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Experimental analysis of a composite bridge under high-speed train passages

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Abstract

Evaluation of resonances phenomenon in high-speed railway bridges during train passage is a dominant issue in order to confirm the structural safety of the bridge and the stability of the ballast on bridge deck. The Sesia viaduct, located in the Turin-Milan high-speed rail line, is one of the most investigated high-speed railway bridges. The maximum accelerations and the resonance train speed were predicted while the bridge vibration modes and the effect of adjacent spans on these were estimated through some detailed experimental and numerical investigations. Nevertheless, some important points remain unclear, such as the actual resonance speed and the influence of deck local vibrations. To verify and clarify these issues, experimental dynamic analyzes based on acceleration measurements of the Sesia viaduct under ETR1000 train passages with speed up to 374km/h were conducted in this study. In the measurements, accelerometers were installed not only on steel box girder but also on concrete deck slab in order to identify local vibration modes of deck members so as to estimate their effect on the evaluation of the accelerations. Comparing the maximum accelerations up to 15Hz with various train speeds, the resonance speed corresponding to the first bending mode of the Sesia viaduct was identified. Furthermore, an analysis based on the identification of the natural frequencies clarified that the high-order resonances between passing train and deck local vibration modes have the largest impact on the maximum acceleration up to 30Hz which is the limit used for the verification of ballast stability in the Eurocode.

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Keywords: High-speed railway; Composite bridge; Deck acceleration; Modal identification; Local vibration mode; Resonance

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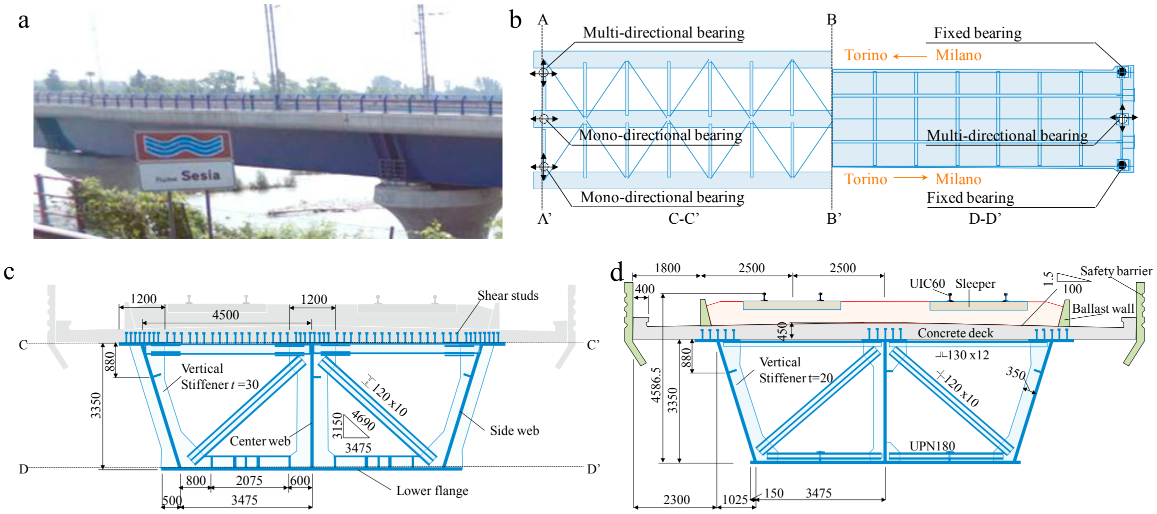

Fig. 1. General overview of the Sesia viaduct. (a) Global view; (b) view from top; (c) side view at A-A; (d) side view at B-B.

1. Introduction

The resonance of railway bridges caused by the high speed (HS) train passage is an important issue related to running safety, ride comfort, structural fatigue and cracking, thus, various standards are established for the design, maintenance and speed up. In the European HS railway with trains speeds over 200 km/h, bridge decks that are supporting ballast have a limitation for the acceleration [1]. This standard establishes the maximum value of the deck acceleration up to 30 Hz in order to prevent destabilization of ballast due to bridge vibration during HS train passage. However, it is still possible to elucidate the vibration phenomenon during HS train passages developing the evaluation method considering not only the basic vibration modes of the entire bridge but also the local vibration modes of deck members [2]. Especially, natural frequencies of deck members are expected to be under 30 Hz (considering their general sizes and materials), thus, deck local modes may be able to affect to the evaluation of the deck acceleration. On the other hand, deck local modes and their dynamics remain almost unexplained.

