

NUMERICAL ANALYSIS OF THE INFLUENCE OF PRESTRESSED STEEL WIRES ON VEHICLE-BRIDGE COUPLING VIBRATION OF SIMPLY SUPPORTED BEAMS ON HIGH-SPEED RAILWAY

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Abstract. By the theory of vehicle-bridge coupled vibration analysis in railways, the dynamic analysis model for space of the train-track-bridge-steel wires coupled system was established. Moreover, a corresponding program was compiled based on the train-track-bridge-steel wires coupling vibration analysis method. Taking a 32 m simple beam which is in high-speed railways as the subject of study, the influence of effective prestress, steel wires eccentricity and vehicle speed on the dynamic response of the vehicle-bridge coupled vibration was analysed. The results show that the bridge dynamic response is remarkably influenced by prestressed steel wires. With the prestress increasing, the crest of the vertical dynamic response at the midspan decreased first, then increased. Moreover, the minimum peak value appeared when the prestress was 1300 MPa.

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When the steel wires were deflected downward relative to the design position, the vertical displacement of the bridge decreased by more than when the downshift occurred. The extreme values of the bridge lateral dynamic response and the train body acceleration response appeared when the train ran at 300 km/h. Prestressed steel wires had little effect on the dynamic response in the transverse direction of the bridge and train body.

Keywords: simple beam in high-speed railway, vehicle-bridge coupling vibration, prestressed steel wires, effective prestress, eccentricity of steel wires, self-compiled program.

Introduction

Prestressed concrete (PC) box girder is used in high-speed railways bridges. As a significant component of the bridge structure, the effective prestress of prestressed steel wires, which is a critical component affecting the loading capacity of bridges, has an effect on the dynamic performance of bridges. At present, many investigations about the influence of prestressed steel wires on the bridge dynamic performance have been carried out by many scholars. The formulas for calculating the foundation frequency of prestressed concrete bridges were determined by Saïidi et al. (1994). However, Dall'Asta & Dezi (1996) thought that the formulas were not suitable and put forward their own opinion. The relationship between the magnitude of prestressing and the natural frequency of PC beams was analysed (Zhao et al., 2019; Bonopera et al., 2019; Toyota et al., 2017). Xu et al. (2019) studied the relationship between simply supported beams and different eccentric distances and tensile force values by establishing a solid model of prestressed simply supported beams through ABAQUS. Dong et al. (2018) proved theoretically that vertical fundamental frequencies of simple beams could be raised by applying prestress. Li et al. (2018) studied the influence of different eccentric distances and prestresses on the fundamental frequency of the simple beam. Xiang et al. (2017) proposed a static reduction stiffness method and a dynamic stiffness correction coefficient method to calculate the fundamental frequency of simple beams with external prestressing tendon. Zhong et al. (2020) studied the influence regularity of external prestressing parameters on the natural frequency of steel-concrete composite simple beam through numerical simulation. Ren et al. (2018) analysed the deviation influence factors of the first-order frequency between the data measured in the dynamic load test and the theoretical calculation result of the prestressed concrete continuous beam. Xiao (2018) analysed the natural vibration of the typical bridge of heavy-duty railways. Guo et al. (2018) studied the fundamental frequency of simple beam with unequal length

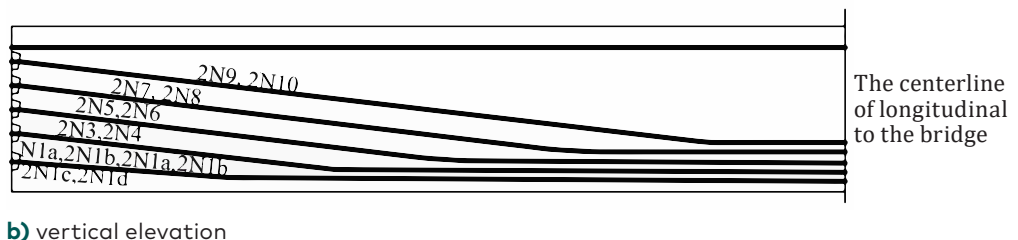
prestressed tendons through theoretical research, finite element method and experiment. Chen (2012) studied the coupling vibration of prestressed simply supported beams under vehicle load and solved the coupled vibration differential equations of T-shaped prestressed simply supported beams by the RKF method. Yu (2012) found that the change of structure frequency due to the increase of prestressing was uncertain and the reinforcement ratio was the main factor. The control equation of the fundamental frequency is amended by the differential orthogonal method (DQ) and some conclusions about the relationship between the fundamental frequency of prestressed concrete beams and their physical parameters are obtained by numerical methods (Peng & Wang, 2021). Fang (2020) found that the natural frequency of PC continuous beams was related to the number of diverters but had little to do with the arrangement of external prestressed steel wires by using the energy method considering the second-order deformation of external prestressed steel wires. Considering the influence of parameters such as prestress, Ghaemdoust et al. (2021) summarised a set of methods for predicting the fundamental frequency of PC beams, comparing the theoretical values with the results of finite element models. The relationship between frequency and the prestressed load is obtained through impact load test on nine PC beams prestressed by the post-tensioning method (Noble et al., 2014).

There are numerous studies about the influence of prestressed steel wires on the natural vibration characteristics of bridges. The solid modelling method is usually used to discretize the bridge structure with prestressed steel wires. The vehicle-bridge coupled vibration analysis is very complicated and calculation efficiency is low due to the increase of the DOF of the finite element model of bridges with prestressed steel wires. Therefore, there is little analysis about the influence of the action of prestressed steel wires on the vehicle-bridge coupled vibration response in highway and railway bridges. Moreover, there are a few studies on the influence of the prestressed steel wires on the coupling vibration of the bridge in high-speed railways. The present paper shows whether and how the prestressed steel wires affect the vehicle-bridge coupled vibration. It is meaningful because there are many studies based on vehicle-bridge coupling vibration. The aim of our research is to make the theory of vehicle-bridge coupling vibration more perfect.

In the paper, the research subject is simple beam bridges in the high-speed railway. The influence of the prestressing tendon on the coupled vibration response of simple beam bridges in the high-speed railway is analysed according to the analysis method of railway vehicle-bridge coupled vibration. Moreover, the paper discusses the effect of the

1. Bridge finite element model with prestressed steel wires

Based on *prestressed concrete simply supported box girder bridge (double track) with ballastless tracks on high-speed railways at a speed of 350 kilometres per hour (2013) 2322A-II-1*, a single box bridge with single-cell was selected as the subject of the study. The length of the single-span box girder was 32.60 m and the computing span between supports of the bridge was 31.50 m. The bridge decking width and height of the beam were 12.6 m and 3.035 m. The strength grade of beam concrete was C50 and the parameter specification of prestressed steel wires could be obtained from the document mentioned above. There were 27 steel wires in the bridge. The standard value of tensile strength of the steel wires (f_{pk}) was 1860 MPa, and the elastic modulus was $1.95 \cdot 10^5$ MPa. The layout of prestressed steel wires of box girder is shown in Figure 1.



