Research Article

Simulation Analysis of Coupled Structural Vibration of Highway Railway Combined Bridge Induced by Overlapping Action of Vehicle and Train

Peng Wan(1), Xiaoyan Lei, Weibin Wu, Qiongqing Hu, Ling Wan, and Guilin Li

1Engineering Research Center of Railway Environmental Vibration and Noise, Ministry of Education, Nanchang 330013, China
2Jiangxi Transportation Research Institute Co., Ltd, Nanchang 330099, China

Correspondence should be addressed to Peng Wan; wanpeng@j.t.jiangxi.gov.cn

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In order to study the dynamic response of bridge coupling caused by the overlapping of vehicle and railway dynamic loads, by using the time-domain samples of pavement irregularity compiled by MATLAB software and the secondary development function of ANSYS software, the vehicle, bridge, and train models are established in ANSYS software at the same time, and the train supported beam bridge coupling program is compiled with MATLAB software. The model considers the influence of initial conditions and vehicle speed on the system before entering the bridge, the interaction between the coupled large-scale highway and railway bridge system, and the influence of different adverse factors on the system. The influence of a vehicle or train on the bridge structure when the vehicle or train passes through the continuous steel truss bridge under different factors is calculated, respectively, and the displacement and acceleration limits of the bridge are evaluated in combination with the corresponding specifications.

1. Introduction

A highway railway dual-purpose bridge is the most economical bridge type in the world. The increase of bridge structure vibration will aggravate the fatigue of bridge components and even cause damage to the bridge structure. The aggravation of train vibration will reduce the comfort of passengers and even cause the derailment of the train. The intensification of vehicle vibration will have a significant impact on driving safety [1–3]. It can be seen that it is essential to study the impact of various factors on the mutual coupling system of the highway railway bridge, vehicle, and train. In order to truly reflect the coupling between them, a complete vehicle bridge train coupling vibration model must be established. However, some scholars at home and abroad have done much research on the coupled vibration of vehicle bridge structure and train line bridge structure, and there is little literature on the coupled vibration of vehicle bridge train systems.

At present, scholars at home and abroad have researched the long-span sea-crossing highway and railway cable-stayed bridge and the bridge response under seismic excitation [4–6]. The analysis of the coupling vibration characteristics of truss bridges and the sensitive impact of various factors under the overlapping action of vehicle and railway dynamic loads needs to be further refined. In order to ensure the safety of bridge structure and vehicle operation, it is necessary to study the vehicle bridge coupling vibration of highway and railway bridges under the action of vehicle train overlap. Reference [7] established a vehicle model with 7 degrees of freedom. The vehicle model only considers vertical vibration, and the wheelset moment is closely attached to the bridge deck; a light rail vehicle model with 15 degrees of freedom is established, and the wheelset moment of the light rail is closely related to the track beam. It is also in contact at all times; the motion equations of bridges, cars, and trains are coupled into a whole motion equation, and the direct integration method is used to solve the motion equation, and the program is written in Fortran95 language. Reference [8] established a train space model with 23 degrees of freedom. Because the calculation model is relatively complex and has many degrees of freedom, the vehicle model was not established, but the vehicle load was added to the bridge.
as a point load. The degrees of freedom of the train and car models established in the literature [9–20] are 15 and 7, respectively. The Monte Carlo method is used for the simulation of track irregularity and road surface irregularity, considering the impact of road roughness on the vibration of the train-bridge-vehicle coupling system. For impact, the results show that the influence of the secondary dead load on the acceleration of the bridge structure is more significant than that of the displacement. This also tells us that all loads need to be taken into account when performing the coupled vibration analysis of the vehicle axle. The vehicle model [21] considers the contact point sideslip model and the non-sideslip model, with $2 \times n + n + 5$ degrees of freedom, where $n$ is the number of wheel pairs; the wheel-rail contact of the train considers the vertical direction for force and creep force, a single-section vehicle is used as a space model of 35 degrees of freedom, and the program is written in Fortran6.5. In the literature [22], the influence of the braking force on the bridge when the train is braking is considered, and the bridge structure modeling adopts SAP2000 software, and the influence of the car is not considered in the study. Studies have shown that the braking force will cause the structure to generate additional bending moments, etc., and its influence needs to be considered in the structural design [23–25]. It can be seen from the above literature that there are many researches on vehicle-bridge coupled vibration or train-bridge coupled vibration, but relatively few studies on vehicle-bridge-train system coupled vibration, and the calculation model used is relatively simple.