Considering the above background, this study investigates the influence of local vibrations of deck members on the acceleration evaluation of a steel/concrete composite bridge by means of a modal analysis on multi-point acceleration measurements.

2. Measurement and analysis method

2.1. Test bridge

Fig.1 shows an outline of the test bridge, the Sesia viaduct. This bridge is a steel/concrete composite bridge of the Turin-Milan HS railway in Italy with a span length of 43.6 m. 15 diaphragms are installed at steel box girder at 3.114 m intervals. Precast concrete deck with a width of 13.6 m and a thickness of 0.4 m is synthesized on shear studs into a two-box type steel girder with a width of 10.2 m and a height of 3.35 m. The rails are UIC60 with concrete sleepers which are installed on the ballasted double type track at 0.6 m intervals.

2.2. Measurement method

Table 1 and Fig. 2 show the specifications of the measurement equipments and their arrangement. In order to measure the train speed during the train passages and the accelerations, two laser light detection systems using photodiodes and 10 accelerometers were installed respectively. Six servo type accelerometers were installed at the position G1 - G6 in Fig. 2 (a) to outline the global bridge dynamics. Regarding the bridge deck, three piezoelectric accelerometers were also installed at the position D1 - D3 in Fig. 2 (a) for the line bound for Milan. Two pairs of laser detection systems were installed at a 110 m interval on wire poles at Sesia viaduct, and these were used to trigger the automatic measurement and to evaluate the train running speed. The signal of each sensor was recorded on the laptop PC at the sampling frequency of 2 kHz via A/D conversion.

2.3. Test train

This study targeted 8-car train, ETR1000, for speed up test. The weight of the test train has been adjusted from 75.6 to 81.8 kN at each axles. The train speed was gradually changed from 280 km/h up to the maximum of 374 km/h. Measurements were conducted during the passage of trains bound for Milan, and the bridge accelerations were measured during a total of 32 train passages.

2.4. Signal processing and identification methods

The acquired acceleration data were resampled to a sampling frequency of 200 Hz and converted to 15HzLPF and 30HzLPF accelerations by means of a band pass filter with 0.5-15 Hz and 0.5-30 Hz pass bands respectively. 15HzLPF and 30HzLPF data were used for the analysis of the global flexural vibration and for the evaluation of the deck acceleration using the Eurocode criterion. Regarding the residual response after a train passage, due to free vibrations, both the entire and the local vibration modes of the Sesia viaduct were identified through a ERA (Eigen Realization Algorithm) method which is based on multipoint acceleration responses [3]. The modal characteristics were identified by the ERA method with a 40 DOF system using a 5 seconds temporal window after a train passage.

Table 1. List of specifications of measurement equipments.

Instruments	Model	Specifications
Servo accelerometer for girder	JA-5L15	Freq. range: 0-2kHz, Sensitivity: 1.0V/g
Capacitive accelerometer for slab	141B	Freq. range: 0-3kHz, Sensitivity: 0.5V/g
Photocells	E3Z-LR86	0-10V switching
Data acquisition system	Ni cDAQ-9172	Sampling frequency: 2kHz

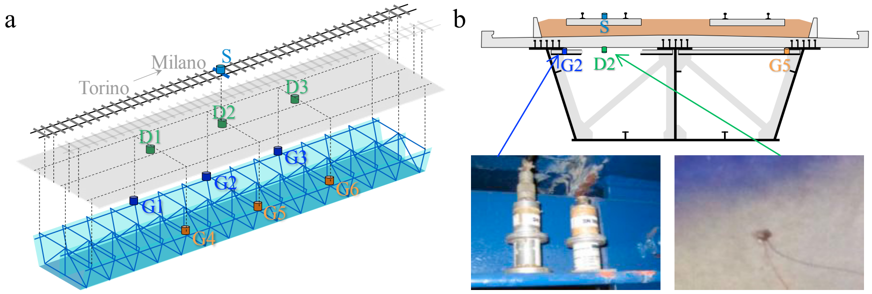

Fig. 2. General overview of the sensor arrangements (a) and detail of the sensor locations at the mid-span (b).