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The three-dimensional finite element model of the bridge was established by the general FEA software ANSYS. The girder was simulated by the Beam44 element, and the prestressed rigid beam was discrete by the Link8 element. Prestress was applied to the bridge by means of initial strain. The main girder and the steel wires were divided into one element every 0.6 m along the bridge, and the prestressed steel wires and the girder were connected by the rigid domain (CERIG) at the node in order to facilitate the coupling connection between the girder

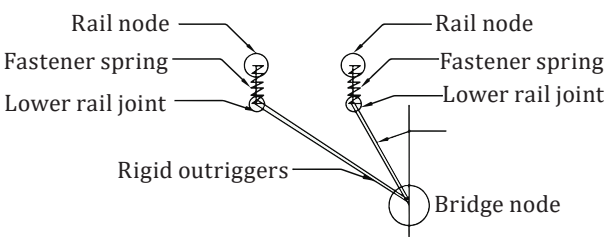


Figure 2. Layout of bridge track connection

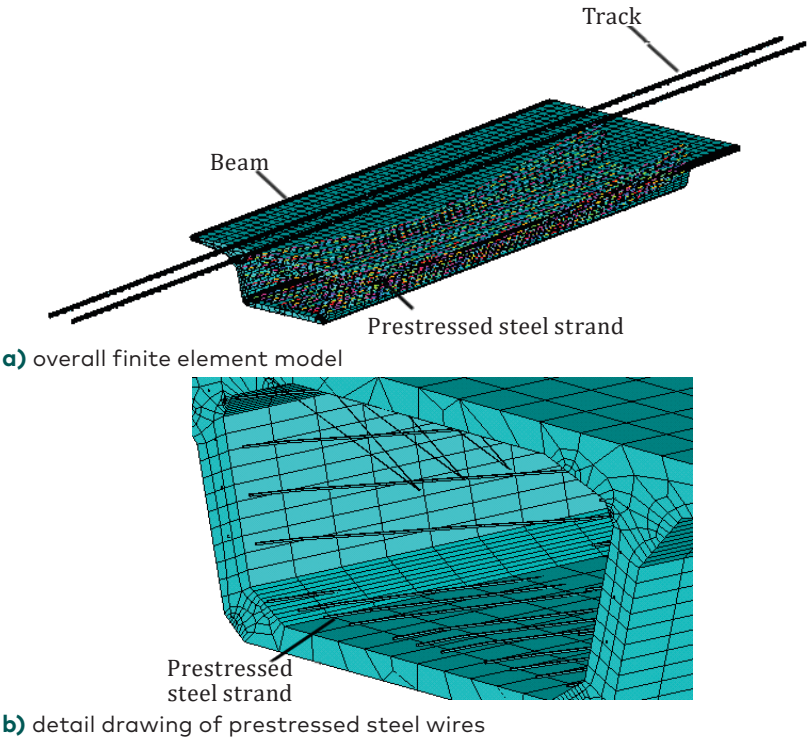


Figure 3. Finite element model of bridge-track system

nodes and the rail nodes. The boundary conditions were set according to the support layout of the simple beam. The left fulcrums were constrained by all translational degrees of freedom and the rotation around the bridge and vertical direction. Moreover, the right fulcrums were constrained vertical translation, lateral translation, rotation around the bridge and rotation around the vertical direction.

The simple-support box beam bridge was in a double-track railway with a line spacing of 5 m and a gauge of 1435 mm. The rail whose standard cross-section was 60 kg/m was simulated by the BEAM44 beam element. The divided length of the rail element was 0.6 m according to the arrangement length of the fasteners. The fastener was simplified as a spring damping member whose vertical stiffness was $12.0 \cdot 10^7$ N/m, while lateral stiffness was $6.0 \cdot 10^7$ N/m. Moreover, its vertical damping coefficient was $15.0 \cdot 10^4$ N/(m/s), while the lateral damping coefficient was $12.0 \cdot 10^4$ N/(m/s). The rail node and rail lower node were connected by the fastener spring. The lower rail joints located on the upper part of the bridge girder were connected with the bridge girder using rigid arms. However, the lower rail joints located outside the end of the bridge were fixed. The train ran on the left track. The schematic diagram of the connection between the track and the bridge in the model is shown in Figure 2.

To ensure running smoothly when the train entered and exited, the steel rails were lengthened on the sides of the train entering and exiting the bridge. The finite element model of the bridge-track system with the prestressed steel wires is shown in Figure 3.

1.2. Model verification

In the past, the bridges were mostly discrete with solid elements, and the prestressed steel wires passing through the girder were coupled with the main girder through common nodes. A PC simple-support rectangular girder with a span of 3 m was taken as a sample in order to verify the feasibility of the beam element model with the prestressed steel wires. Moreover, its section size is 150 mm $\times$ 100 mm. Considering a single prestressed steel wire with an area of 139 mm² in the girder, a beam element model and a solid element model were established, respectively. The beam element model was simulated by the Beam44 element, and the solid element model was discrete by the Solid95 element. Moreover, the prestressed steel wires were simulated by the Link8 element. The finite element model is shown in Figure 4. Under the prestress of 1395 MPa, the comparison of static displacement of the two models is shown in Figure 5 (the result in the solid model is the node displacement at the centreline of the beam bottom surface). Moreover,

the comparison of the vertical natural frequency of the main beam is shown in Table 1.

Figure 5 shows that the static displacement curves of two models are in good agreement under the action of prestressed load. Moreover, the difference in vertical displacement does not exceed 3%. It can also be found from Table 1 that the difference of the vertical natural frequency between the two models is also within 3%. It shows that the

Table 1. Comparison of vertical natural frequencies of two models

Order	Beam element model	Solid element model	Vibration characteristics
1	38.788 Hz	38.832 Hz	First-order vertical bending
2	151.33 Hz	154.12 Hz	Second-order vertical bending

