the other hand, no decrease in frequency due to prestressing was observed. It is assumed that stiffness reductions such as cracking would influence the decrease in frequency. Additionally, as for the degradation of the bearing, while the frequency of the first bending mode indicated a large value due to changed restraint condition, the larger the order of frequency was, the smaller the decreasing rate of the frequency became.

Furthermore, when the measurement results of Section 4 (**Table 3**) and the analysis results of Section 6 (**Table 6**) are compared regarding the first bending and third bending of vibration modes, rates of decrease in the frequency were 0 ~ 4% in the measurement results and about 0 ~ 2% in the analytical results, indicating differences among values. Nevertheless both the measurement results and the analytical results could confirm decrease in frequencies due to damages in the third bending mode. Therefore, the possibility of detecting remaining strength due to damages was indicated by measurement of the change in post-service vibration characteristics after having understood the frequency of the PC bridge during its sound state.

CONCLUSION

Followings were confirmed based on this vibration measurement.

(1) The third through fifth dominant frequencies (20 Hz ~ 40 Hz) were confirmed in the weight drop method as a result of the vibration test performed for the existing PC bridge. The third through fifth dominant frequencies indicated a decrease of frequency in severely damaged spans compared to almost sound spans.

(2) Decrease in frequency due to each degradation factor was confirmed based on the numerical analysis result of the PC bridge in which damage conditions were modeled.

(3) The possibility of detecting PC bridge damage based on changes in vibration characteristics through vibration measurements, performed during its sound state and post-service time periods, was indicated.

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