Dynamic Behavior of A Steel-Truss Railway Bridge Under the Action of Moving Trains

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ABSTRACT

The dynamic behavior of the steel railway bridge attracts intensive attentions in engineering communities due to the significance and particularity of moving train loads. The monitoring of the railway bridge is thus becoming more and more popular as it offers a straightforward approach to help understand the structural dynamic behaviors under train actions. In this study, a railway bridge with an efficient structural health monitoring system (SHMS) in China is taken as an example to investigate the train-induced vibrations of the steel railway bridges. With the field-measured data by SHMS, the vertical vibrations of three types of girders are analyzed. Treating the bridge as a time-varying system, the modal frequencies of the structures are emphatically investigated via the wavelet-based time-frequency analyses. Also, the transverse displacements of the piers are also presented to evaluate the working status. Dynamic factor induced by train loads is visited and compared with the suggestions in specifications. The dynamic performance of the train during movement is also evaluated. The findings are expected to provide references for the design of steel railway bridges.

1. INTRODUCTION

Nowadays, the railway bridge has been working as an important element in the land transportation to meet the rapid development of economics and society. Numerous
railway bridges have been constructed or are still under construction in the world. For example, up to 2015, the distance of railway roads in China has achieved 121,000km, among which 19,000km are for high-speed trains. In the existing railway transportation, the bridge is the most frequently used media to cross obstacles (rivers, mountains, or even seas) or facilitate the design of intersections (Wang et al. 2016). Moreover, in modern high-speed railway lines, the bridge has become the most popular carrier for transportation as the high-flatness requirement must be satisfied.

Compared to highway bridges, a typical feature of the railway bridge is that the train loads are much huger with apparent impact effects. Also, the frequency of train vehicles is quite intensive due to the function of railway lines. Thus, in light of the characteristics of daily experienced actions, the railway bridge must be equipped with enough stiffness and well dynamic performance. In engineering communities, the girder bridge is the most frequently employed type as it could easily meet the requirement of high stiffness. Concrete and steel are two main materials available for the construction of railway bridges. The steel girder bridge is not only easily designed with high stiffness, but also with lighter weight than the concrete bridge. Hence, the steel girder bridge is very popular in railway transportation, e.g., the Nanjing Yangtze River Bridge in China and the Government Bridge in USA.

As the key engineering projects, the steel railway bridges are usually designed with an expected service life of more than 50 years. In such a long service period, the bridge will inevitably suffer from a great many long-term and short-term environmental actions (e.g. traffic loads, thermal variation, environmental corrosion, even earthquakes, etc.), so that many problems including fatigue effects, material aging, component damages, etc. are gradually produced and developed with the time, directly or indirectly causing slow performance deterioration, severe structural damages and even catastrophic collapse to bridge structures (Li et al. 2006; Lynch and Koh 2006; Ou and Li 2010).

In recent few decades, the structural health monitoring technology provides an efficient approach to monitor the health status and evaluate the working condition of a structure, offering the potentials to ensure the structural sustainability and serviceability during its service life (Aktan et al. 2000; Ou and Li 2010; Xu et al. 2012). Structural health monitoring is realized by an integrated intelligent system called structural health monitoring system (SHMS). Many SHMSs have been proposed and successfully applied on railway bridge projects: e.g., the New Arsta Railway Bridge in Sweden (Enckell 2007), the Dashengguan Bridge in China (Ding et al. 2015). The intelligent system consists of various sensors to collect both input actions and output responses. In such a view, the railway bridge installed with a SHMS can be considered as a full-scale experimental platform, making people to better understand the dynamic behaviors of the structure under moving train loads.

For the highway bridge, the dead loads of the vehicles are quite small compared to the gravity of the whole bridge. Hence, the bridge is always treated as a time-invariant system. However, the gravity of the train cannot be separately treated in the dynamic
analysis of the railway bridge since it could not be neglected when compared to the gravity of the whole bridge. Thus, the railway bridge should be considered as a time-varying system in the train-induced dynamic analysis. In such a case, it is necessary to investigate and fully understand the train-induced dynamic behavior of the railway bridge via full-scale measurements, which offer the most direct approach to present the inherent time-varying features.