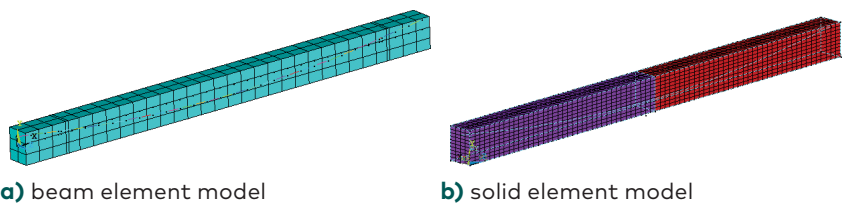


Figure 4. Finite element model of simply supported beams

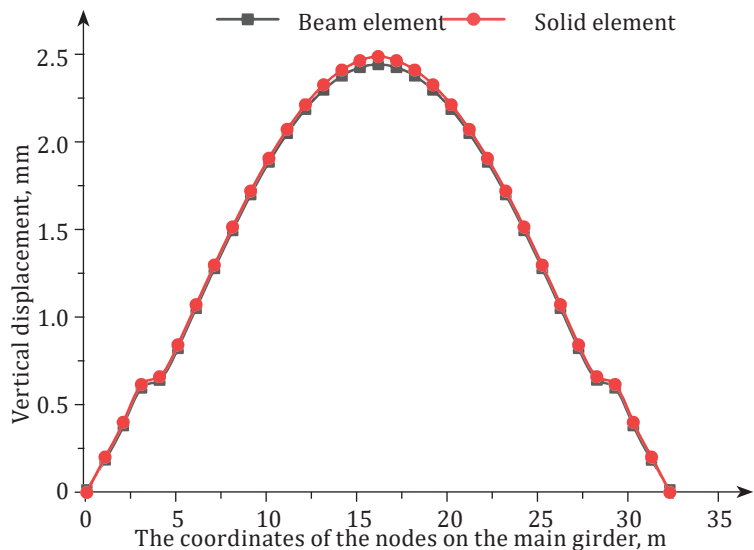


Figure 5. Comparison of static displacement between two models

beam element model with the prestressed steel wires can accurately simulate the vertical stiffness and mass distribution of the structure compared with the solid model with prestressed steel wires. The beam element model with the prestressed steel wires can greatly reduce the number of overall degrees of freedom of the bridge structure, improve the calculation efficiency, and has better calculation accuracy. Therefore, it is suitable to analyse complex vehicle-bridge coupling vibration subsequently.

2. The analysis method of vehicle-bridge coupled vibration in the high-speed railway

The bridge-track-train coupling system with the prestressed steel wires could be split into two subsystems: the train and bridge-track. Then it is possible to establish the finite element analysis models and motion equations, respectively. The two subsystems are coupled by the wheel-rail interaction relationship. Finally, the dynamic response can be obtained by using the time-domain method to solve the vehicle-bridge vibration equation.

2.1. The train model and motion equation

The train vehicle model can be predigested to a multiple rigid bodies system which has seven rigid bodies, including one car body, two bogies and four wheelsets. The connections between these rigid bodies are longitudinal, horizontal and vertical spring damping elements. There are also anti-rolling springs and anti-snaking damping dampers in the secondary suspension. As Figure 6 illustrates, each rigid body node includes up to 6 degrees of freedom.

Based on the equilibrium state of the train under self-weight, D'Alembert's principle is used to establish the motion equation, as shown in Eq. (1):

$$M_v \ddot{U}_v(t) + C_v \dot{U}_v(t) + K_v U_v(t) = F_v(t), \quad (1)$$

where K_v , C_v , M_v – the matrices of stiffness, damping and mass of the structure; U_v , $\dot{U}_v$, $\ddot{U}_v$ – the arrays of displacement, velocity and acceleration of the nodes of the train rigid body; F_v – the load matrix of the train which is mainly composed of wheel-rail forces regardless of vehicle weight.

2.2. Bridge-track model and motion equation with prestressed steel wires

The bridge-track structure model with the prestressed steel wires is established by the conventional FEA. Discretizing the main girder and the track by spatial beam elements and the rod elements is used to simulate the prestressing tendons. Taking the equilibrium position without the self-weight as the initial state, the motion equation of the bridge-track structure considering prestress is obtained:

$$M_B \ddot{U}_B(t) + C_B \dot{U}_B(t) + K_B U_B(t) = F_B(t), \tag{2}$$

where K_B , C_B , M_B – the matrices of stiffness, damping and mass of the structure. The damping matrix includes damping of the material of the bridge structure and the spring-damping element of the under-rail fasteners; U_B , $\dot{U}_B$, $\ddot{U}_B$ – the arrays of displacement, velocity and acceleration of the nodes of the bridge structure; F_B – the external force matrix of the structure, which is mainly composed of train weight, wheel-rail normal force and creep force (moment).

2.3. Wheel-rail spatial dynamic coupling relationship

The implementation of the wheel-rail spatial dynamic coupling relationship mainly includes the solution of the wheel-rail spatial

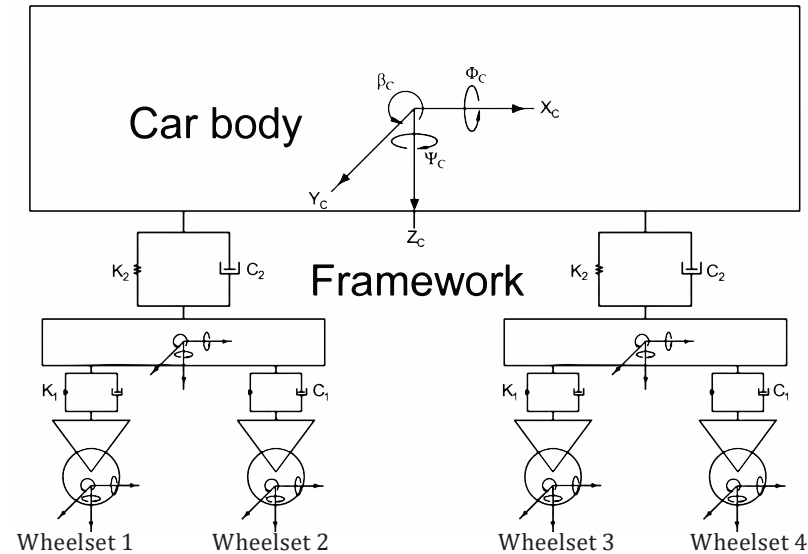


Figure 6. Side elevation of train model

contact geometric relationship, the wheel-rail normal force and the creep force (moment). In the new dynamic wheel-rail contact geometric relationship, the wheel-rail spatial contact geometric relationship is determined based on the trace method. From the spatial coordinates of the spatial trace on the wheel tread (x_c, y_c, z_c) and the coordinate values of the discrete points on the rail head (y_g, z_g) the normal compression of the wheel-rail contact is obtained through the calculation formula of absolute coordinates:

$$\varphi_z = Z_c - Z_g. \quad (3)$$

According to the normal compression, the Hertzian nonlinear contact theory is used to calculate the wheel track normal force. The Kalker's linear creep theory is used to calculate the wheel track longitudinal and transverse creep forces and rotational creep moments. Then the Shen-Hedrick-Elkins theory is used for nonlinear correction. Finally, the external load acting on the train subsystem and the bridge-track subsystem considering prestressed steel wires is obtained by converting the obtained wheel-rail creep force (moment) and normal force to the absolute coordinate system.