For the bridge structure, due to the large number of degrees of freedom of the highway-railway bridge, some literatures use the mode shape superposition method to improve the calculation efficiency and make it possible to solve the coupled vibration of the vehicle and the bridge. This method cannot consider the influence of nonlinearity. For the vehicle structure, there are relatively few literatures on the simultaneous establishment of car and train models. Some literatures simplify the car or train as a moving constant force model, so it is impossible to consider the mutual influence between them and analyze the driving comfort. Most of the literatures use multiple software to simulate the process together, so there will be different degrees of errors in the mutual calls, and the analysis steps are more complicated. In order to make up for the above shortcomings, this paper will use the secondary development function of ANSYS to establish models of cars, bridges, and trains in ANSYS software at the same time and couple the three separated subsystems through constraint equations, so as to consider the mutual interaction between them, impact, and other factors on the system.

2. Train Vehicle Bridge Theoretical Model

2.1. Basic Assumptions and Derivation Methods. The vehicle model is established under the following assumptions:

(1) All vehicle bodies, frames, and wheelsets are assumed to be rigid bodies.

(2) The rigid bodies are connected by springs and dampers.

(3) The wheelset and the bridge deck are closely attached without separation.

(4) The establishment of a vehicle model can be regarded as the category of multirigid body dynamics. The derivation of the motion equation mainly includes the energy method and the D’Alembert principle. The derivation of this paper makes use of the latter.

2.2. Train Model and Vehicle Model. This paper adopts the vertical train model with 10 degrees of freedom, and its calculation diagram is shown in Figure 1.

The derivation of the vehicle operation equation is the same as that of the train motion equation, which will not be repeated here. The car model used in this paper includes a car body and two wheelsets. The vehicle body has two degrees of freedom, such as vertical displacement and rotation, and the wheelset has only vertical displacement degrees of freedom.

Through the above analysis, the motion equation of vehicle or train can be obtained, as shown in

$$[M_v] \{ \ddot{Z}_v \} + [C_v] \{ \dot{Z}_v \} + [K_v] \{ Z_v \} = \{ F_v \}. \quad (1)$$

2.3. Simulation of Irregularity

2.3.1. Road Surface Irregularity. According to the recommendations of vehicle vibration input—pavement flatness representation method, the displacement power spectral density is shown in equation (2), and the pavement flatness coefficients of different pavement grades are shown in Table 1.

$$S_x(\Omega) = \frac{k A_s \Omega^2}{\Omega^2 + (\Omega^2)$

$$S_y(\Omega) = \frac{k A_s \Omega^2}{\Omega^2 + (\Omega^2)$

$$S_z(\Omega) = S_y(\Omega) = \frac{4k A_s \Omega^2}{(\Omega^2 + (\Omega^2) + (\Omega^2)^2).} \quad (2)$$

2.3.2. Long-Wave Track Irregularity. The American arr track spectrum is adopted, and the irregularity expression is shown in

$$S_x(\Omega) = \frac{1}{(\Omega^2 + \Omega_c^2)}.$$
where $S(\Omega)$ denotes the power spectral density; $\Omega$ denotes the spatial frequency, rad/m; and $A$ denotes the roughness constant, whose value is between $4.15 \times 10^{-8}$ m rad and $5.0 \times 10^{-7}$ m rad. In this paper, $A$ is taken as $3.15 \times 10^{-7}$ m rad. The expression of the America class 6 track irregularity spectrum is omitted to reduce the length of the paper.
The method is used to generate sample waves in a time series according to the expression of the power spectral density of the track irregularity via superposition of a simple random process with various statistical properties. The generated sample waves in time series according to the Sato track irregularity spectrum (Equation (3)) and American class 6 track irregularity spectrum are shown in Figures 2(a) and 2(b), respectively. The combined sample, as shown in Figure 2(c), is generated by adding each amplitude of two samples whose coordinates are the same.

2.3.4. Round of Analysis of Wheel Irregularity. The abnormal development of wheelset irregularity of metro vehicles is common in domestic high-speed railway operations, and some secondary problems caused by vibration also occur frequently. In order to improve the wheelset out of roundness, the wheel-rail relationship and wheelset circular runout are analyzed. Grinding treatment is not carried out before operation. The maximum running speed of the vehicle is 350 km/h, and the section running speed is 300-350 km/h. The model of wheelset rim involved is CL60, and the model of the main line rail is U65. Vehicle wheelsets mainly include the wheel out of roundness test, vehicle tread wear, and hardness test. According to the rail condition, it mainly includes the rail wave wear test, rail welded joint irregularity test, rail profile, and hardness test. For the wheel-rail relationship, it mainly includes vibration test, vibration acceleration frequency domain and time domain characteristic analysis, and vibration acceleration statistics. Based on various test and analysis results, as well as the analysis of traffic organization characteristics, vehicle operation, and line layout characteristics, this paper explains the regular characteristics of wheelset nonroundness and relevant key incentives (such as wheel-rail vibration characteristics, welded joint irregularity, and vehicle operation characteristics). At the same time, practical improvement measures are put forward through verification.