In this paper, a railway bridge with an efficient structural health monitoring system (SHMS) in China is taken as an example to investigate the train-induced vibrations of steel bridges. With the field-measured data by SHMS, the vertical vibrations of three types of girders are analyzed. Treating the bridge as a time-varying system, the modal frequencies of the structures are emphatically investigated via the wavelet-based time-frequency analyses. Also, the transverse displacements of the piers are also presented to evaluate the working status. Dynamic factor induced by train loads is visited and compared with the suggestions in specifications. The dynamic performance of the train during movement is also evaluated. The findings are expected to provide references for the design of steel-truss railway bridges.

2. ENGINEERING BACKGROUND

2.1 Description of the railway bridge

The steel railway bridge investigated in this study consists of 11 simply supported bridges, which are in three structural types. The structure of this railway bridge is shown in Fig. 1. In the three main types, two of them belong to steel truss bridges and another is the steel plate girder bridge. As shown in Fig. 1, seven top-bear truss bridges, of which the train moves on the top plane, are employed and named as G1, and from G5 to G10. The length of each span is 72.8m, and the girder is 9.5m in height and 3m wide. G2, G3 and G4 are navigable spans, so the bottom-bear truss bridges are utilized. The span of this bridge is 75.0m, and the height is 10.0m and the width is 5.8m. G11 with a span of 35.0m is the steel plate girder bridge, which are constituted with two flanged beam via the truss elements as the horizontal connections. The height of this bridge is 3.28m and the width is 2.0m. In addition, there are ten hollow reinforced concrete piers in this bridge and the heights range from 19.67m to 26.28m.

![Fig. 1 Structure of the railway bridge (Unit: m)](image-url)

This railway bridge was built in 1936 and open to traffic in 1954. Up to 2017, it has
been in service for 63 years. This bridge is mainly used for both passenger and freight transport. In each day, about 110 trains will pass this bridge. The general view of this railway bridge is shown in Fig. 2.

![General view of the railway bridge](image)

**Fig. 2 General view of the railway bridge**

### 2.2 Description of the SHMS

Under the action of heavy train loads, the health status of this old bridge must be carefully evaluated. Hence, a systematic SHMS is implemented on this bridge. The layout of the SHMS is presented in Fig. 3. There are four types of sensors included in the SHMS, namely accelerometer, displacement sensor, strain gauge and temperature meter. Most of the sensors are installed on G1, G2 and G11, which are the three main types. The accelerometers installed on the span are utilized to monitor the vertical and lateral vibrations of the main span. The displacement sensor is employed to monitor the lateral displacements of the main span and the piers. Two displacement sensors are installed on top of the first and fourth piers (Z1 and Z4), respectively. A wheel-force monitoring sensor is installed between G1 and G2 to collect the load information of the passing train. During measurement, the sampling frequency of all the sensors are set as 200Hz.

![Structural health monitoring system of the railway bridge](image)

**AC**: Accelerometer  **DIS**: Displacement sensor  **SG**: Strain gauge  **TEM**: Temperature meter

**Fig. 3 Structural health monitoring system of the railway bridge**

### 3. VIBRATION ANALYSIS OF THE BRIDGE STRUCTURE

Treating the railway bridge under the action of trains as a time–varying system, the
vibrations of the main bridge structure are analyzed. Traditionally, the Fourier transform is frequently utilized in the spectral analysis. However, the vibration of the time-varying system is time-dependent, which means the signal is nonstationary in nature. Hence, the transient information of the vibration cannot be fully captured by the Fourier transform. The development of modern signal analysis has realized the time-frequency representation via several approaches, such as wavelet transform and Hilbert-Huang transform. Time-frequency data analysis tools express the energy concentration as a joint function of both time and frequency, thus the transient features of the signal can be integrally presented without leakage. In this study, the wavelet analysis is utilized to analyze the vibration of the bridge structure.

3.1 Theoretical background of wavelet analysis

The wavelet transform is a linear transform, which decomposes a signal via basis functions that are simply dilations and translations of the mother wavelet, through the convolution of the signal and the scaled mother wavelet (Kijewski and Kareem 2003). The continuous wavelet transform of a signal is given by

$$W(a, t) = \frac{1}{\sqrt{a}} \int_{-\infty}^{+\infty} x(t) \varphi^* \left( \frac{t - \tau}{a} \right) dt = \frac{\sqrt{a}}{2\pi} \int_{-\infty}^{+\infty} \hat{x}(\omega) \hat{\varphi}^*(\omega) e^{j\omega t} d\omega$$

