Evaluation and Research on the Load-Bearing Performance of Railroad Steel Plate Girder Bridges Under Long-Term Service

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Abstract. To objectively evaluate the dynamic and static load performance of a railroad bridge that has been in service for nearly seventy years, a steel plate girder bridge along a railroad line in China is used as a case study to conduct on-site static load tests and dynamic load tests under 5 km/h, 10 km/h, 15 km/h, 20 km/h, and 25 km/h working conditions. Compare the finite element model and theoretical calculation values. Then, analyze the flange plate stress, mid-span deflection, transverse and vertical maximum acceleration, and transverse and vertical maximum amplitude of the steel girder bridge at each measurement point. The results show that: 1) the finite element calculation model can better analyze the actual mechanical behavior of the bridge structure; due to the long-term service of the bridge structure, the performance of the steel structure decreases, resulting in an increase in the stress of the upper edge of the left and right-wing plates, and the ratio of the measured upper flange strains of the right and the left rafters of the span 1 is 1.217. The mid-span deflection calibration coefficient of the bridge is 0.76. The bridge structure is in good working condition, and the load-carrying performance is close to the design value under the static load. 2) the bridge structure is in good working condition, and the load-carrying performance is close to the design value. The difference in transverse acceleration at each control point is small. The performance of the span is relatively similar. The structural integrity of the superstructure is better. The amplitude of the vertical amplitude of the girder in the span under the maximum condition of 25 km/h is directly proportional to the speed of the traveling vehicle. In the experimental condition, the transverse amplitude of girders and pier tops is the largest in the 25 km/h condition, and there is no abnormality in the structural integrity of the bridge superstructure in the test range of span 1 and 2. The vertical amplitude difference between two adjacent span in span 4 is larger than the rest of the span. 3) The impact coefficient values of the measurement points in holes 1 and 2 are close to each other, and both reach the maximum under the working condition of 15 km/h. The results of this study can provide a reference for the maintenance and performance evaluation of bridges in long-term service.

Keywords: Long-term service; Steel plate girder bridge; Dynamic and static load testing; Dynamic performance; Finite element analysis

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1 Introduction

The railroad bridge is an important part of railroad transportation, and its dynamic and static bearing capacity is crucial to railroad transportation. In the 1950s and 1960s, China built a large number of railroad bridges and culverts in order to accelerate the construction of railroads. Among the many small and medium-span railroad bridges and culverts designed after liberation, steel bridges accounted for a high proportion. Due to the service environment, load conditions, structural shape, and other comprehensive factors of the coupling effect [1], steel structure bridges are prone to a variety of diseases. As a result, many scholars have conducted studies. Xiao Xin et al. [2] study the mechanical behavior of mega-span steel truss girder suspension bridges under static loading. Qin S. Q. et al. [3] Combining finite element modeling for dynamic characterization of bridges. Li Qiang [4] conducted foot-scale modeling of fully prestressed concrete channel beams under asymmetric vertical loads. Tian Zhiyong [5] and others conducted comprehensive static and dynamic load tests on a three-span self-anchored suspension bridge with steel-concrete composite girders. VILLALBA-HERRERO [6] evaluated the bending behavior in static load tests. HARASID et al. [16] definitively demonstrated the effectiveness of their approach by performing a finite element analysis of static load tests on Rusunawa Engineering in Jakarta. Xin Xuezhong et al. [7] definitively outlined the differences between the standards before and after the revision of train live loads, as well as the references to the current design standards. Hu Shouting et al. [8] [9] studied the load schema of railroad trains in China and investigated the current status of the application of power coefficients at home and abroad. The results clearly showed that China needs to improve the design of the power coefficient system further with the improvement of China's railroad design and construction level. Railroad live load has been completely converted from steam locomotives to electric locomotives and internal combustion locomotives. Most of the long-service railroad bridges face speed-up operations. Further in-depth research is required to gain a full understanding of its measured performance. It has important theoretical significance and practical application value.

An actual long-service steel plate girder bridge was used as the subject of this study. A series of static and dynamic load tests were conducted on railroad bridges that had been in service for a period of 66 years. A comprehensive assessment of the bridge's load-bearing and load-carrying capacity was undertaken.

2 Project Overview and Finite Element Modeling

2.1 Project Overview

The Fenghe Bridge is situated in Xi'an City, Shaanxi Province. It was constructed in 1956 and has a total length of 159.9 meters. It is a six-span, 24-meter top-supported steel plate continuous girder bridge with a beam width of one meter and a beam height. As an important transportation hub, the Fenghuang River Bridge carries a large, long-term high load. The bridge is designed for a medium-22 live load.