2.4. Solution to the vehicle-bridge coupled vibration

The bridge-track-train coupled system with the prestressed steel wires can be split into two subsystems: the bridge-track and train. Then the motion equations are established respectively, as shown in Eqs. (1) and (2). The two subsystems are connected with the equilibrium relationship of the wheel-rail interaction force. Then the new fast explicit numerical integration method and the Newmark- β method are combined to solve the motion equation of the train-rail-bridge coupled system. The details in every time step are shown below.

1. The displacement and speed of the vehicle-bridge coupled system are predicted by the new fast explicit integral method. According to the track irregularity value at the current moment, the trace method is used to determine the parameters of the wheel-rail contact geometric relationship.
2. On the basis of the wheel-rail contact geometry relationship, the wheel-rail interaction force is calculated. If the wheel and rail are not separated, the Hertz nonlinear elastic contact theory is used to solve the wheel-rail normal force and the Kalker linear theory is used to calculate the wheel-rail creep force. Finally, the Shen-Hedrick-Elkins theory is used for nonlinear correction. The wheel-rail interaction force is zero when the wheel and rail are separate.

3. The load matrix of subsystems of the train and bridge-track is obtained through the train weight and wheel-rail interaction force.
4. Eqs. (1) and (2) are solved respectively by the Newmark- β method to obtain the dynamic response of the train subsystem and the bridge-track subsystem in the current time step.

Based on Microsoft Visual Studio 10.0 platform, the dynamic analysis program of the train-track-bridge coupled system was compiled using Intel Visual Fortran language according to the above-mentioned vehicle-bridge coupled vibration analysis method. The program block diagram is shown in Figure 7.

2.5. Example of numerical verification of vehicle-bridge coupled vibration analysis

Taking the standard simple box girder and steel rails in the high-speed railways in Section 1.1 as an example, a 5-span bridge model was established. At first, the piers and prestressed steel beams were not considered in the model. Simply supported boundary conditions were set according to the actual support layout. Steel rails were laid between 300 m in front of the bridge head and 200 m behind the bridge tail to fully reflect the vibration state of the train on the bridge. The full bridge finite element model based on the common FEA software ANSYS was imported in the vehicle-bridge coupling analysis software SIMPACK to form the bridge analysis model. To decrease the number of model freedoms, only the left rail elements were established when the train was running on the single line. The whole bridge had 5382 nodes and 5921 elements (see Fig. 3).

The ICE3 high-speed passenger train, which consists of eight carriages, is selected for analysis. The leading and trailing vehicles are trailers and six vehicles in the middle are motor cars. One carriage consists of one car body, two frames and four wheelsets, a total of seven rigid bodies. The rigid bodies were linked by the spring damping elements. The vehicle model parameters are shown in literature mentioned above. The train formation and single vehicle model in SIMPACK is shown in Figure 8.

The dynamic responses of the bridge and train were obtained by self-made program and SIMPACK when the train passed the bridge-track model at 300 km/h. Figures 9 and 10 illustrate the comparison of the vertical and lateral displacement responses at the midspan of the third span of the bridge. Figures 11 and 12 illustrate the comparison of the vertical dynamic responses of the first car body.

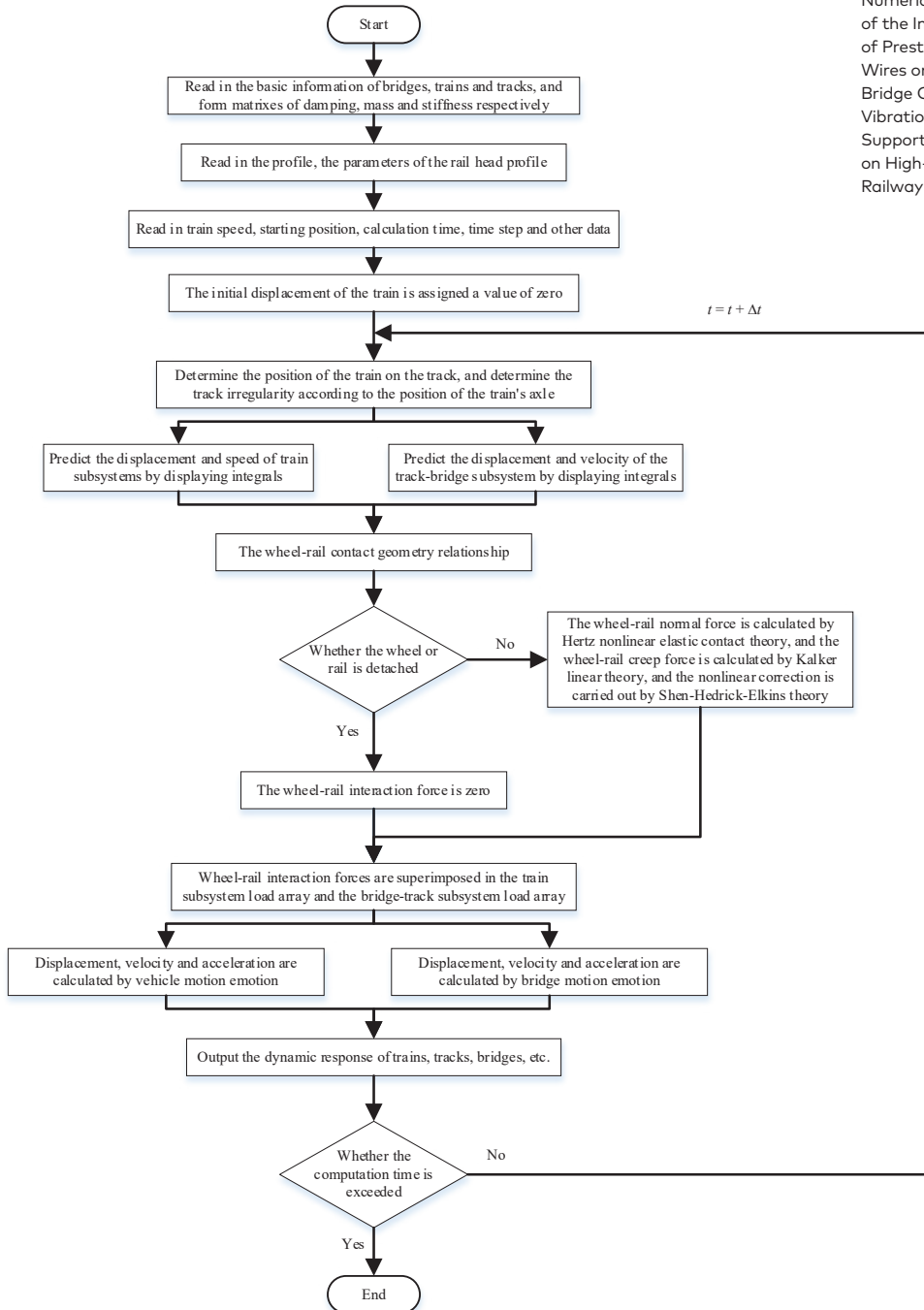
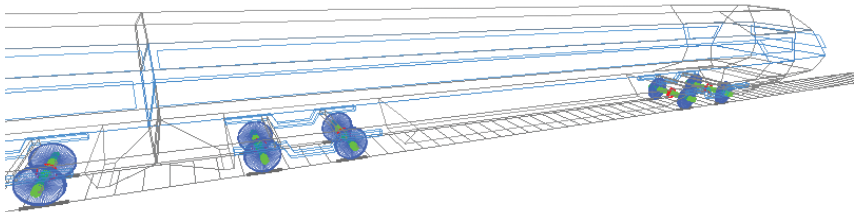