(1)

where $W(a, t)$ is the wavelet coefficient, $a$ is the dilation parameter, $t$ is the translation parameter, $\omega$ is the frequency, $x(t)$ is the signal, $\varphi(t)$ is the mother wavelet function, and $\hat{x}(\omega)$ and $\hat{\varphi}^*(\omega)$ are their Fourier transforms, which are defined as

$$\hat{x}(\omega) = \int_{-\infty}^{+\infty} x(t) e^{-j\omega t} dt$$

(2)

$$\hat{\varphi}^*(\omega) = \int_{-\infty}^{+\infty} \varphi^*(t) e^{-j\omega t} dt$$

(3)

Although there are countless mother wavelets used in practice, e.g., Daubechies wavelet, Morlet wavelet, Meyer wavelet, etc., the Morlet wavelet is utilized in study due to its analogs to the Fourier transform. The modulus of its wavelet coefficients is directly proportional to the amplitude of harmonic signals, thus making it quite attractive for harmonic analysis. The expression of the Morlet wavelet and its Fourier transform are given by

$$\varphi(t) = e^{-t^2/2} \cdot e^{j\omega_0 t}$$

(4)
where $\omega_0$ is the central frequency of the Morlet wavelet.

The Morlet wavelet function is illustrated in Fig. 4(a), and its Fourier transform is shown in Fig. 4(b).

\[
\varphi(a\omega) = e^{-\left(\frac{\omega_0 - \omega}{a}\right)^2/2} \tag{5}
\]

Since the wavelet coefficient is not a function of time and frequency, but a function of time and the dilation parameter $a$, one need to establish a relationship between frequency and the dilation parameter to obtain a density function of time and frequency. For the Morlet wavelet, a unique relationship between the dilation parameter and frequency is obvious by maximizing Eq. (5) to yield

\[
\omega = \omega_0/a \tag{6}
\]

With Fig. 4(b), it can be predicted that the modulus of the wavelet coefficient attains the maximum at a frequency of $\omega_0/a$ when the signal $x(\tau)$ most resembles the mother wavelet with scale $a$ and location $t$.

3.2 Analysis of the main girder

The aforementioned three types of girders, as shown in Fig. 5, are taken to analyze the dynamic behavior of the railway bridge under the action of moving trains. The structures of G1 and G2 are similar, but the loading locations are different. Generally, the stiffness of G2 is larger than that of G1, and the stiffness of G1 is larger than that of G11.
There is one accelerometer installed in the middle of the span for G1, G2 and G11. The accelerometer employed in the SHMS is the 891-2 type vibration sensor made by the Institute of Engineering Mechanics in China. The typical pictures of this accelerometer and its installation on the railway bridge are presented in Fig. 6. The acceleration ranging from 0 to 40m/s² can be accurately recorded with a resolution of $10^{-5}$ m/s² when equipped with the assortative amplifier.

With the accelerometer, the vertical accelerations in the middle of three main spans are successfully recorded when a typical train moves with a speed of 35km/h. The accelerations of the three main girders are shown in Fig. 7. It is obvious that the amplitudes of the three girders mainly indicate that $G_2 > G_1 > G_3$, which indirectly reflect the same sequence of the general stiffness.
The wavelet only the time-frequency densities. 7 Vertical acceleration transform signal vs for the time-analyses of the three cases are shown in Fig. 8. The time-frequency presentations of the energy densities for the three cases are shown in Fig. 8.
As shown in Fig. 8, the energy distribution of G1 in time-frequency domain is similar to that of G2. The difference between them is that only the low-frequency content (around 4Hz) of G1 is apparent in the whole duration, but the energies at different frequencies of G2 are remarkable simultaneously. Similar to G2, the energies at almost all dominated frequencies of G11 are prominent in the same duration. However, the energy of the vibration of G11 at the beginning is mainly concentrated around 5Hz and ends around a frequency around 6Hz.

For the modal frequencies of the railway bridge, typical time-varying features with evident intermittencies can be captured. This is quite different from the time-invariant system. Due to the significant effect of the moving masses from the train, the modal frequencies of the train-bridge coupled system become time-dependent. Also, some close-spaced modal frequencies are easily coupled in the train-induced vibration. When comparing G1, G2 and G11, the time-varying feature of the modal frequency is the most prominent in G11, which is lightest in the three cases and most easily influenced by the gravity of the train.

