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Structural health monitoring-based dynamic behavior evaluation of a long-span high-speed railway bridge

D.P. Mei*[1,2]

[1]*School of Civil Engineering, Southwest Jiaotong University, Chengdu 6177567, China*
[2]*China Railway Major Bridge Reconnaissance & Design Institute Co., Ltd., Wuhan 430050, China*

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Abstract. The dynamic performance of railway bridges under high-speed trains draws the attention of bridge engineers. The vibration issue for long-span bridges under high-speed trains is still not well understood due to lack of validations through structural health monitoring (SHM) data. This paper investigates the correlation between bridge acceleration and train speed based on structural dynamics theory and SHM system from three foci. Firstly, the calculated formula of acceleration response under a series of moving load is deduced for the situation that train length is near the length of the bridge span, the correlation between train speed and acceleration amplitude is analyzed. Secondly, the correlation scatterplots of the speed–acceleration is presented and discussed based on the transverse and vertical acceleration response data of Dashengguan Yangtze River Bridge SHM system. Thirdly, the warning indexes of the bridge performance for correlation scatterplots of speed–acceleration are established. The main conclusions are: (1) The resonance between trains and the bridge is unlikely to happen for long-span bridge, but a multimodal correlation curve between train speed and acceleration amplitude exists after the resonance speed; (2) Based on SHM data, multimodal correlation scatterplots of speed–acceleration exist and they have similar trends with the calculated formula; (3) An envelope line of polylines can be used as early warning indicators of the changes of bridge performance due to the changes of slope of envelope line and peak speed of amplitude. This work also gives several suggestions which lay a foundation for the better design, maintenance and long-term monitoring of a long-span high-speed bridge.

Keywords: high-speed trains; long-span bridge; correlation of speed–acceleration; warning index of bridge performance

1. Introduction

High-speed railways have become a reflection of the prosperity of a nation, and the bridges are an essential element on the high-speed railways. When a high-speed train crosses the bridge, the bridge girder will generate serious vibration. Therefore, the response of bridge under high-speed trains and their long-term performance of vibration are required to be investigated.

As trainload is one of the most common load for high-speed railway bridges in serving process, the dynamics theory of bridge under trains become an important research content in bridge engineering. Li *et al.* (2003) studied the natural frequency of bridges under vehicle loads and the vehicle-bridge interaction system for railway simple support bridge. Ju and Lin (2003a), Yang and Lin (2005), Kong *et al.* (2006), Huang (2012) established the finite element (FE) model of the bridge, analyzed the nonlinear vibration responses of the whole bridge, and determined the parameters which affect the dynamic responses under vehicle loads. Lacarbonara and Colone (2007) studied the dynamic response of arch bridges traversed by high-speed trains. Dinh *et al.* (2009) explored the dynamic response based three-dimensional bridge-high-speed train interaction model considering wheel-rail contact.

The high running speed is an important characteristic for high-speed railway bridges, and higher speed trains generally result in a larger dynamic response. So the analysis about the correlation between bridge response and vehicle speed becomes foci in recent years. Many studies focus on the dynamic performance and the resonant speed of small-span railway bridges. Li and Su (1999) established the calculated formula of bridge response for a simply supported girder bridge under high-speed trains and analyzed the resonant train speed of bridge vibration. Ju and Lin (2003b), Martinez-Rodrigo *et al.* (2006), Dehestani *et al.* (2009) investigated the dynamic performance of existing high-speed railway bridges under trainloads and discussed the critical speed of resonance. Xia *et al.* (2010) studied resonance mechanism of short span bridge (20~40 meter) on vertical and transverse direction under moving trainloads. Mao and Lu (2013) analyzed the critical speed of bridge resonance under moving trains and put forward a Z factor to describe the severity of bridge resonance. In the field of vehicle induced vibration of long-span bridges, Wang *et al.* (2015) investigated the dynamic response of a cable-stayed concrete-filled steel tube arch bridge under vehicle loading and analyzed the correlation of impact factors (IMs) of bridge response and vehicle speeds.

With the wide application of testing technology in civil engineering, the dynamic response of long-span bridges under high-speed trains can be accurately measured. Some promising works about vehicle induced vibration of bridges,