In order to ensure that the wheel can rotate freely around the axis during the test, jack is needed to lift the axle boxes on both sides of the coaxial and fix the BST (or BBM) out of roundness tester above the rail. The displacement sensor is in vertical contact with the wheel and is located at the nominal rolling circle on the wheel tread to record the noncircularization information at the nominal rolling circle of the wheel. The irregularity space irregularity diagram and polar coordinate diagram can reflect the wheel irregularity state. The abscissa of the irregularity space irregularity map reflects the circumferential position information (circumference) of the displacement sensor from the test starting point, which is recorded by the rotation sensor. The ordinate represents the wheel radial jump fluctuation value (corresponding to out of roundness), that is, the difference between the maximum radius and the minimum radius of the wheel. The polar chart converts the abscissa into the corresponding angle, which can more vividly reflect the out of roundness state.

In order to further reflect the polygon state characteristics of wheels, the Fourier formula can be used to expand the contribution proportion of polygons of different types of vehicles to obtain the polygon order diagram of noncircular wheels, from left to right. The wheelset out of roundness is caused by the high proportion of first-order (eccentric), 6~10 side shape, and 14~18 side shape, respectively. The inducing factors of wheelset nonroundness with different polygon characteristics have strong regularity, which is the main entry point to analyze the inducement of wheelset nonroundness. In this paper, the out of roundness of 172 wheelsets of 4 trains is randomly sampled. Through statistical analysis of the test results, the following rules are found.

(1) 66% of the wheels show the noncircular characteristics of 5~8 sides, of which most of them are 6~7-sided

(2) The out of roundness of the wheels is poor. The maximum circular runout of nearly 1/4 of the wheelsets exceeds 0.6 mm, about 1/7 of the wheelsets exceeds 0.8 mm, and the maximum measured radial runout reaches 1.6 mm (generally considered). When it is 0.8 mm, it needs to be lathed, and the development rate of wheelset radial jump is 0.03~0.04 mm/10000 km. Figure 3 shows the average value of the maximum measured radial jump of the selected 4 train wheelsets.

2.4. Coupling of Vehicle Bridge Motion Equation and Solution of Equation. In the previous derivation, the vehicle and the bridge are separated. That is, the coupling between them is not taken into account. In order to realize the interaction between vehicles and bridges when vehicles cross the bridge, their motion equations must be coupled.

There are two main methods of vehicle bridge motion equation coupling—integral method and the separation method. The whole method couples the motion equation of vehicle and the motion equation of bridge structure into a whole motion equation through the balance condition of force and deformation coordination condition and then solves the whole motion equation. The separation method lists the motion equations of vehicle and bridge, respectively, judges the convergence of this step through the relative error between two iterations of bridge structure displacement, and then solves it in turn. When there are many degrees of freedom, the separation method can reduce the complexity of the degree of freedom coupling in the overall method. In addition, the interaction force between wheel and rail can be fully exposed, which lays a foundation for the study of the derailment coefficient. This paper uses Newmark collected in most literature-β law. The example in Section 3 shows that when the relative error control is very small, it only needs 2 to 4 iterations. It can be seen that the solution efficiency of this method is also very high [27].

3. Vibration of Bridge under the Action of Train and Automobile

3.1. Project Overview. This paper takes the Jiujiang Yangtze River Highway and Railway bridge as an example. The bridge has a total length of 780 m and adopts
(180 + 226 + 180) five-hole continuous steel truss beam, with a truss height of 16 m, truss width of 11 m, and internode length of 10 m.

The upper layer is a two-way four-lane highway with a design speed of 60 km/h, and the lower layer is a railway with a design speed of 120 km/h. This paper uses ANSYS to establish the bridge model, and the beam188 spatial beam element is used for steel truss modeling. This element has a directional node, which can quickly locate each section. Combine14 element is used for support simulation, considering the spring stiffness in each direction, and caring command is used for eccentricity treatment at the connection of different elements, considering the impact of actual eccentricity on structural analysis.

