Dynamic Parameter Variations of a 5-Span Prestressed Railway Bridge

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1. Introduction

As the deterioration of bridges is becoming a serious problem all over the world, it is urgent to develop the maintenance methods for the aging bridges. Under this condition, the structural health monitoring technology is development rapidly, and remarkable achievements have already been made[1-3]. The research points have become more and more specific and in-deep.

In the previous studies, there are enough evidences to show that the environment variations can affect the structural dynamic parameters[4,5]. As the dynamic parameters are the core prerequisites of an effective structural damage identification, the environment variations will affect the success of damage identification significantly.

Meanwhile, in order to overcome the high cost, and high labor requirement of the installation and maintenance of the traditional wired sensor system, the wireless acceleration sensor network system can be a feasible solution with the fast development of wireless communication technology[6,7].

According to the above two points, this study mainly discusses about the mode frequency and damping ratio variations under difference environment conditions of a 5-span prestressed concrete railway bridge by employing a wireless acceleration sensor network system.

2. Overview of the bridge

The bridge is a 5-span prestressed concrete railway bridge which construction was completed in 1977. The bridge lies over Muka River (Mukagawa) in Kitami City. Bridge length is 158.8m, and span length is about 31.3m. The service of the bridge has been stopped for several years. The sleepers and the tracks were destructed before all the experiments. A general drawing of the bridge was shown in Fig.1 and the different heights of abutments and piers were shown in Fig. 2.

3. Dynamic tests on Mukagawa Bridge

3.1. Layout of the experiment

A series of vibration experiments were conducted on the bridge from November 2015 to November 2016 to investigate the dynamic parameter variations of the bridge. As the structural type of the bridge is multiple-span simply supported beam bridge, it is difficult to obtain the mode shapes of the whole bridge with

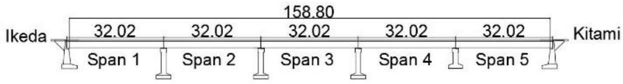


Fig. 1　General drawing of Mukagawa Bridge

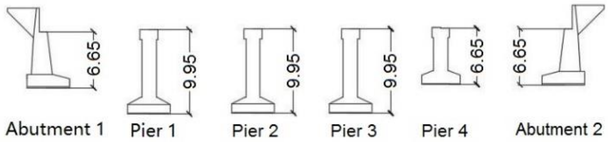
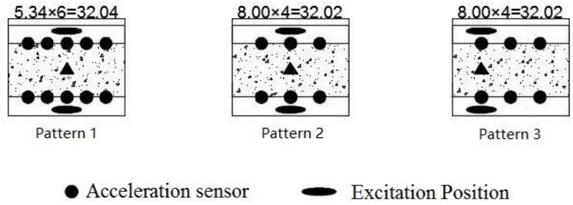
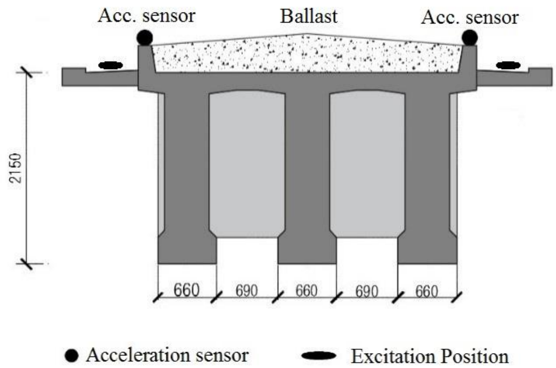


Fig. 2　Length of abutments and piers



a）　Top view



b）　Section view

Fig. 3　Wireless acceleration sensor distribution maps

limited acceleration sensors. So in these experiments, only the dynamic characteristics of all the single spans are discussed in this paper. The sensor distribution maps and the detail locations were shown in Fig. 3. In the first experiment, the excitation method was 2 people jumping and landing on the middle point of the spans synchronously, as shown in Pattern 1 of Fig. 3(a). From the second experiment the quarter point excitation was aided to obtain the second bending and torsional mode, and the number of sensors was reduce, as shown in Pattern 2 and 3 of Fig. 3(a). The section view of the bridge was also presented to show the detail sensor positions, as shown Fig. 3(b). In order to identify better torsional modes, asymmetric excitation were also implemented on every pattern.