∗Corresponding author, Senior Engineer
E-mail: meidapeng@qq.com

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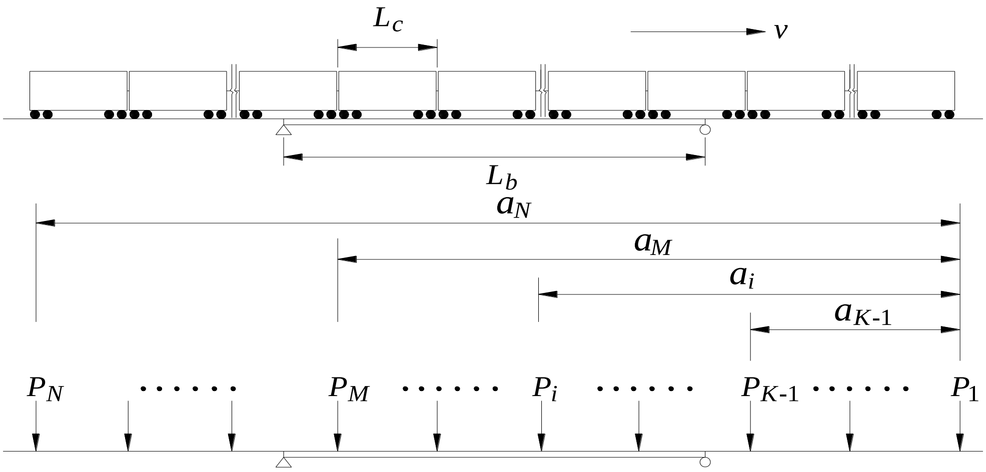


Fig. 1 A long-span bridge subjected to a series of moving loads

which base on field test or structural health monitoring (SHM) system, have been conducted in recent years. Ni *et al.* (2010, 2012) and Ye *et al.* (2012, 2017) conducted the research on the vibration-induced fatigue life assessment of a suspension bridge carrying both the highway and railway traffic using long-term dynamic strain monitoring data. Ju *et al.* (2009) studied dominant frequencies of train-induced vibrations of bridges by acceleration sensors. Xia and Nan (2005), He *et al.* (2008), Mellat *et al.* (2014), Feng and Feng (2015) used the results of a field test or SHM system to update the FE model and analyzed the dynamic response of railway bridges. Ashebo *et al.* (2007) evaluated the dynamic load factor of a skew box girder continuous bridge by field test data. Wiberg and Karoumi (2009), Kosnik and Dowding (2013) analyzed the dynamic behavior of a long-span railway bridge depending on SHM data. Yi *et al.* (2013a,b) and Ye *et al.* (2015, 2016) assessed the service performance of the bridge structure using field test data.

The vibration of a long-span railway bridge under high-speed trains receives much concern since its construction. But the current researches and design codes of each country (AREMA 2007; National Railway Administration of the People's Republic of China 2014) about bridge dynamics theory under high-speed trains are most for short span bridge (below 40 meters), and most of those studies have not be validated by field test or SHM system. As more and more field test data or SHM data is used for the vibration research of long-span bridges under high-speed trains, a detailed study of the correlation between long-span bridge vibration and train speed is required. The present paper investigates the train-bridge vibration of long-span railway bridges based on structural dynamics theory and SHM system. As a typical long-span high-speed railway bridge, the correlation between train speed and bridge acceleration response of Dashengguan Yangtze River Bridge is analyzed. The warning indexes of bridge dynamic performance under high-speed trains are established. Several suggestions are given which lays a foundation for the better design, maintenance and long-term monitoring of a long-span high-speed bridge.

2. Calculation formula of train-bridge vibration

The vibration of train-bridge system for short- and medium-span bridges, whose length is near the length of a carriage of the train, has been deeply studied (Li and Su 1999; Yang and Lin 2005; Xia *et al.* 2006; Mao and Lu 2013). The resonance between bridge and train is a precondition in those studies. But for bridges whose length are near the length of trains (e.g., the Dashengguan Yangtze River Bridge), their vibration performance and the possibility of resonance under high-speed trains needed a further research. Define L_b is the bridge length, L_c is the length of a carriage, N is the total number of carriages, K-1 is the last carriage leaving the bridge, M is the last carriage on the bridge (Fig. 1).

The analysis of the bridge vibration can include the fundamental mode alone. It would be reasonable to expect the displacement to vary with x and t as distinctly separate functions. Therefore, we assume that

$$y(x,t) = \phi(x)q(t) \tag{1}$$

where $y(x,t)$ is the vertical displacement, $\phi(x)=\sin(n\pi x/L_b)$ is the fundamental mode shape, $q(t)$ are the generalized co-ordinates that define the amplitude of vibration with time.