3.2. Unidirectional and Bidirectional Action Conditions.

When the train runs on the bridge structure, there may be only a single train running or multiple trains running in.
the same direction or opposite [28]. In order to qualitatively understand the impact of different conditions on the bridge structure, three working conditions are set in the subsequent analysis. In order to observe the difference in the impact of these three working conditions on the structure, three working conditions are set, including working condition 4 to working condition 6.

Condition 1. Single train running in one direction.
Condition 2. The two trains run in the same direction.
Condition 3. Two trains run in reverse.
Condition 4. (Condition 2-condition 3)/condition 2.
Condition 5. Condition 2/condition 1.
Condition 6. (Condition 3-condition 1)/condition.

Figure 5(a) shows the displacement time history curve of No. 3 node in condition 1~condition 3; Figure 5(b) shows the acceleration time history curves of different midspan bridge structures under the same working condition; Figure 5(c) shows the acceleration time history curve of bridge structure at different nodes of the same span under condition 1 (i.e., eccentric load).

3.3 Working Conditions under Different Train Loads. The influence of different loads on the dynamic response of bridge structure is mainly considered in this paper. The loads are 0 tons, 8 tons, 16 tons, 10 tons, and 40 tons.

Figure 6 shows the variation trend of peak acceleration and peak displacement of No. 1, No. 7, No. 3, and No. 5 nodes of the bridge structure under different loads. Figure 7, respectively, shows the acceleration time history response curves of the No. 1 node and No. 7 node of the bridge structure when the load is 5 tons.

It can be seen from Figure 6 that the influence of vehicle load on the displacement and acceleration peak of the bridge structure is almost linear. Among them, the slope of displacement peak under different loads is more significant. It can be seen that the influence of load on the displacement of the bridge structure is more significant than that of acceleration. Nodes 3 and 5 are located in different spans in the same plane, and their spans are the same. It can be seen from Figure 6 that their peak acceleration and peak displacement
Condition 1
Condition 2
Condition 3

(a) Displacement time history curve of No. 3

(b) Node 5-point acceleration time history curve under different working conditions

Figure 5: Continued.
coincide with the increase of load and are larger than the peak displacement and acceleration of side span (the span of side span is 120 m, and the span of other spans is 180 m). It can be seen that the influence of span on the dynamic response of the bridge structure is enormous. As shown in Figure 6, the acceleration response of railway pavement at the side span is smaller than that of highway pavement under different loads. As shown in Figure 7, this difference mainly occurs at a specific instant, and the response at other times is not different.

3.4. Vehicle Line Condition. With the increase in passenger volume, the number of train formations will also increase in order to understand in more detail the impact of the change of train formation number on the safety of bridge structure, driving comfort, and safety and prepare a corresponding degree to study its impact.

In this working condition, the influence of train formation on the dynamic response of the bridge structure is specially studied. This time, the dynamic response of the bridge structure is mainly considered when the train formation is
one group, two groups, three groups, four groups, five groups, and six groups.

3.5. Single- and Two-Way Driving Conditions. The vehicle is random when driving on the bridge structure. This section analyzes five typical adverse working conditions to find some laws, in which the vehicle running speed is 20 m/s, and the number of vehicle groups in each lane is 4. Table 3 show the peak values of displacement, velocity, and acceleration of nodes 1, 3, and 5 on the railway deck and nodes 7, 9, and 11 on the highway pavement under different working conditions and list the maximum values of each node peak value.

Condition 1. Lane 1.
Condition 2. Lane 1 + Lane 2.
Condition 3. Lane 1 + Lane 2 + Lane 3.
Condition 4. Lane 1 ~ Lane 4.
Condition 5. Lane 1 + Lane 4.

3.6. Different Groups. As a basic condition, the train and vehicle act on the bridge structure at the same time, the vehicle speed and train speed are maintained at 60 km/h, and the track irregularity and road irregularity are not imposed. The impact on the bridge structure and vehicles when the train formation is 1 group, 3 groups, and 5 groups shall be considered, respectively. Table 4 shows the peak displacement of each node of the bridge structure under different train formations; Table 3 shows the comfort levels of trains in different lines; Figure 8 is the displacement time history curve of No. 1 node under different lines and the time history curve of wheel-rail interaction force of the first wheelset of the train body; Figure 9 is the time history curve of angular displacement of train body and the corresponding spectrum diagram; Figure 10 is the time history curve of vertical acceleration of vehicle body and its corresponding spectrum diagram.