3.2. Data processing method

As the excitation method is only human jumping and landing in the vertical direction, the horizon directions were not excited enough. So only the z direction data was analysis in this research, and the x and y direction data was ignored. After the acceleration data was collected from the wireless acceleration sensors, the acceleration data was analyzed by fast Fourier transform, then the power spectrum was obtained. As only the simple jumping impulse excitation method was applied in the experiments, the free damped vibration waveforms were quite regular and the power spectrums had very distinct peaks. So natural frequencies were identified by peak picking method. Damping ratios were calculated by half-power bandwidth method. Meanwhile, experimental modal shapes were estimated by cross spectrum method.

4. Modal analysis results

The bridge vibration experiments were conducted on September 6th 2015, February 11th 2016, August 18th 2016, October 18th 2016, and November 29th 2016 successively. The experiment on August 18th 2016 did not have the data of Span 4 and 5 owning to some battery problems. The air temperature was measured by a EL-USB-2 temperature sensor. In every experimental day, the air temperature fluctuation was very small, so only the average air temperatures of the experimental days were summarized in Fig. 4.

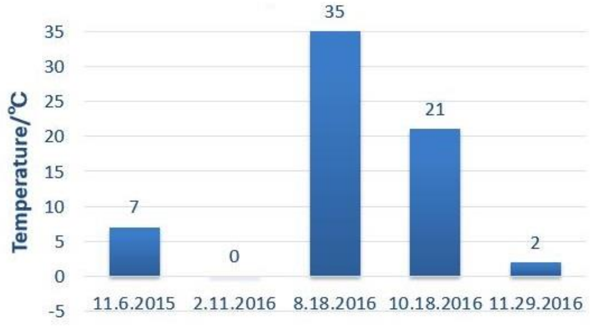
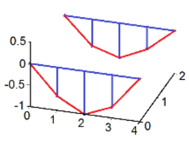
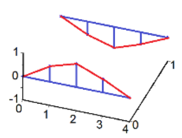
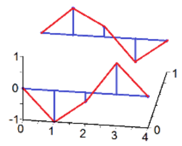
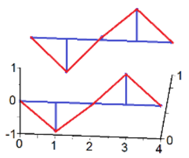
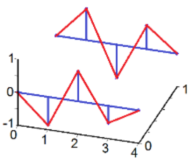
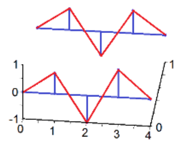


Fig.4 Average air temperatures

Table 1 Mode shapes of Span 2 on Experiment on 2.11.2016

Mode 1	Mode 2	Mode 3
		
Mode 4	Mode 5	Mode 6
		

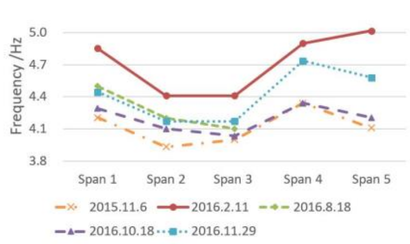
In this series of experiments, considering to the reliability of the data, six modes could be identified effectively. The six mode shapes of Span 3 on February 11th 2016 were shown as an example in Table 1. From the lowest mode to the highest mode, the six modes are the first bending mode (Mode 1), the first torsional mode (Mode 2), the second torsional mode (Mode 3), the second bending mode (Mode 4), the third torsional mode (Mode 5), and the third bending mode (Mode 6).

In the first experiment (September 6th 2015), only 4 modes were identified because the middle points of all spans were the only excitation positions. The 4 identified modes were the first bending mode (Mode 1), the first torsional mode (Mode 2), the third torsional mode (Mode 5) and the third bending mode (Mode 6). The second bending and torsional modes (Mode 3 and 4) could not be identified in the first experiment. From the second experiment (February 11th 2016), the quarter point excitation was added (except the experiment on August 18th 2016). The second bending mode (Mode 3) and the second torsional mode (Mode 4) could also be identified. So in total 6 modes were identified (Mode 1~6).

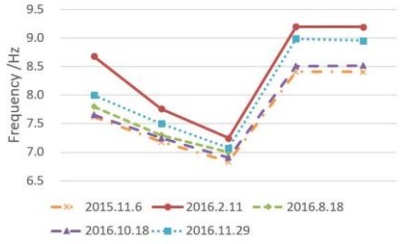
In order to show the dynamic parameter variations and make obvious comparison between difference spans, the variations of the natural frequencies and damping ratios were summarized in Fig. 4 and 5. In all the figures in Fig. 4 and 5, the results of different experiments were expressed in different curves. Generally, for the mode frequencies, the curves in every figure had the same variation trend. For the damping ratios, the curves in every figure were quite disordered.