Let the first and the last moving loads on the bridge be P_K and P_M at time t (Fig. 1). If the fundamental mode of the long-span bridge alone is taken into account, the equation of vibration motion in generalized co-ordinates under a series of moving loads at a constant speed can be expressed as

$$\ddot{q}(t) + 2\xi\omega\dot{q}(t) + \omega^2 q(t) = \frac{2}{mL_b}\sum_{i=K}^{M}\sin\frac{\pi(vt-a_i)}{L_b}P_i \tag{2}$$

where m is the mass per unit length, ω is the first natural frequency (circular), ξ is the damping ratio, v is the velocity of moving loads (trains), P_i is the ith moving load, $a_i=L_c(i$-$1)$ is the distance between moving loads P_1 and P_i. The solution of $q(t)$ may be sought by first examining the forced vibration of the bridge under a single moving load and the free vibration after the passage of the single load and subsequently using the principle of superposition. For the convenience of the superposition operation, consider the single moving load as the *i*th load in the moving load series, and let t=0 be the time when the first moving load enters the bridge. Thus, the forced vibration caused by the ith moving load takes place when $a_i/v \le t \le a_i/v + L_b/v$. Specializing Eq. (2) for the *i*th single moving load only, the solution can be expressed as the combination of a steady (forced) term and a transient (free) term as

$$q(t)=\frac{\beta}{1-\beta^2}\sin\frac{\pi(vt-a_i)}{L_b}-\frac{y_{st}\beta}{1-\beta^2}e^{-\xi\omega(t-a_i/v)}\sin\omega(t-\frac{a_i}{v}) \quad (3)$$

where $y_{st}=PL_b^3/48EI$ is static midspan deflection with moving load P acting on the midspan. $\beta=\pi v/L_b\omega$ with ω being the bridge natural frequency (circular).

For the time $t \ge t' = a_i/v + L_b/v$, the free vibration after the passage of the single (*i*th) moving load is

$$q(t)=e^{-\xi\omega(t-t')}\left[\frac{\dot{q}_i(t')+\xi\omega q_i(t')}{\omega}\sin\omega(t-t')+q_i(t')\cos\omega(t-t')\right] \quad (4)$$

where $q_i(t')$ and $\dot{q}_i(t')$ are the displacement and velocity excited by the *i*th load. Substituting Eq. (3) into the $q_i(t')$ and $\dot{q}_i(t')$ in Eq. (4), the Eq. (4) can express as

$$q(t)=e^{-\xi\omega(t-a_i/v-L_b/v)}\frac{y_{st}\beta}{1-\beta^2}\sin\omega(t-\frac{a_i}{v}-\frac{L_b}{v})-e^{-\xi\omega(t-a_i/v)}\frac{y_{st}\beta}{1-\beta^2}\sin\omega(t-\frac{a_i}{v}) \quad (5)$$

According to Eqs. (3) and (5), the bridge response under a series of moving loads can be established using superposition. Consider a generic scenario of (K-1), for which moving loads have crossed the bridge, while (M-K+1) moving loads are acting on the bridge, and the bridge response (Li and Su 1999) at the time $a_{K-1}/v+L_b/v \le t \le a_K/v+L_b/v$ can be expressed as

$$\begin{aligned}q(t)=&\frac{y_{st}}{1-\beta^2}\times\sum_{i=K}^{M}\sin\frac{\pi(vt-a_i)}{L_b}-\frac{y_{st}\beta}{1-\beta^2}\times\sum_{i=K}^{M}e^{-\xi\omega(t-a_i/v)}\sin\omega(t-\frac{a_i}{v})\\&-\frac{y_{st}\beta}{1-\beta^2}\times\sum_{i=1}^{K-1}e^{-\xi\omega(t-a_i/v-L_b/v)}\sin\omega(t-\frac{a_i}{v}-\frac{L_b}{v})-\frac{y_{st}\beta}{1-\beta^2}\times\sum_{i=1}^{K-1}e^{-\xi\omega(t-a_i/v)}\sin\omega(t-\frac{a_i}{v})\end{aligned} \quad (6)$$

In the Eq. (6), the first two terms represent the bridge the first two terms represent the bridge vibration caused by the acting of (M-K+1) moving loads, the first one is a forced vibration term and the second one is a free vibration term; the last two terms are the bridge free vibration caused by the preceding (K-1) moving loads. Analyzing the last three-term (free vibration) of Eq. (6), the free vibration response neglecting damping can be written as follows

$$q_f(t)=-\frac{y_{st}\beta}{1-\beta^2}\times\sum_{i=1}^{M}\sin\omega(t-\frac{a_i}{v})-\frac{y_{st}\beta}{1-\beta^2}\times\sum_{i=1}^{K-1}\sin\omega(t-\frac{a_i}{v}-\frac{L_b}{v}) \quad (7)$$

The acceleration response of bridge free vibration can be expressed as the second derivative of Eq. (7)

$$q_f''(t)=\frac{y_{st}\omega^2\beta}{1-\beta^2}\times\sum_{i=1}^{M}\sin\omega(t-\frac{a_i}{v})+\frac{y_{st}\omega^2\beta}{1-\beta^2}\times\sum_{i=1}^{K-1}\sin\omega(t-\frac{a_i}{v}-\frac{L_b}{v}) \quad (8)$$