2.2 Finite Element Modeling

In this study, the finite element software Midas Civil was used to establish the finite element model of the Fenghe Bridge. The full bridge model is shown in Figure 1. To avoid conflict between old and new operational loads. According to the relevant codes [10-11], a low-grade old medium-18 load is used for load arrangement for simulation analysis. This full bridge model has a total of 1254 cells with 897 nodes. The findings indicate that the model is more effective in simulating the static and dynamic state changes of the Fenghe Bridge.

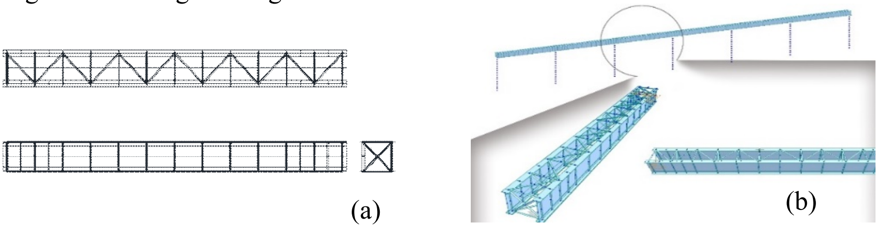

Fig. 1. Overview of top-loading steel plate girder bridges : (a) Three views of the steel bridge single-span structure; (b) Finite element model of the main bridge

3 Experiments and analysis of results

The test was conducted using trains of the current formation The combination of test conditions and test locations shall be in accordance with the specification [12]. The principle is that the bridge may be in the most unfavorable state of stress when in operation. Static load test using HXN5 single (25t axle weight) and HXN5 single A dynamic load test was conducted using a single HXN5 and four C70, with all four C70 fully loaded and four C70 unloaded. The load test points are shown in Figure 2.

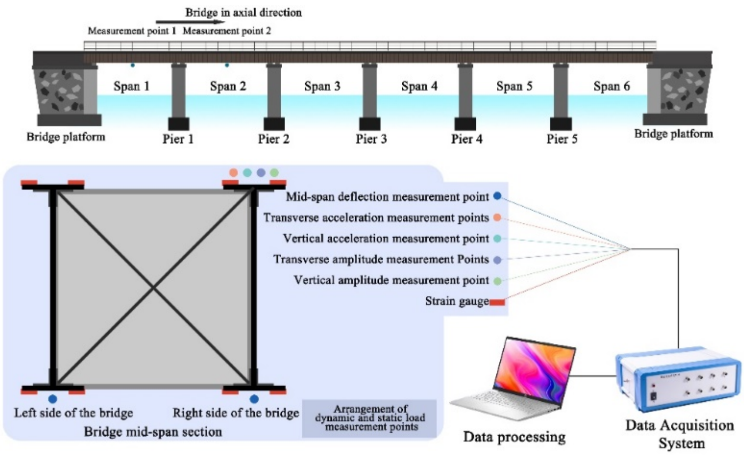

Fig. 2. Dynamic and static load test data acquisition system

The test utilizes DH-5922, DH-5983, and JM-3840 test instruments for simultaneous data acquisition. The self-oscillation frequency test was performed using the residual vibration method for data acquisition. A total of 4 control sections and 24 measurement points were set up. The static load test was divided into two working conditions for loading, and each working condition was loaded three times, with each loading duration of 3-5 min. The dynamic load test was selected for the mid-span inbound dynamic load test. Designed formation trains are loaded three times on the bridge at 5, 15, 20, and 25 km/h in uniform forward speed. Running in reverse at 5 km/h after each load is completed.

4 Static Load Test Results and Analysis

4.1 Bridge Upper Flange Stresses and Lower Flange Stresses

This steel plate girder bridge was subjected to a static load test under the action of a formation train. The stresses on the upper and lower flanges of the left and right plate girders of Fenghe Bridge are shown in Fig. 3, and the negative values are compressive stresses. For the convenience of recording the experimental results, the following data are abbreviated to a mixed form of numbers and letters.