Fig. 8 Wavelet scalogram of the vibrations of the main girders
3.3 Analysis of the pier

Due to many reasons, such as the irregularity of tracks, hunting oscillation of the bogie of the train, lateral movement of the carriages, the bridge and piers usually experience undesirable lateral vibrations, which should be paid special attentions. As the pier is the last defending component of safety, it is emphatically analyzed in this study. The lateral vibrations of the piers (Z1 and Z4) are also recorded by the displacement sensors. The displacement sensor utilized is the 892-1 type accelerometer as well. The displacement measurement switch of this sensor is activated. The dynamic displacement ranging from 0 to 15mm can be accurately measured with a resolution of $2 \times 10^{-5}$mm. The lateral displacements of Z1 and Z4 during the movement of train are presented in Fig. 9. It is found that the amplitude of Z4 is smaller than that of Z1 although the structures of the two piers are the same. Thus it is believed that there is randomness existing in the lateral vibration of the pier.

![Fig. 9 Lateral displacements of the piers under moving train loads](image)

The lateral displacements of the piers in Fig. 9 are also analyzed with the wavelet transform. Considering the importance of the vibration with large amplitudes, only the signal with a 150s duration in the dotted grid in Fig. 9 is taken for the time-frequency analysis for each case. The time-frequency presentations of the energy densities for the two cases are shown in Fig. 10.
As shown in Fig. 10, the presented modal frequencies of the two piers are similar, but the energy distributions are different. The vibration energy of Z4 is the most prominent under 2Hz, while the energies of Z1 around 4Hz and 7Hz are obvious as well. Different from the vibration of the main girder, there are periodic tails in the vibrations of Z1 and Z4. This phenomenon is more apparent in the time-frequency spectrum shown in Fig. 10. The periodic tails are mainly attributed to the travelling effect of vibrations in subsequent spans. Hence, the consideration of the travelling effect of train actions is highlighted in the analysis and design of railway bridges, as the tail vibration may shorten the fatigue life of the components.

4. Assessment of the impact factor

In current design of railway bridges, the dynamic effect of the train is considered by the product of an impact factor and the static load. Then the dynamic loading effect is simplified to an amplified static load. Traditionally, the impact factor is calculated by theoretical solutions or finite element method. However, the calculated results are not accurate enough due to many reasons, such as the uncertain boundary conditions,
inaccurate assessment of the material properties, performance deterioration of the structure, etc. In such a case, obtaining the impact factor from field measured data becomes much more meaningful and the results can be utilized to verify existing design theories.

In this study, the multi-level wavelet decomposition (Tao et al. 2017) is utilized to calculate the impact factor. The impact factor is defined as the maximum value of the ratio of the dynamic response to the corresponding static response, which is detailed as

$$\delta = \max \left( \frac{u_d}{u_s} \right)$$

(7)

where $\delta$ is the impact factor, $u_d$ is the dynamic response, $u_s$ is the static response.

Therefore, the critical step to calculate the impact factor from the field measured data is to obtain the static response. A numerical case by Wang et al. (2014) is taken as an example here. The simulated dynamic and static displacements of a railway bridge are presented in Fig. 11. Actually, the static response could be treated as the time-varying trend of the dynamic response. Hence, the multi-level wavelet decomposition is able to extract the static response as a low-frequency content. A comparison of the simulated static response and the extracted static response in Fig. 11 indicates that the extracted static response by the multi-level wavelet decomposition is reliable and can be utilized in the calculation of the impact factor.

![Fig. 11 Extraction of the static response via wavelet-based decomposition](image)

The strains of key components of the three typical girders are successfully recorded with CYB-YB-F1kB type strain gauges, which are made by the Beijing Shengsaike Technology Development Co., Ltd. Typical pictures of this strain gauge and its installation on the railway bridge are presented in Fig. 12. The strain ranging from 0 to 2000$\mu$ε can be accurately measured with a resolution of 0.2$\mu$ε. The time-histories of the strains on the three main girders are shown in Fig. 13. Also, the static strain is extracted for each case with the multi-level wavelet decomposition. Then, the impact factor is calculated according to Eq. (7), and the results are presented in Table 1.
According to the Fundamental Code for Design on Railway Bridge and Culvert (MRPRC 2005) in China, the impact factor for a steel simply supported bridge is suggest as

\[ \delta = 1 + \frac{28}{40 + L} \]  

(8)
where $L$ is the length of the span. Therefore, the suggested impact factors of G1, G2 and G11 by MRPRC (2005) are 1.248, 1.243, and 1.373, respectively.