To facilitate the derivation, two expansions of triangular series must be given

$$\sum_{i=1}^{m}\sin(a-ix)=\sum_{i=1}^{m}\left[\sin a\cos ix-\cos a\sin ix\right] \quad (9)$$

$$\begin{cases}\sum_{i=1}^{m}\sin ix=\sin\frac{mx}{2}\sin\frac{(m+1)x}{2}\csc\frac{x}{2}\\ \sum_{i=1}^{m}\cos ix=\sin\frac{mx}{2}\cos\frac{(m+1)x}{2}\csc\frac{x}{2}\end{cases} \quad (10)$$

Substituting Eqs. (9) and (10) into the Eq. (8), and $a_i=L_c(i$-1), the Eq. (8) can express as

$$\begin{aligned}q_f''(t)=&\frac{y_{st}\omega^2\beta}{1-\beta^2}\times\frac{\sin\frac{ML_c\omega}{2v}\sin\left[\omega t-\frac{(M+1)L_c\omega}{2v}\right]}{\sin\frac{L_c\omega}{2v}}\\&+\frac{y_{st}\omega^2\beta}{1-\beta^2}\times\frac{\sin\frac{(K-1)L_c\omega}{2v}\sin\left[\omega t-\frac{\omega L_b}{v}-\frac{KL_c\omega}{2v}\right]}{\sin\frac{L_c\omega}{2v}}\end{aligned} \quad (11)$$

Merging the two terms in Eq. (11) and it can express as

$$q_f''(t)=A\sin(\omega t-\theta) \quad (12)$$

where

$$A=\frac{y_{st}\omega^2\beta}{(1-\beta^2)\times\sin\alpha}\sqrt{A_1^2+A_2^2},\quad \tan\theta=\frac{A_1}{A_2},\quad \alpha=\frac{L_c\omega}{2v}$$

$$A_1=\sin M\alpha\sin(M+1)\alpha+\sin(K-1)\alpha\sin(\frac{\omega L_b}{v}+K\alpha),$$

$$A_2=\sin M\alpha\cos(M+1)\alpha+\sin(K-1)\alpha\cos(\frac{\omega L_b}{v}+K\alpha),$$

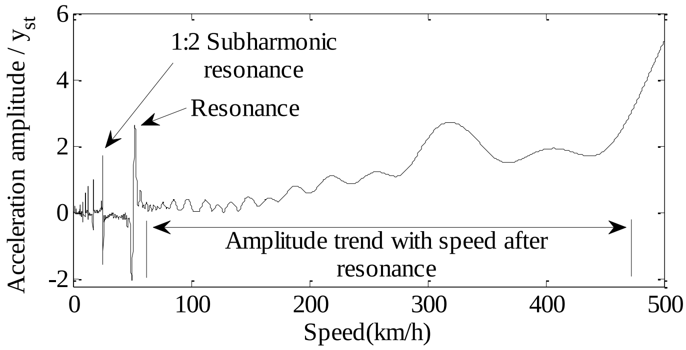
According to the length of the main span of the Dashengguan Yangtze River Bridge, and the parameters of CRH3 Electric Multiple Units which serving on the Dashengguan Yangtze River Bridge (Zhao *et al.* 2016), determine the length of span and length of a carriage by L_b=336 m, L_c=25 m. The value of ω refers to the first vertical and transverse natural frequency from monitoring data (Fig. 5). The typical correlation between amplitudes of acceleration of bridge and train speeds is shown in Fig. 2. In Fig. 2(a), $\omega=2\pi\times0.561$, M=16, K=15; in Fig. 2(b), $\omega=2\pi\times0.689$, M=16, K=6.

In generalized single-degree-of-freedom system. A simple support bridge under a series of moving load will occur resonance at some critical moving speed. The critical speed of resonance (Xia *et al.* 2006) can be expressed as

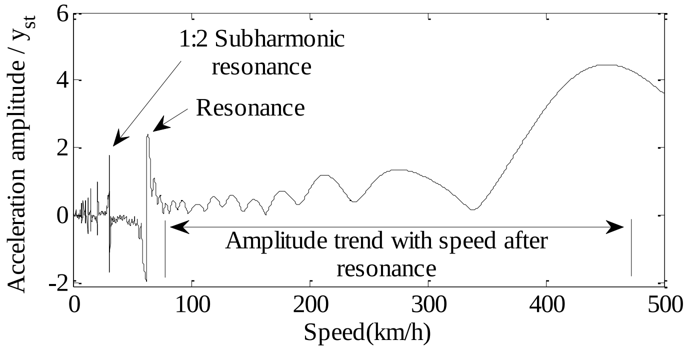
$$V_{cr}=\frac{3.6\cdot f_n\cdot L_c}{i}\quad(n=1,2\cdots;i=1,2\cdots)\quad(\text{unit}:km/h) \quad (13)$$

where f_n is the nth natural frequency of bridge, if considering a vehicle-bridge system, f_n is the effective natural frequency of vehicle-bridge system; i is used for the critical speed of superharmonic resonance.