Figure 7. Flow chart of dynamic analysis program



a) train formation



b) single section vehicle

Figure 8. Train model in SIMPAK

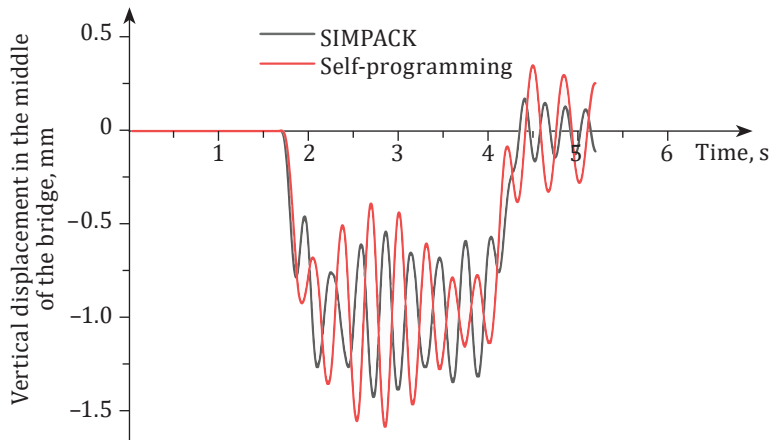


Figure 9. Comparison of vertical displacement time history at the midspan of the bridge

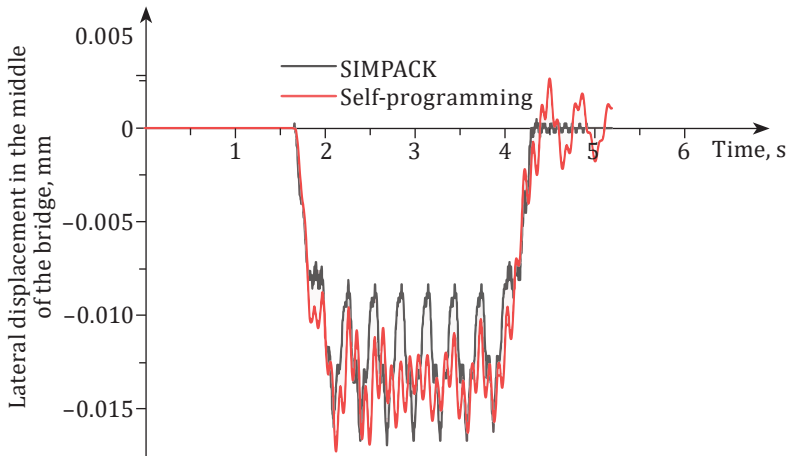


Figure 10. Comparison of lateral displacement time history at the midspan of the bridge

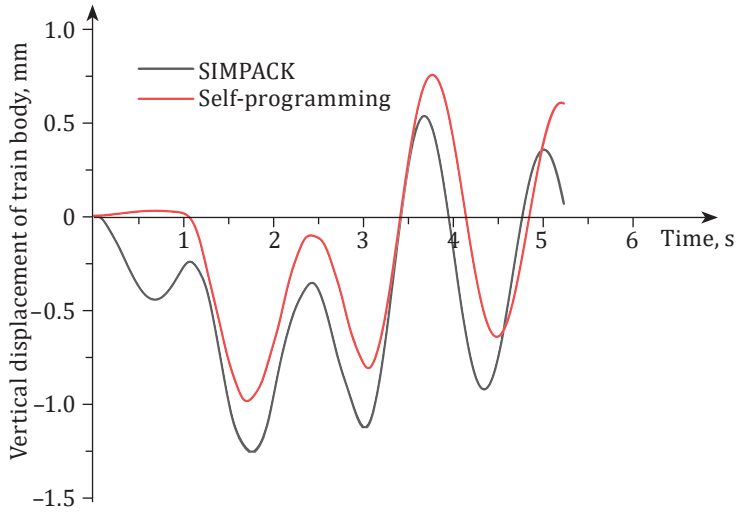


Figure 11. Comparison of vertical displacement time history at the vehicle

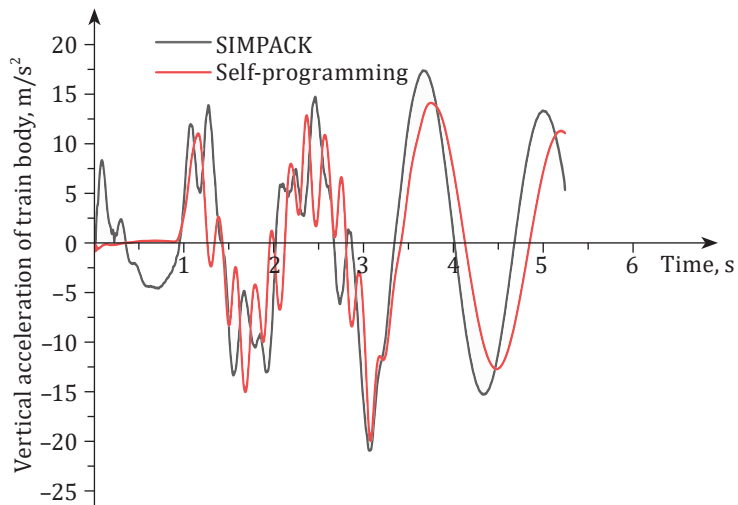


Figure 12. Comparison of vertical acceleration time history at the vehicle

It can be seen from Figs. 9–12 that the dynamic response trend obtained from the self-programmed program basically coincides with that obtained from SIMPACK. There are slight differences in value. Taking the complexity of the vehicle-bridge coupled vibration into account, the parameters and assumptions were slightly different in the calculation process in different analysis programs. The error of the calculated results was within the allowable range. Therefore, the self-compiled program could be used for the subsequent coupled vibration analysis of railway bridges since the calculation result of the self-compiled program in this article was basically consistent with that of SIMPACK software.