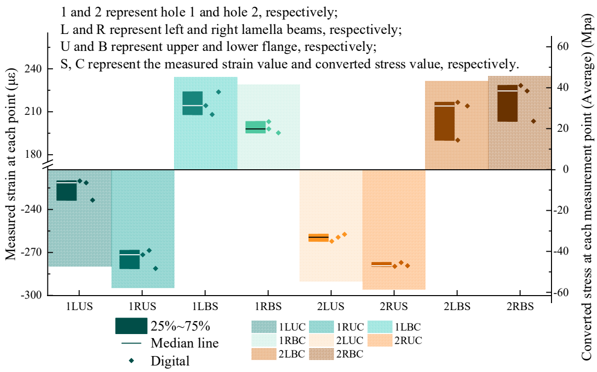

Fig. 3. 1st and 2nd aperture conversion strain values

The mean values of the three tests for 1LUS, 1RUS, 2RUS, and 2LUS were -225.0 με, -273.8 με, -259.7 με, and -278.7 με. The ratios of 1RUS to 1LUS and 2RUS to 2LUS were 1.217 and 1.07. The mean values of 1LUC and 1RUC were -47.250 MPa and -41.748 MP. The mean values of 2LUC and 2RUC were -54.537 MPa and -58.527 MP. The mean values of 2LBC and 2RBC were 43.449 MPa and 45.927 MPa. This steel plate girder bridge has a double layer of 10 mm thick composite steel plates in the span, and the stress measurement points are arranged in the girder span. The result is that the measured upper flange spanning strain is much smaller than the measured lower flange.

Midas Civil calculated the theoretical stresses to be 72.5 MPa for the upper flange and 78.2 MPa for the lower flange. The stress values at each measurement point are in the theoretical calculation range, indicating that the static load-carrying capacity of the bridge meets the requirements.

4.2 Mid-span deflection

When the static load of the two machines is applied to spans 1 and 2 of the bridge, the results are shown. The mid-span deflections of the bridge body in spans 1 and 2 of the steel bridge are 13.97 mm and 13.62 mm. It is slightly lower than the theoretical mid-span deflection value of 14.27 mm. This indicates that the structural stiffness of the girder is high and that the structure is in good working condition.

The difference between the measured mid-span deflections of spans 1 and 2 is small. This indicates that the bridge has similar structural stiffness at mid-span for both spans. After conversion, the maximum deflections were found to be 19.11 mm and 18.63 mm. This translates to vertical deflection ratios of 1/1256 and 1/1288, Both of which are less than the usual values of deflection ratios for top-bearing plate girders (⩽1/1200). Under static load, the theoretical allowable mid-span deflection of the bridge is 24 mm, and the theoretical allowable mid-span deflection check coefficient of the bridge is 0.76. The theoretical allowable mid-span deflection of the bridge under static load is 24 mm. The theoretical allowable mid-span deflection of the bridge is a check factor of 0.76. The maximum values of deflection check coefficients for spans 1 and 2 are 0.58 and 0.57. The check coefficients for the two spans are close. The static load performance is closer, and no large deflections are observed.

5 Dynamic load test results and analysis

5.1 Transverse and vertical acceleration

The maximum values of transverse acceleration in spans 1 and 2 are 1.045 m/s^2 and 1.013 m/s^2, respectively. The maximum values of transverse acceleration in Pier 1 and Pier 2 are 0.361 m/s^2 and 0.527 m/s^2. The vertical acceleration test results are shown in Fig.4. The vertical accelerations all reach their maximum values at 25 km/h, showing a tendency to increase with increasing speed.

The reason for the large difference in lateral acceleration at the top of Pier and Pier is that the height of Pier 1is lower than that of Pier . The structure is different, and thus, the lateral acceleration difference is larger. Both piers show lateral acceleration maxima at 20 km/h.

On the vertical vibration acceleration. The bridge abutment and Pier 1# are built on relatively high ground. The height of Pier 1 is lower than that of other piers. Therefore, the structural stiffness of span 1, span 2, and span 6 are smaller than the rest of the spans. Thus, it can then be observed from Fig. 4 that the vertical acceleration is higher in spans 1, 2, and 6. With the increase in velocity, the vertical acceleration of each span is gradually increased. And the difference in vertical acceleration between each span is also growing.

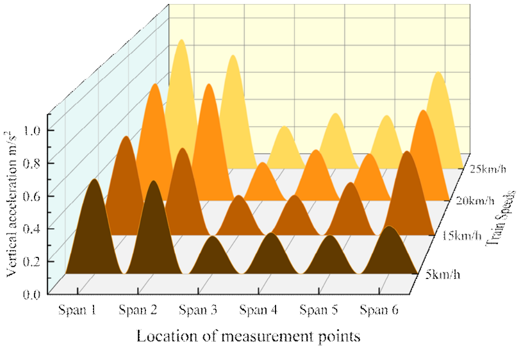

Fig. 4. Transverse and Vertical Acceleration Testing of Bridge Structures

5.2 Transverse and vertical amplitudes

The results of the tests, which were conducted to measure the mid-span transverse amplitudes of the girders at piers 1 and 2 and the transverse spokes at the top of the piers, are presented in Fig. 5. When the train traverses the steel bridge at a velocity of 25 km/h, the mid-span transverse amplitudes of the girders at piers 1 and 2 and the girders at span 1 and 2 reach their maximum values. These values are 0.105 mm, 0.143 mm, 1.784 mm, and 1.995 mm, respectively.