It can be seen from Table 4 and Figure 8 that with the increase in train formation, the displacement peak value of midspan nodes of each span of the bridge structure is increasing. When the number of formations reaches a certain degree, the change amplitude is decreasing or even no longer changes. However, the increase in the number of train formations will increase the time when the bridge midspan node is at a large value. It can be seen from Figures 9–11 that the number of train formations has a relatively small impact on trains and vehicles, mainly because different train formations are independent of each other and the mutual coupling between train bodies is not considered. By comparing with the moving constant force model, it can be found that the difference between the displacement peak of the bridge structure obtained by the moving constant force model and the displacement peak obtained by the programming idea is very small, but the difference in acceleration is still relatively large. The acceleration response of the bridge structure under the simultaneous action of train and vehicle and the acceleration response of bridge structure under the separate action of train or vehicle are not simply superimposed, and there is no certain regularity. In order to analyze the impact of vehicle and train on the acceleration of the bridge structure, a perfect vehicle bridge train coupling vibration model must be established. Without track irregularity, the dynamic response peaks of vehicles, trains, and bridge structures are small, and the interaction
Figure 8: Displacement time history curve of first node.

Figure 9: Force time history curve of first wheel of train.
Figure 10: Continued.

(a) Time history curve
between vehicles and trains is also very small. The node displacement and impact coefficient in a span of the continuous steel truss bridge are mainly related to the dynamic load inside the span, but almost independent of the dynamic load of other spans. The displacement of the bridge structure is related to the span and vehicle formation, has little to do with track irregularity, and has no clear relationship with train speed. In order to obtain the peak displacement of the bridge structure under different vehicle speeds, we must take the envelope of the peak displacement under the research vehicle speed or an envelope of the impact coefficient. The increase in track irregularity and train speed will aggravate the peak acceleration of the train body, which is unfavorable to the ride comfort. In the vehicle bridge coupling program simulated by this method, the force between train wheel and rail and the interaction between vehicle and pavement fluctuate up and down around the constant force of wheel pair on the bridge structure under the action of self-weight, which is more in line with the actual situation than the moving constant force model.

4. Conclusion

(1) The mass and stiffness of each part of the bridge structure greatly influence its dynamic characteristics. There is not a simple superposition relationship between the working conditions under which the vehicle and train act on the bridge structure simultaneously and the working conditions under the action of the vehicle only or the train alone. The peak dynamic response of the bridge structure or vehicle does not necessarily appear under which working condition. In order to analyze this kind of structure more accurately, it is necessary to establish models under a variety of different working conditions and then take envelope design. Under the abnormal load condition of a train or vehicle, even if only the vertical load is applied, it will cause certain lateral displacement, which is unfavorable to the operation of the vehicle or train. The peak values of displacement and acceleration do not necessarily all appear in the same working condition. In order to find the most unfavorable load condition, we must study the adverse effects of various working conditions.

(2) In order to obtain a more realistic acceleration response of bridge structure and evaluate vehicle driving comfort, a more perfect vehicle bridge coupling vibration model must be established. Compared with the influence of vehicle spacing and vehicle formation on the peak value of vertical
Figure 11: Continued.

(a) Time history curve
deflection of the bridge structure, the influence of vehicle load on vertical deflection is more prominent. For different bridge structures, the weight of vehicles must be limited in combination with the actual situation.

5. Discussion

The analysis shows that the stiffness between the wheel and the pavement or track significantly impacts the evaluation of the dynamic response of the bridge structure. In order to truly reflect the dynamic response of the bridge structure, the stiffness must be set according to the actual situation. The deflection of the bridge structure is mainly related to the span of the bridge structure, vehicle load, vehicle formation, etc., and the track irregularity and vehicle speed have relatively little impact on it. At the same time, through sensitivity analysis, it is found that the increase in track irregularity will increase the interaction between trains and vehicles. In order to evaluate the driving comfort of cars or trains, we must consider the impact of cars or trains on each other.

The transverse stiffness of long-span continuous steel truss bridges is often weak, and the corresponding transverse displacement will occur even under the action of vertical load. Particular attention should be paid to the strengthening of transverse stiffness in design. When the train meets, the working condition of reverse driving is more unfavorable to the bridge structure than that of the same direction. The increase of train formation will increase the peak value of displacement and acceleration of the bridge structure. Under the action of the train, the displacement peak at the vehicle pavement is almost the same as that of the railway pavement in the same span. It can be seen that the impact of the train on the vehicle is also relatively significant. The increase in train formation will lead to an increase in the time when the acceleration at the same node is at its peak, which will aggravate the fatigue failure of steel structures. The influence of higher-order modes on acceleration is more significant than that of lower-order modes.
Data Availability
The data used to support the findings of this study are included within the article.

Conflicts of Interest
The authors declare that they have no conflicts of interest.

References