As shown in all the figures of Fig. 4, the 5 spans on February 11th 2016 experiment had the highest mode frequencies. The mode frequencies of the 5 spans on November 6th 2015, August 18th 2016, and October 18th 2016 were relatively lower, and the difference between these three curves was very small. Generally the 5 spans on November 6th 2015 had the lowest mode frequencies. The 5 spans on November 29th 2016 had the middle-ranking of the mode frequencies.

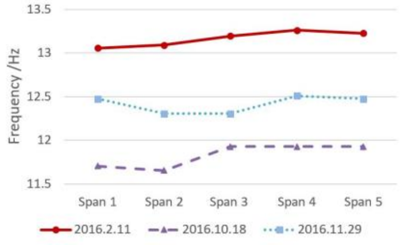
Referring to the Fig. 4, the experimental day of February 11th 2016 had the lowest average air temperature. The lowest temperature led to the highest Young's modulus of the concrete and rebar. Meanwhile, the temperature difference between the two experimental days on February 11th 2016 and November 29th 2016 was only 2°C。In this case, the Young's modulus difference between two experiments on February 11th 2016 and November 29th 2016 which were caused by the 2°C air temperature difference could be ignored. However, the mode frequency differences between the two experiments on February



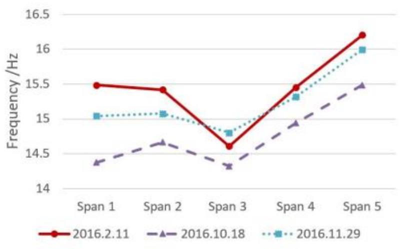
a)　Mode 1



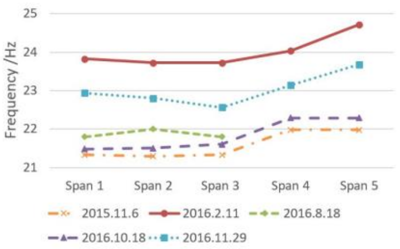
b)　Mode 2



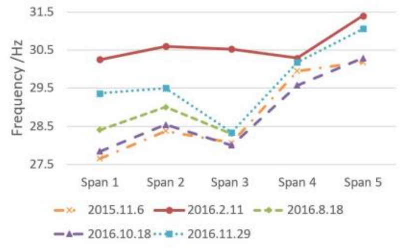
c)　Mode 3



d)　Mode 4

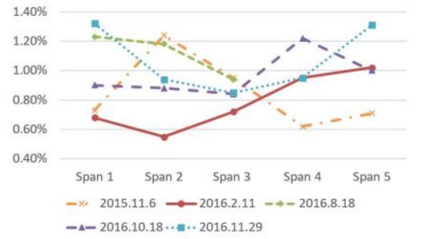


e)　Mode 5

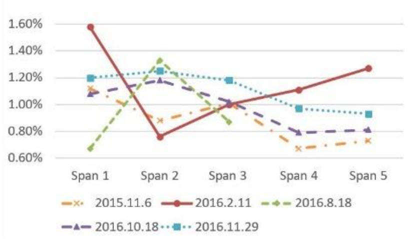


f)　Mode 6

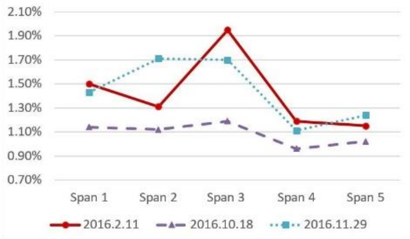
Fig. 4 Mode frequencies of the 5 spans



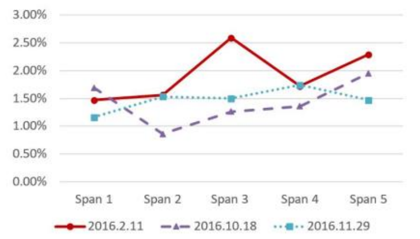
a)　Mode 1



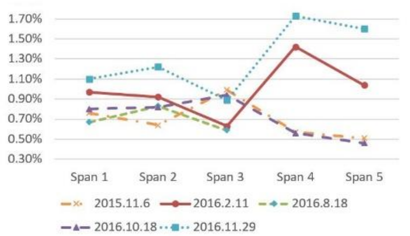
b)　Mode 2



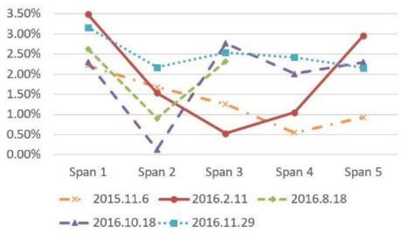
c)　Mode 3



d)　Mode 4



e)　Mode 5



f)　Mode 6

Fig. 5　Damping ratios of the 5 spans

a)　on Nov 6th 2015

b)　on Feb 2nd 2016

c)　on Nov 18th 2016

Fig. 6　Ballast conditions

11th 2016 and November 29th 2016 were salient. This phenomenon could not be simply explained by the Young's modulus difference of concrete and rebar. The material property of the ballasts on the deck should gain more attention.

The three kinds of ballast conditions were shown in Fig.6. In Fig.6(a), the ballasts were in very dry condition (investigated on

Nov 6th 2015), and the ballasts could be separated easily. In Fig.6(b), the ballasts were totally frozen (investigated on Feb 11th 2016), all the ballasts were combined together and filled with ice. In Fig.6(c), the ballast condition was neither dry nor frozen (investigated on Feb 11th 2016). There was some snow or small ice in the space between the ballasts. The ballasts more likely behaved in the intermediate state between dry and frozen.

It is easy to understand that the frozen ballasts could give the girders additional stiffness. With the ice between the ballasts, the relative movement between the ballasts was strongly constrained. At the same time, the relative movement between the ballasts and the deck was also constrained. In this circumstances, the ice in the space between the ballasts performed as the filler, and the ballasts could perform like concrete, so the structural intergrity could be increased significantly. Thus, there is no doubt that the bridge had the highest mode frequencies in the experiment on February 11th 2016. By