As show in Fig. 2(a), the ratios of acceleration amplitude and y_{st} occur two extremums at 25.245 km/h (critical speed of 1:2 subharmonic resonance) and 50.49 km/h (critical speed of resonance), they are consistent with the results of Eq. (13). After the critical speed of resonance, a correlation curve between train speed and acceleration



(a) transverse acceleration



(b) vertical acceleration

Fig. 2 Typical acceleration amplitude trend with speed

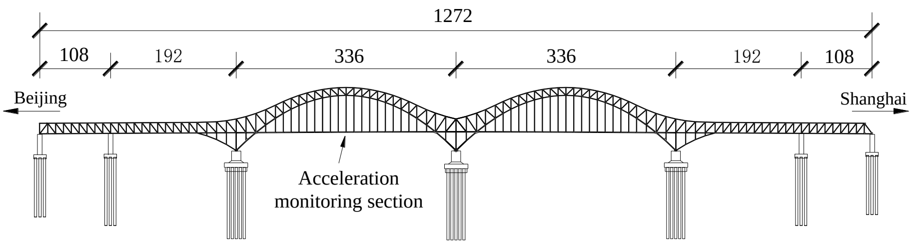


Fig. 3 Dashengguan Yangtze River Bridge (Units: m)

amplitude appears, and the amplitudes of acceleration vary with speed increasing at complicated trigonometric curve which has several peak points. As well as the Figs. 2(b), after 1:2 subharmonic resonances occurring at 31.005km/h, resonance occurring at 62.01 km/h, there is a correlation curve between train speed and acceleration amplitude.

For a long-span simple support bridge under a series of moving load, it can be found from Eq. (12) and Fig. 2 that: The resonant speeds of a long-span high-speed bridge (e.g., the Dashengguan Yangtze River Bridge) are usually below 100 km/h, the resonance between trains and the bridge is unlikely to happen in daily operations. But a multimodal correlation curve between train speed and acceleration amplitude appears after the resonance speed. The acceleration amplitude of bridge vibration are affected by the following parameters: (1) the train speed (moving load speed); (2) natural frequency of bridge; (3) the span of the bridge; (4) the length of a carriage of train; (5) the number of the carriages on the bridge; (6) the number of the carriages leaving the bridge. In this paper, the boundaries of bridge and damping can also influence the characteristic of bridge acceleration response which does not be introduced into the calculated formula.

3. Measurement system and field test result for acceleration of bridge

3.1 Measurement system of bridge acceleration response

Dashengguan Yangtze River Bridge, shown in Fig. 3, is the longest 6-track high-speed railway bridge in the world. With 336-m-long main spans and 6 tracks, it consists of six tracks that include two lines on the downstream side for the Beijing-Shanghai (B-S) high-speed railway, two lines on the upstream side for the Shanghai-Wuhan-Chengdu (S-W-C) railway and two lines on the outer sides of the bridge deck for the Nanjing Metro.

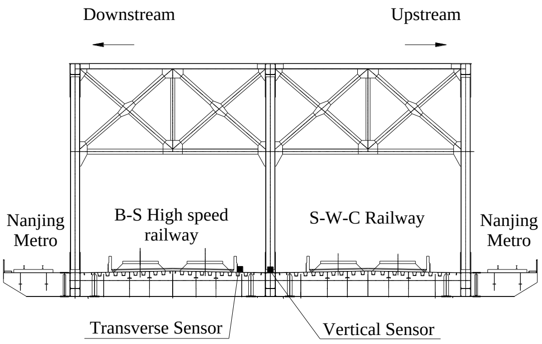


Fig. 4 Locations of the acceleration sensors

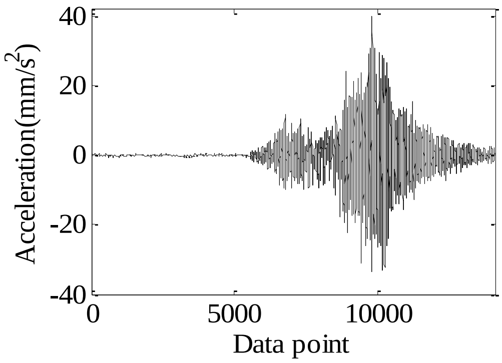
This bridge is the longest high-speed railway bridge and has the heaviest design load. The design train speed of 300 km/h is among the highest in the world. As shown in Fig. 3, the design of the bridge consists of two continuous steel truss arches and approach spans with a span configuration of 108+192+2×336+192+108 m. As shown in Figs. 4, the arches feature three main trusses in the transverse direction spaced 15 m apart and integrated joints for connecting the truss members. High-strength S420Q-grade steel (yield strength of 420 MPa) was used. The structure includes an orthogonal steel deck system. Spherical steel bearings were used for the supports.