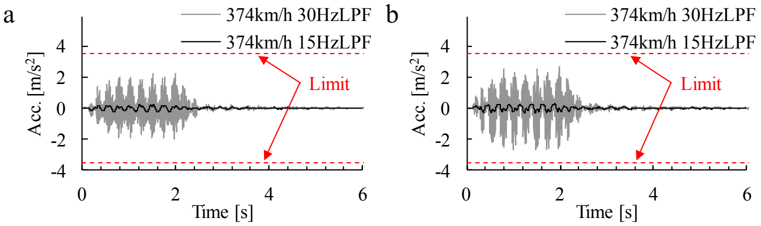

Fig. 3. Acceleration responses during a train passage at: (a) girder G2; (b) deck D2.

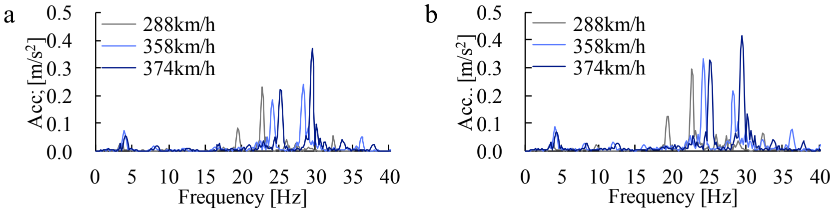

Fig. 4. Acceleration spectra during three trains passages at; (a) girder G2; (b) deck D2.

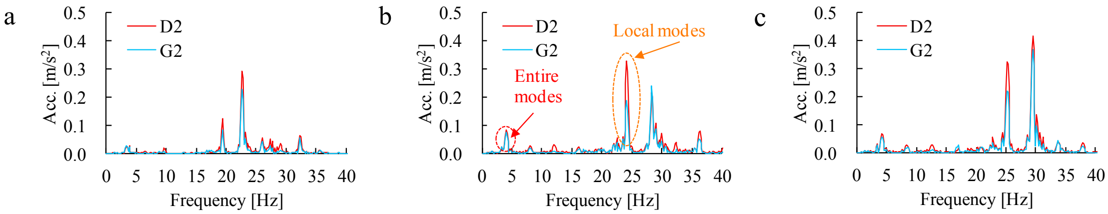

Fig. 5. Spectra comparison between girder G2 and deck G2 the train speed of; (a) 288km/h; (b) 358km/h; (c) 374km/h.

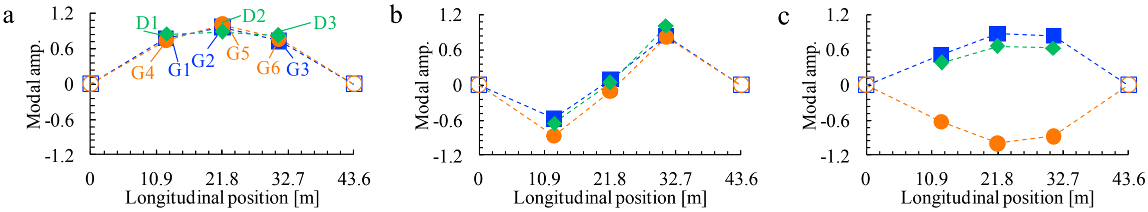

Fig. 6. Identified global modes; (a) first bending: 3.66 Hz; (b) second bending: 9.44 Hz; (c) first torsional: 9.13 Hz.

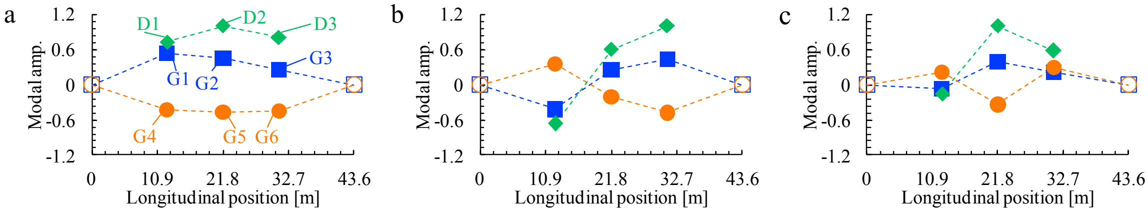

Fig. 7. Identified deck local modes; (a) first deck local: 20.40 Hz; (b) second deck local: 21.13 Hz; (c) third deck local: 23.68 Hz.