constrast, the ballasts on the intermediate state between dry and frozen were very complicated. Through the repeated freezing and thawing of the snow, the water will penetrate into the space between the ballasts. When the temperature is around 0°C, this water would become instable ice. With the effects of filling and bonding, the relative movement between the ballast would be constrained to some extent. The stiffness of the girders would also be increased, but the additional stiffness should be less than the totally frozen ballast condition and larger than the dry ballast condition.

Making a crosswise comparison between the mode frequencies of the 5 spans in all the experiments, a phenomenon was presented that the mode frequencies of Mode 1 and 2of Span 2 and 3 were significantly lower than Span 1, 4 and 5. The reason for this phenomenon was already explained in the previous research[8]. Owning to the excitation energy was not sufficient to produce the bearing displacement of the line bearing supports, the girders and the piers or abutment could be seen as frame structures. The frames of all the single spans have different natural frequencies owning to the different heights of abutment or piers.

In all the figures of Fig.5, the damping ratio variation regularities were quite complicated to understand. For every mode, the damping ratio variation curves of the 5 spans did not have the same variation trend. No matter comparing the damping ratios between the different experiments or comparing the damping ratios between the different spans, it is easy to find that the damping ratios increased or decreased in a completely disordered way. It is very difficult to draw a conclusion that how the damping ratios were affected by the environment change. However, there were still some points need further study. Firstly, how the variation of the ballast property lead to the damping ratio variation of the bridge. After raining, the water content of the ballasts will change significantly. The friction between the ballasts and the mass of the ballasts would change at the same time. And in winter, the ice between the ballasts would increase the stiffness of the girders obviously. Secondly, after snowing, the snow on the bridge would also absorb the vibration energy and increase the mass of the bridge. The detailed mechanism still need a great number of investigations.

5. Conclusions

In this paper, a discussion about the mode frequency and damping ratio variations under difference environment conditions of a 5-span prestressed concrete railway bridge was presented.

Firstly, one point was confirmed that the environment variations would have a significantly effect on mode frequencies and damping ratios. Secondly, for this kind of ballasted railway bridge, the variation of the ballast property would have more apparent effect on the mode frequencies and damping ratios. The detailed mechanism of that how the frozen ballasts, wet ballasts with a high water content, and the ballasts in the intermediate state between totally frozen and wet vary the dynamic parameters still need further study.

6. Acknowledgements

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