Because of the unusual characteristics of the Dashengguan Yangtze River Bridge, including the long-span of the girders, the heavy design loading and the high train speeds, a long-term SHM system was installed. In this monitoring system, acceleration sensors are employed to measure the dynamic response of bridge vibration. The monitoring section of these acceleration sensors is shown in Fig. 3. The monitoring section is at the middle of the main span near the Beijing side, a transverse acceleration sensor and a vertical acceleration sensor are employed. The location of sensors in their monitoring section is shown in Fig. 4. The sampling rate of the acceleration sensors is 200 Hz, which allows for vibration measurements by the monitoring system.

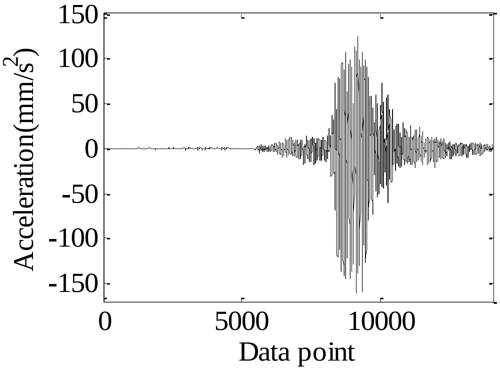
The SHM system on the Dashengguan Yangtze River Bridge includes fiber Bragg-grating (FBG) thermal sensors to monitor the temperature and speedometers to monitor the train speed distribution over the bridge. All the acceleration data used for the analysis in this study is accompanied with temperature data and speed data.

3.2 Field test result

The acceleration data of all monitoring section on the Dashengguan Yangtze River Bridge is filtered with a low pass digital filter of 20 Hz, and the acceleration responses of bridge girder under high-speed trains are obtained. Fig. 5 shows a typical acceleration monitoring results of transverse vibration and vertical vibration for a train crossing.

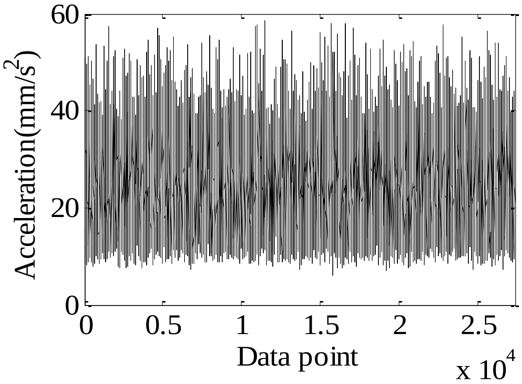


(a) Transverse vibration

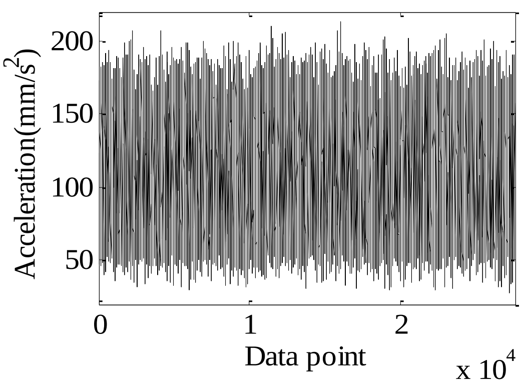


(b) Vertical vibration

Fig. 5 Typical acceleration monitoring results for a train crossing

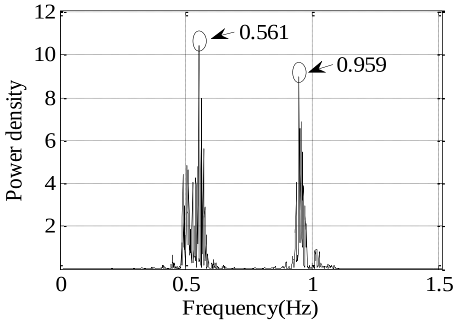


(a) Transverse vibration

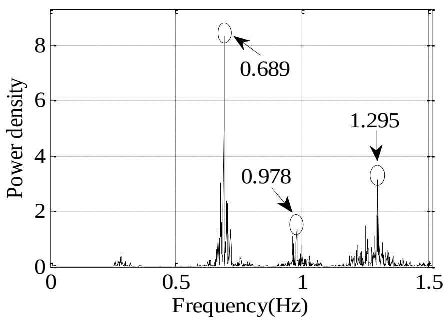


(b) Vertical vibration

Fig. 6 Maximum values for each train crossing from January 1, 2014 to December 31, 2015