3. Analyses of the influence of prestressed steel wires on vehicle-bridge coupled vibration

On the basis of the finite element model of the vehicle and bridge mentioned above, the track irregularity value was simulated by the German low interference spectrum. The dynamic response of the vehicle and bridge was calculated by the self-compiled program when the train passed through the left line of the bridge at 300 kilometres per hour. The horizontal and vertical dynamic responses of the lead vehicle and middle span of the 3rd span were selected as the dynamic responses of the car body and bridge, respectively. Two working conditions were considered. There were two cases, including arranging only the steel wires described in Section 2.1 in the 3rd span simply supported beam and the beam without steel wires. The comparison of the peak dynamic responses of the vehicle and bridge under the two working conditions is shown in Table 2. Figures 13 and 14 illustrate the comparison of the vertical displacement and acceleration time history of the 3rd span bridge with and without prestressed steel wires.

Table 2. Comparison of peak dynamic response of vehicle and bridge

Orientation		Response	With prestress	Without prestress
Bridge	Lateral	Displacement, mm	$0.838 \cdot 10^{-4}$	$0.177 \cdot 10^{-4}$
		Acceleration, m/s^2	0.043	0.032
	Vertical	Displacement, mm	1.534	1.597
		Acceleration, m/s^2	0.105	0.331
Car body	Lateral	Displacement, mm	0.483	0.491
		Acceleration, m/s^2	0.015	0.016
	Vertical	Displacement, mm	1.568	1.434
		Acceleration, m/s^2	0.040	0.037

Table 2, Figures 13 and 14 show that the dynamic responses of the car body do not change obviously in the case of prestressed steel wires compared with that in the working conditions without prestressed steel wires. However, the dynamic response of the bridge is quite different, especially the vertical acceleration responses of the bridge

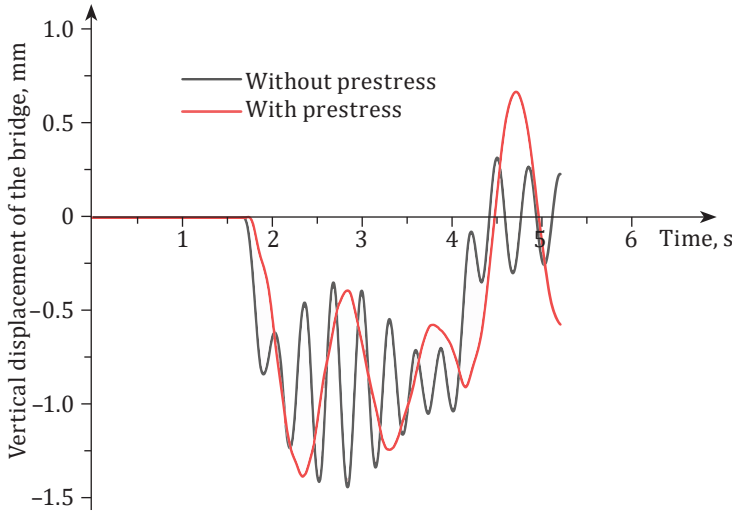


Figure 13. Comparison of vertical displacement time history at bridge midspan

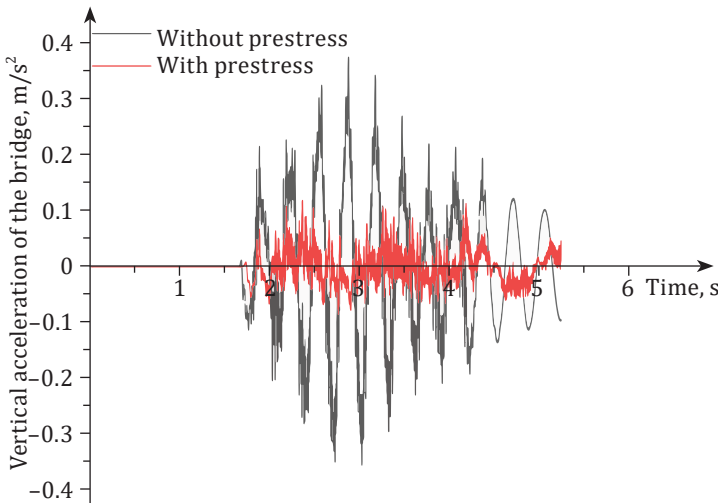


Figure 14. Comparison of vertical acceleration time history at bridge midspan

whose peak values reduce from 0.331 m/s² to 0.105 m/s² after laying out the prestressed steel wires. The reason causing this phenomenon is that the prestressed steel wires change the stiffness distribution of the bridge, which changes the vibration form of the bridge. It shows that the prestressed steel wires have major implications on the dynamic response of the bridge.

In practice, however, some possible situations happen, for example, the prestress loss of the bridge structure, the installation deviation during the construction of the pre-stressed steel strands and the train running speed, etc. To further analyse the influence of the prestressed steel wires on the coupled dynamic response of the high-speed railway simply supported beam bridge, a common 5-span double-track simple beam bridge with a single-span span of 32.6 m in high-speed railways and the ICE3 high-speed train consisting of eight carriages were selected for analysis. 27 prestressed steel wires were laid out in the third span bridge. Other parameters were the same as the above example. The situation of the train running at a constant speed at the bridge was simulated and the self-compiled program was used to calculate the dynamic response of the vehicle bridge. Taking the responses of the third span bridge and lead car body as research objects, the influence of effective prestress, prestressed steel wires eccentricity and speed of the vehicle on the dynamic responses of vehicle and bridge were discussed.

3.1. Influence of effective prestress

Five cases of different prestress values were defined: 0 MPa, 1200 MPa, 1300 MPa, 1400 MPa and 1500 MPa. The dynamic responses of the train and bridge were analysed when trains passed the bridge at 300 km/h.

3.1.1. Bridge dynamic response

The vertical and lateral displacements and acceleration peak values at the midspan of the bridge under five different prestress values are shown in Table 3. The vertical displacements and acceleration time history curves at the midspan are shown in Figures 15 and 16, respectively.

Table 3. Peak dynamic response at bridge midspan

Prestress effect, Mpa	0	1200	1300	1400	1500
Vertical Displacement, mm	1.522	1.462	1.445	1.534	1.474
Vertical Acceleration, m/s ²	0.121	0.113	0.102	0.105	0.107
Lateral Displacement, mm	0.082	0.084	0.083	0.084	0.085
Lateral Acceleration, m/s ²	0.042	0.043	0.043	0.043	0.044

It can be seen from Table 3 and Figures 15 and 16 that with the increase of the prestress, the peak values of the vertical dynamic responses at the midspan show a tendency of decreasing and then increasing. The peak values of the vertical displacements and acceleration at the midspan of the bridge are minimum when the prestress of the steel wires is 1300 MPa. The lateral dynamic responses at the midspan of the bridge are less affected by the prestress of the steel wires.