Table 2. Identified natural frequencies and modal damping ratio and its variations

Mode	Natural frequency [Hz] Ave.	Natural frequency [Hz] Std./Ave.	Modal damping ratio Ave.	Modal damping ratio Std./Ave.
First bending	3.66	0.019	0.046	0.640
Second bending	9.44	0.029	0.038	0.603
Third bending	17.82	0.023	0.036	0.385
First torsional	9.13	0.015	0.027	0.361
Second torsional	18.64	0.018	0.028	1.337
Third torsional	20.04	0.014	0.020	0.648
First deck local	20.40	0.004	0.013	0.370
Second deck local	21.13	0.004	0.015	0.320
Third deck local	23.68	0.011	0.011	0.415
Fourth deck local	24.15	0.002	0.011	0.394
Fifth dech local	27.49	0.007	0.015	0.597

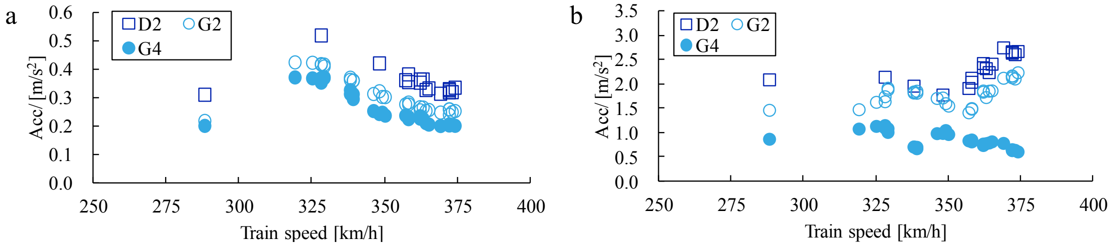

Fig. 8. Maximum acceleration at girder G2, G4 and deck D2 vs train speed; (a) 15HzLPF; (b) 30HzLPF.

3. Measurement results

3.1. Time series

Fig. 3 shows 15HzLPF and 30HzLPF accelerations at two measurement points with a train speed of 374 km/h. The maximum values for both the G2 and D2 are lower than the acceleration threshold of 3.5 m/s^2. Much smaller amplitude of 15HzLPF than that of 30HzLPF expresses that the lower vibration modes are not dominant components in 30HzLPF acceleration. In addition, the larger amplitude of the deck can indicate that measuring the girder acceleration to evaluate the deck acceleration leads to an underestimation and possible safety hazards.

3.2. Spectrum

Fig. 4 shows the PSD spectra of the accelerations measured on the two points at the train speed of 288, 358 and 374 km/h. The peak frequencies shift to the high side at any measurement points due to the increase in train speed. The larger peak amplitude at 20-30 Hz than that around 4 Hz implies that the accelerations at 20-30 Hz are the main components of 30HzLPF acceleration. Fig. 5 shows spectrum comparison between girder G2 and deck D2. As shown in Fig. 5 (b), peak around 4 Hz is caused by global vibration mode since they are of same amplitude in both the members. On the other hands, the peak of deck spectrum around 25 Hz has approximately twice amplitude of the girder. It means that the local vibration of deck member contributes to the 30HzLPF acceleration.

3.3. Modal properties

Fig. 6 and Fig. 7 show six examples of identified entire and deck local vibration modal shapes. In the deck local modes shown in Fig. 7, modal amplitudes of the deck are equal to or larger than that of the girder. Natural frequencies of the deck local modes stand in the band 20-30 Hz that overlaps with the frequency band of some peaks of Fig. 5. This fact emphasizes that the deck local modes contribute to the evaluation of the 30HzLPF acceleration.

Table 2 shows the identified natural frequencies, modal damping ratios and their variations. Although modal damping ratio has a larger variation, it can be asserted that the mode damping ratios of the global modes are larger than the ones of the deck local modes at higher frequencies.