Generally, the impact factors suggested by the code are a little larger than the measured results, which means the code is on the safe side. It is noticeable that the impact factor of the chord member is about 10% smaller than that of the web member for truss bridges. The local actions of the wheel loads may account for this phenomenon. In such a case, it is possible to suggest that the chord and web members be designed with different impact factors.

<table>
<thead>
<tr>
<th>Cases</th>
<th>G1</th>
<th>G2</th>
<th>G11</th>
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<tbody>
<tr>
<td></td>
<td>Chord</td>
<td>Web</td>
<td>Chord</td>
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<tr>
<td></td>
<td>1.080</td>
<td>1.181</td>
<td>1.112</td>
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</table>

### 5. DYNAMIC PERFORMANCE OF THE TRAIN

The dynamic performance of the train is another consideration when a train runs on the railway bridge. The derail coefficient (DC) and rate of load reduction (RLR) are two important parameters related to the risk of train derailment and usually utilized to evaluate the dynamic performance of the train (He et al. 2017).

The derail coefficient is defined as

$$DC = \frac{Q}{P}$$

(9)

where $Q$ is the lateral force induced by the wheels of a running train, $P$ is the vertical force exerted by the wheels of a running train. The upper limit showing an outstanding status of DC is 0.6 by (MRPRC 2004). Also, a common maximum value of DC is defined as

$$(DC)_{\text{max}} = \frac{Q}{P} + 1.65\sigma$$

(10)

where $\overline{Q/P}$ is the statistical average of $Q/P$, $\sigma$ is the variance of $Q/P$. The corresponding upper limit of $(DC)_{\text{max}}$ is set as 0.4.

The definition of RLR is given by

$$RLR = \frac{|P_L - P_R|}{P_L + P_R}$$

(11)

where $P_L$ is the vertical force exerted by the left wheel, $P_R$ is the vertical force by the right wheel. The upper limit of the RLR is set as 0.6 in the specification (MRPRC 1985).

For this railway bridge, a pair of wheel-force monitoring sensors is symmetrically installed on the tracks at the connection of G1 and G2. The sensor is developed by China Academy of Railway Sciences and can realize the dynamic monitoring of vertical and lateral forces exerted by the wheels. A picture of the sensor is shown in Fig. 14.
With the measured lateral and vertical forces, the DC and RLR are calculated, as shown in Fig. 14. The common maximum values of the left and right wheels are 0.175 and 0.263, respectively. They are much smaller than the upper limit 0.4. In Fig. 14, it can be seen that both the measured DC and RLR are smaller than the specified upper limits in specifications, which means the train is moving with a good dynamic performance.

![Graph showing DC and RLR values](image)

**Fig. 14 Evaluation of the dynamic performance of the moving train**

**6. CONCLUDING REMARKS**

The dynamic behavior of a steel railway bridge is analyzed in a time-frequency perspective with the aid of real-time measured data from the SHMS. The dynamic factor induced by the train is visited and the dynamic performance of the train is evaluated. Some conclusions from this study are summarized as follows:
(1) Typical time-varying features with evident intermittencies can be captured in the modal frequencies of the railway bridge. Some close-spaced modal frequencies are easily coupled in the train-induced vibration.

(2) Comparing G1, G2 and G11, the time-varying feature of the modal frequency is the most prominent in G11, which is lightest in the three cases and most easily influenced by the gravity of the train.

(3) The periodic tails existing in the lateral vibration of piers are mainly attributed to the travelling effect of vibrations in subsequent spans. Thus the consideration of this travelling effect is highlighted in the analysis and design of railway bridges, as tail vibrations may shorten the fatigue life of components.

(4) The static response can be well extracted from the measure dynamic response with the multi-level wavelet decomposition. Generally, the impact factors suggested by the code are a little larger than the measured results, which means the code is on the safe side.

(5) The local actions of wheel loads induce the impact factor of the chord member about 10% smaller than that of the web member for truss bridges. Hence, it is possible to suggest that the chord and web members be designed with different impact factors.

(6) All the measured DC, RLR and common maximum values of DC are smaller than the specified upper limits in specifications, which means the train is moving with a good dynamic performance.

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