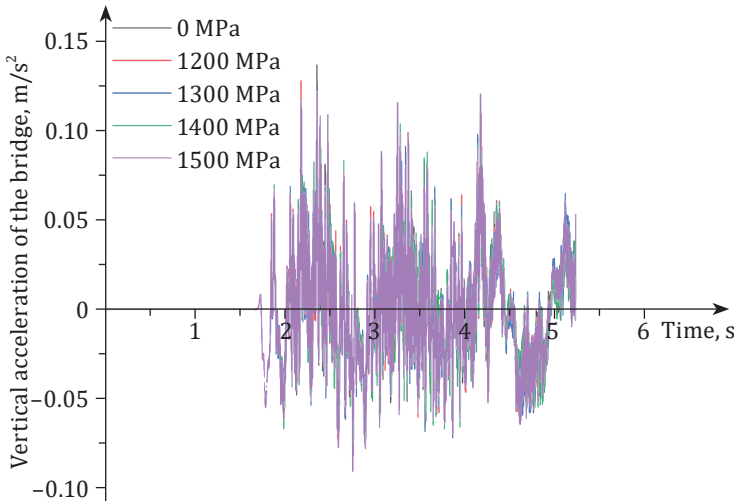


Figure 15. Comparison of vertical acceleration time history at bridge midspan

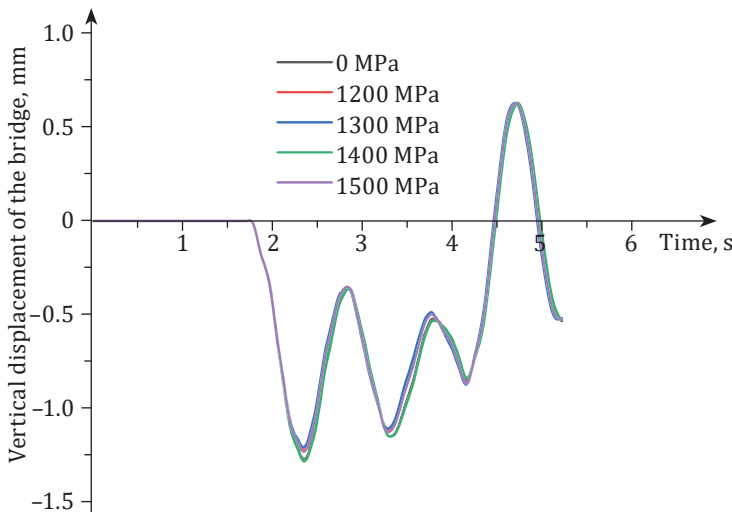


Figure 16. Comparison of vertical displacement time history at bridge midspan

3.1.2. Vehicle body dynamic response

Table 4 shows the vertical and lateral displacements and acceleration peak values of the vehicle body under five different prestress values.

Table 4. Peak dynamic response of vehicle

Prestress effect, Mpa	0	1200	1300	1400	1500
Vertical Displacement, mm	1.568	1.569	1.569	1.568	1.569
Vertical Acceleration, m/s ²	0.040	0.040	0.040	0.040	0.040
Lateral Displacement, mm	0.484	0.484	0.484	0.483	0.484
Lateral Acceleration, m/s ²	0.015	0.015	0.015	0.015	0.015

It is obvious that the dynamic response of the car body is slightly affected by the prestressing of the steel wires.

3.2. The influence of steel wires eccentricity

In the engineering practice, the position of the steel wires may not be in line with the design position due to construction deviation or other reasons. Considering the thickness of the steel bar protection layer, the cross-sectional shape and the overall force, the overall deviations of the steel wires from the design position were considered +5 cm, 0 cm, -5 cm, -10 cm and -15 cm, where + means upward deviation and - means downward deviation. The dynamic responses of the bridge were calculated when the train passed the bridge at 300 km/h.

3.2.1. Bridge dynamic response

The peak values of dynamic responses at the bridge midspan under five kinds of eccentricities of the steel wires are shown in Table 5. The vertical displacements and acceleration time history curves at the bridge midspan under different eccentricities are shown in Figures 17 and 18, respectively.

Table 5. Peak dynamic response at bridge midspan

Offset, cm	+5	0	-5	-10	-15
Vertical Displacement, mm	1.290	1.534	1.548	1.391	1.449
Vertical Acceleration, m/s ²	0.105	0.105	0.106	0.110	0.112
Lateral Displacement, mm	0.081	0.084	0.086	0.087	0.085
Lateral Acceleration, m/s ²	0.043	0.043	0.042	0.042	0.040

It can be seen from Table 5, Figures 17 and 18, the vertical displacements of the bridge at first increase and then decrease when the steel wires are offset downward. However, the vertical displacements of the bridge reduce from 1.534 mm to 1.290 mm when shifting upwards. The reduction is greater than the working condition when there is downward shift. The eccentricity of steel wires has little effect on the dynamic responses of the transverse bridge and the vertical accelerations of the bridge.

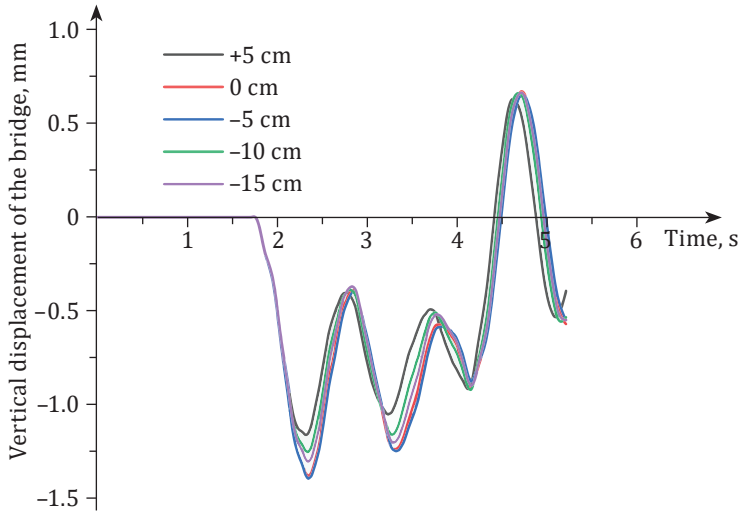


Figure 17. Comparison of vertical displacement time history at bridge midspan

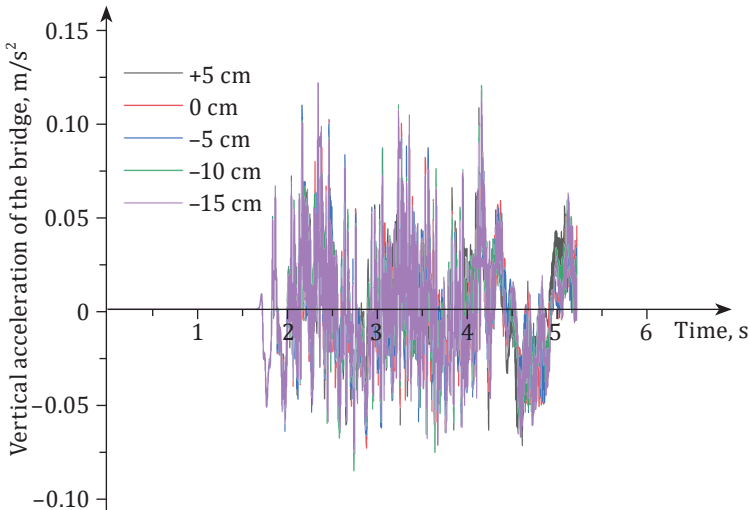


Figure 18. Comparison of vertical acceleration time history at bridge midspan

3.2.2. Vehicle body dynamic response

The acceleration peak values and the vertical and lateral displacements of the car body under different steel wires eccentricities are shown in Table 6.