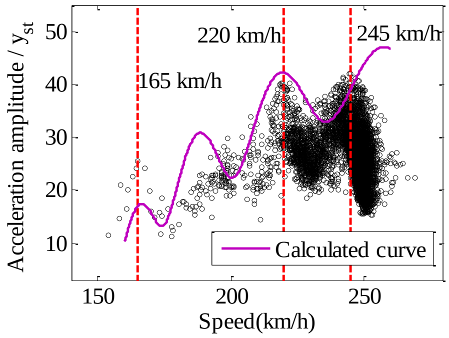


(a) Transverse vibration

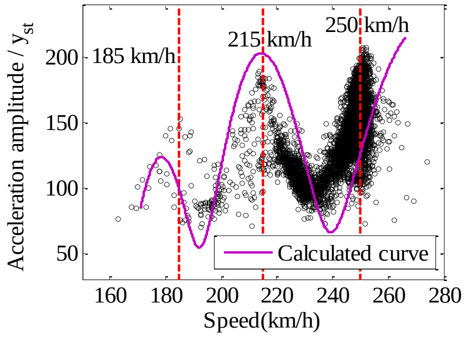


(b) Vertical vibration

Fig. 7 Power spectrum of girder vibration



(a) Transverse



(b) Vertical

Fig. 8 Correlation scatterplots about the transverse acceleration and train speed

As it can be seen in Fig. 5, when a train crossing the bridge, the vibration of bridge girder showed significant non-stationary vibration characteristics and the vertical acceleration response is significantly greater than the transverse acceleration response.

The maximum values (absolute value) of every acceleration sensor data for each train crossing from January 1, 2014 to December 31, 2015 are calculated for long-term vibration analysis of the Dashengguan Yangtze River Bridge. Fig. 6 shows the maximum values (absolute value) for each train crossing from January 1, 2014 to December 31, 2015. As can be seen in Fig. 6, the maximum values of acceleration showing a significant randomness, and the correlation of the speed–acceleration should a further research.

Fig. 7 shows the auto power spectrum of girder vibration of the Dashengguan Yangtze River Bridge based on the monitoring data of SHM system. The first two order of transverse natural frequencies of the bridge are 0.561 and 0.959; the first three order of vertical natural frequencies of the bridge are 0.689, 0.978, and 1.295. It can be concluded that the model parameters results of bridge girder based on field monitoring data are close to the field test results and FE model results (Meng *et al.* 2015; Zhao *et al.* 2016).

4. Correlation of speed-acceleration based on monitoring data

The vibration of the bridge structure was influenced by the speed of trains, the trains on Dashengguan Yangtze River Bridge are the Electric Multiple Units (EMU), they generally have 16 carriages. Based on the acceleration date on the SHM system of Dashengguan Yangtze River Bridge, the maximum of the acceleration amplitudes for each train crossing are obtained. The train speeds for each train crossing come from the data of speedometers on the bridge.

For analyzing the correlation between the acceleration response of bridge girder and train speed, the maximum of acceleration data and the train speed data for each train crossing are graphed as correlation scatterplots. Fig. 8 shows the correlation scatterplots about the maximum of acceleration and train speed on transverse and vertical direction.

As shown in Fig. 8, the train speeds of transverse acceleration data are at the range of 150 km/h~280 km/h, the correlation scatterplots have 3 speed point with much higher acceleration amplitude than another speed band, they are 165 km/h, 220km/h and 245 km/h; the train speeds of vertical acceleration data are at the range of 160 km/h~280 km/h, the correlation scatterplots have 3 speed point with much higher acceleration amplitude than other speed band, they are 185 km/h, 215 km/h and 250 km/h.

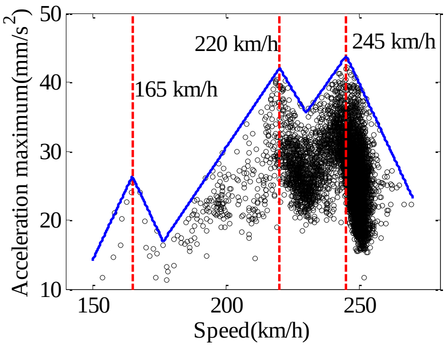
The following observations can be made from Fig. 8:

1. During the operation of trains (train speed at 150~280 km/h) on the Dashengguan Yangtze River bridge, the transverse and vertical acceleration amplitude of bridge girder will enhance at some special train speed, which can be called peak speed of amplitude.