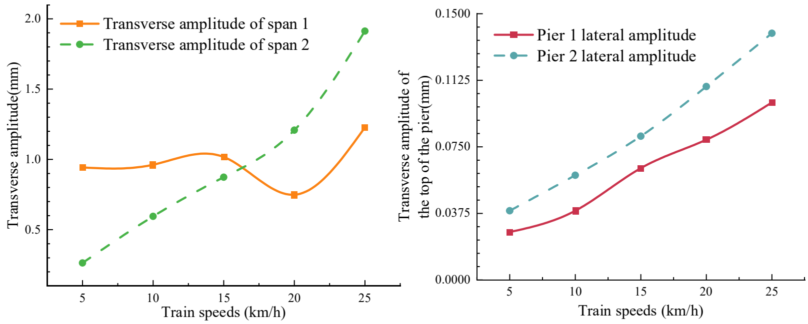

Fig. 5. Transverse Spoke Test Result

The transverse amplitude values in spans 1 and 2 do not exceed the usual value of 3.16 mm, nor do they exceed the safety limit value of 4.36 mm. However, the transverse amplitude at the top of the pier demonstrates an increasing trend with the increase of train speed. This indicates that the dynamic performance of piers 1 and 2 is superior, with pier 2 exhibiting slightly enhanced performance compared to pier 1. From the analysis of Fig. 6, the maximum values of the mid-span vertical amplitudes at the measurement points all occur at a speed of 25 km/h. It was observed that the vertical amplitude in the span of the girder increased in proportion to the increase in traveling speed.Vertical amplitudes of two adjacent spans are similar.

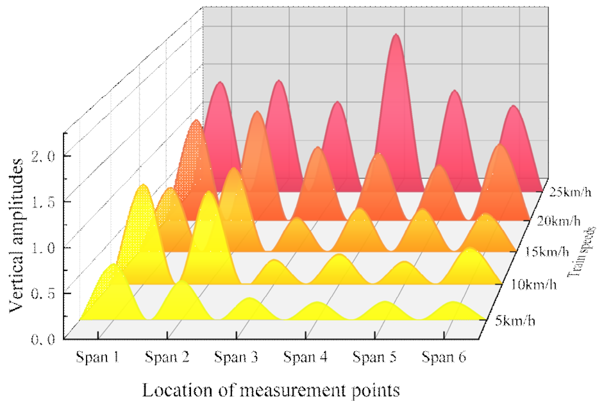

Fig. 6. Spanwise vertical amplitude of test span girders of Fenghe Bridge under the action of trains

Following the 5 km/h test condition, the vertical amplitudes of the initial and subsequent span measurement points are 0.624 mm and 0.432 mm, respectively, which exhibit a slight increase compared to the vertical amplitudes of the remaining span measurement points. At a velocity of 25 km/h, the vertical amplitude of the fourth span was 2.035 mm, which is a value that exceeds the remaining test results for this specific point. The transverse amplitude test data for the first span at speeds of 5, 10, and 15 km/h are greater than those for the second. The structural and support conditions of the abutments and piers can have an effect on the vibration response of the bridge. Loose bridge bolted supports in the first span can also cause the girders to generate greater amplitudes during train passage.

6 Conclusion

1)Due to the influence of the steel bridge structure, the measured upper flange strains of the bridge are smaller than the lower flange strains at the corresponding measurement points. The ratios of the measured upper flange strains of the right girder to the measured upper flange strains of the left girder for spans 1 and 2 are 1.217 and 1.073, respectively, and the structural damages brought by the long-term service of the bridges may cause the measured strains to decrease. The mid-span deflections of the bridges at spans 1 and 2 are 13.97 mm and 13.62 mm, respectively, and the check coefficient of the mid-span deflection of the bridges is 0.76.

2) The maximum values of transverse acceleration in spans 1 and 2 are 1.045 m/s^2 and 1.013 m/s^2, respectively. The difference in transverse acceleration is small. The structural integrity of the beam is good.

3) Under the test condition, the vertical amplitude in the span of the girder is proportional to the traveling speed. For all spans except span 4, the difference in vertical amplitude between two adjacent spans is small. Under the test condition, the transverse amplitudes of the girders and pier tops reach the maximum value at 25 km/h. The transverse amplitudes of the girders and pier tops reach the maximum value at 25

km/h. The transverse amplitude of span one increases with increasing speed from 5 km/h to 15 km/h. It decreases and reaches a minimum value of 0.749 mm at 20 km/h. The transverse amplitude of span two increases with speed. The dynamic performance of the bridge is good.

4) The overall structural performance of the bridge is good. For a bridge that is 66 years old, the steel plate girder structure shows little deterioration in performance during operation and maintenance.

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