3.4. Global Resonance

Fig. 8 (a) shows the maximum acceleration of 15HzLPF acceleration at girder (G2 and G4) and at deck (D2) as a function of the train speed. The maximum value of deck accelerations is 1.3 times the ones of the passing side girder. Fig.8 (a), which mainly includes the first bending mode vibration, confirm the resonance peak at around 330 km/h. This result corresponds approximately to the 3.7 Hz natural frequency of the first mode. Several literatures [2] have indicated that the first mode also exists near 4.2 Hz due to interaction with adjacent spans, and numerical analyzes were conducted with model adjusted to 4.2 Hz. However, this study proved that the actual resonance speed corresponds to the first bending mode of 3.7 Hz. This means that the interaction between adjacent spans has little influence on the response during train passages.

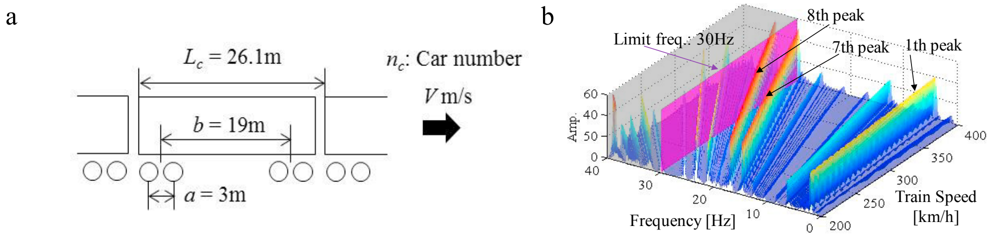

Fig. 9. Axle arrangement of the ETR1000 train (a) and excitation spectrum of the ETR1000 train (b).

3.5. Local Resonance

Fig. 8 (b) shows the 30HzLPF maximum acceleration at the girder and the deck as function of the train speed. In 30HzLPF acceleration, the maximum acceleration greatly differs between the passing and non-passing side girder (G2 and G4). The maximum value of girder acceleration up to 374 km/h is 2.24 m/s^2 in this test series. The maximum value of deck acceleration is 2.76 m/s^2, which is 1.3 times the girder acceleration on average. This means that when the girder acceleration is substituted with the deck one, the response value is underestimated by nearly 25%. Therefore, in the examination of the European acceleration criterion based on actual measurement, it is necessary to measure the deck member on the train passing side.

Furthermore, several peaks of the deck acceleration of Fig. 8 (b) cannot be observed in Fig. 8 (a). These peaks may relate to high-order resonance phenomena between structural modes in the band 15-30 Hz and cyclic excitation of the passing train. Considering the axial arrangement of the ETR1000 train (Fig. 9), if each axle is assumed to be a concentrated load, the Fourier transform of the train excitation is obtained by Fourier transformation [3].

$$|H_c(\omega)| = |H_a(\omega)| \times |H_b(\omega)| \times |H_{Lc}(\omega)| \tag{1}$$

$$|H_a(\omega)| = 2|\cos(a\omega/2v)|,\ |H_b(\omega)| = 2|\cos(b\omega/2v)|,\ |H_{Lc}(\omega)| = 2\left|\frac{\sin(n_c L_c\omega/2v)}{\sin(L_c\omega/2v)}\right| \tag{2}$$

Where, v is train speed in [m/s] while a, b and Lc are intervals [m] shown in Fig. 9. Fig, 9 (b) shows the calculated excitation spectrum of this train. The ETR1000 has dominant components not only at the frequency determined by the vehicle length but also at 7 and 8 times it. The 7th excitation peak frequency of the vehicle length can be calculated as 27.6 Hz at 380 km/h which is the resonance peak observed in Fig. 8 (b). This value agrees with the natural frequency of the fifth order deck local mode. This confirms that the high-order resonance between deck local modes and passing train contributes to cause the dominant peaks seen at the maximum value of 30HzLPF acceleration.

4. Conclusions

In this study, multi-points acceleration measurement and modal identification were conducted focusing not only on global modes but also on deck local modes of the Sesia viaduct up to 30 Hz. Through the experimental analysis, it was found that the 7th and 8th order resonances between high-order deck local modes and passing train excitation have dominant influence on the 30HzLPF acceleration. In addition, it was shown that, in the Sesia viaduct case, when the maxima values of the girder accelerations are adopted for the deck acceleration validation, as it is conventionally done, it leads to an underestimation of approximately 25% if compared to the ones of the deck.

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