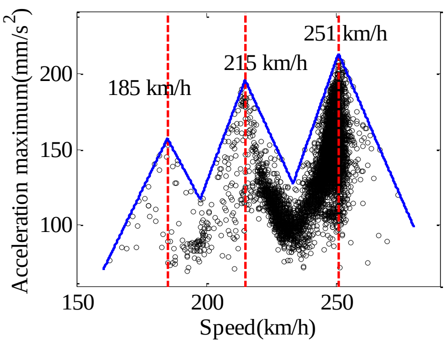
2. The correlation scatterplots of acceleration amplitude maximum and speed based on monitoring data have similar trends with the calculated formula. But because the monitoring result is influenced by structure damping, and

Table 1 Fitting polylines of the early warning indicators

Direction /Sensor	Speed zone (km/h)	Fitting polylines	Direction /Sensor	Speed zone (km/h)	Fitting polylines
Transverse	150-165	y= 0.8154x-108.2	Vertical	160-185	y= 3.5x-490.4
	165-176.5	y= -0.8205x+161.7		185-197.7	y= -3.157x+741.2
	176.5-220	y= 0.5809x-85.69		197.7-215	y= 4.538x-780.1
	220-230	y= -0.655x+186.2		215-233.5	y= -3.692x+989.3
	230-245	y= 0.5516x-91.31		233.5-251	y= 4.889x-1014
	245-270	y= -0.8289x+246.9		251-280	y= -3.929x+1199



(a) Transverse



(b) Vertical

Fig. 9 Power spectrum of girder vibration

wheel–rail contact of train–bridge system, boundaries of the bridge, the trends of correlation scatterplots based on monitoring data cannot fully consistent with the calculated formula. Meanwhile, the calculated formula in this paper can only describe the response caused by a single mode (fundamental mode), but the vibration responses from SHM data are the superposition of several modes.

5. Warning index of bridge performance based on correlation of speed-acceleration

As the analysis of the calculated formula and monitoring results for the correlation between train speed and acceleration amplitude, for a long-span high-speed railway bridge in serving, the trend of correlation scatterplots will be influenced by the stiffness variation of bridge (influenced degeneration of structure performance) and the natural frequency of bridge (changes of bridge boundary and degeneration of structure performance). The variation of the correlation scatterplot trend (in each direction) based on long-term monitoring data will reflect the performance variation of the bridge in serving. So warning indexes of the bridge performance based on the correlation between train speeds and acceleration amplitude can play an important role in bridge monitoring.

For a typical correlation scatterplots between train speeds and acceleration amplitude, used the peak speed point, an envelope line of polylines can be used as early warning indicators of the changes of bridge performance.

The polylines not only can be used as the early warning threshold of acceleration amplitude under different train speeds. Moreover, when the bridge performance occurs mutation, the slope of envelope line will change dramatically. Fig. 9 shows the early warning indicators of bridge performance for the typical correlation scatterplots of speed–acceleration on transverse and vertical direction. Table 1 shows the fitting polylines of the early warning indicators of bridge performance for the typical correlation scatterplots.

6. Conclusions

In this study, the calculated formula of acceleration response under a series of moving load is deduced, and the correlation between train speed and acceleration amplitude is analyzed for the situation that train length is near the length of the bridge span. The transverse and vertical vibrations under high-speed trains are gained based on the acceleration data of Dashengguan Yangtze River Bridge SHM system. The correlation scatterplots of the speed–acceleration is obtained based on SHM monitoring data. The temperature effect on correlation scatterplots of the speed–acceleration is investigated. The warning indexes of the bridge performance for correlation of speed–acceleration are presented. The conclusions can be summarized as follows:

(1) The calculated formulas of acceleration response for a long-span bridge under a series of moving load have been given based on the classic structural dynamics. The calculated formulas of an acceleration response of bridge free vibration indicate that the resonance between trains and the bridge is unlikely to happen in daily operations for a

long-span high-speed bridge. A multimodal correlation curve between train speed and acceleration amplitude exists after the resonance speed. The acceleration amplitude of bridge free vibration is affected by train speed, the natural frequency of bridge, a span of the bridge, the length of a carriage of the train, the number of the carriages on the bridge, number of the carriages leaving the bridge.

(2) The transverse and vertical acceleration amplitude of bridge girder will enhance at some special train speed point (peak speed of amplitude) in field monitoring. The multimodal correlation scatterplots of the speed–acceleration have similar trends with the calculated formula.

(3) For a long-span high-speed railway bridge in long-term serving, an envelope line of polylines can be used as early warning indicators of the changes of bridge performance due to the changes of slope of envelope line and peak speed of amplitude.

Several suggestions are given as follows. In the process of design, the maintenance and the long-term monitoring of a long-span high-speed railway bridge, the operational speeds of the train should be avoided at a peak speed of bridge vibration. The vibration of the bridge under the train with peak speed is required to be paid more attention.

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