Table 6. Peak dynamic response of vehicle

Eccentricity, m	1.519	1.569	1.619	1.669	1.719
Vertical Displacement, mm	1.569	1.568	1.569	1.570	1.570
Vertical Acceleration, m/s²	0.040	0.040	0.040	0.040	0.040
Lateral Displacement, mm	0.486	0.483	0.484	0.486	0.485
Lateral Acceleration, m/s²	0.015	0.015	0.015	0.015	0.015

It can be seen from Table 6, the influence of the eccentricity of the steel wires on the dynamic response of the vehicle is not obvious.

3.3. The impact of vehicle speed

Let us suppose that the prestress value of the steel wires is 1400 MPa and other parameters are kept unchanged. The dynamic responses of the bridge were obtained when the trains passed the bridge at varying speeds. The speeds selected for analysis were 200 km/h, 250 km/h, 300 km/h, 350 km/h and 420 km/h, respectively.

3.3.1. Bridge dynamic response

At different vehicle speeds, the peak values at the bridge midspan dynamic responses are shown in Table 7. The vertical displacements and acceleration time history curves at the midspan are shown in Figures 19 and 20.

Table 7. Peak dynamic response at bridge midspan

Vehicle Speed, km/h	200	250	300	350	420
Vertical Displacement, mm	1.334	1.441	1.534	1.618	1.623
Vertical Acceleration, m/s²	0.078	0.078	0.105	0.149	0.273
Lateral Displacement, mm	0.027	0.027	0.084	0.044	0.029
Lateral Acceleration, m/s²	0.019	0.024	0.043	0.032	0.034

It can be seen from Table 7, Figures 19 and 20 that the vertical dynamic responses at the midspan of the bridge keep increasing with the speed growing. The peak vertical displacements of the bridge increase from 1.334 mm to 1.623 mm. The peak values of the lateral dynamic responses of the bridge reach an extreme value as the speed is 300 km/h.

It can be seen from Table 7, Figures 19 and 20 that the vertical dynamic responses at the midspan of the bridge keep increasing with the speed growing. The peak vertical displacements of the bridge increase from 1.334 mm to 1.623 mm. The peak values of the lateral dynamic responses of the bridge reach extreme value as the speed is 300 km/h.

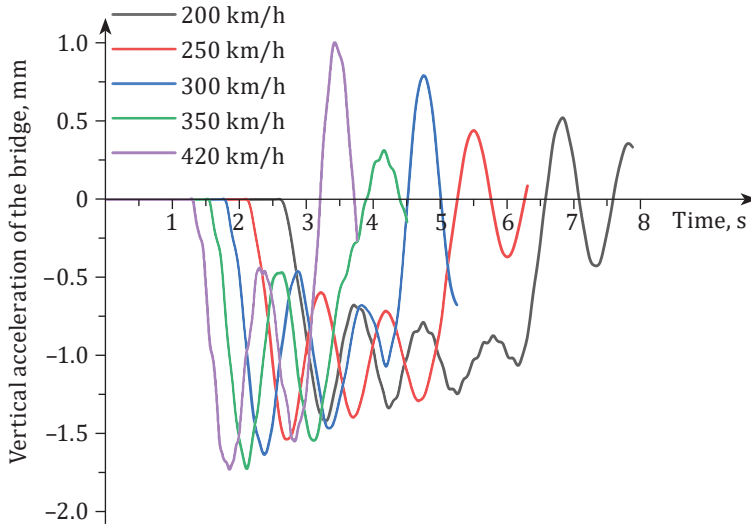


Figure 19. Comparison of vertical displacement time history at bridge midspan

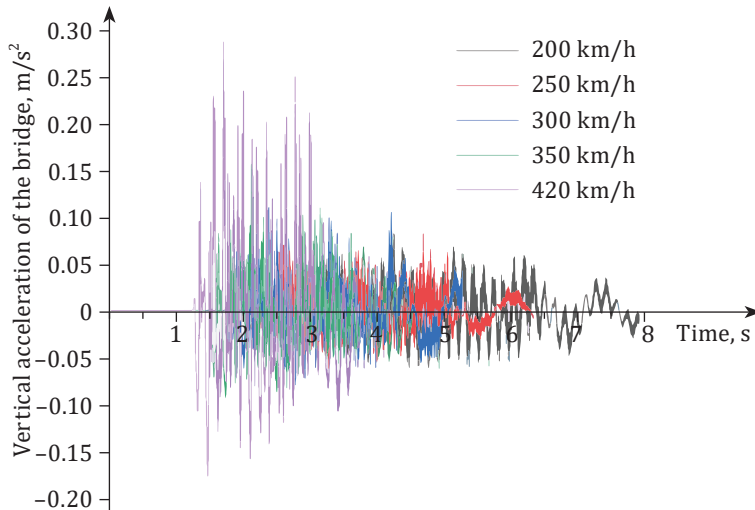


Figure 20. Comparison of vertical acceleration time history at bridge midspan

3.3.2. Vehicle body dynamic response

Table 8 shows the peak values of vehicle body dynamic responses of different speeds.

Table 8. Peak dynamic response of vehicle

Vehicle Speed, km/h	200	250	300	350	420
Vertical Displacement, mm	1.641	1.874	1.568	1.167	1.193
Vertical Acceleration, m/s ²	0.029	0.036	0.040	0.030	0.029
Lateral Displacement, mm	0.600	0.654	0.483	0.353	0.237
Lateral Acceleration, m/s ²	0.018	0.017	0.015	0.015	0.019

It can be seen from Table 8 that as the speed increases, the peak values of the vertical accelerations of the vehicle body reach the maximum value when the vehicle speed is 300 km/h, while the lateral accelerations of the vehicle body reach the minimum value when the vehicle speed is 300 km/h. The peak values of the lateral displacements of the body gradually decrease from 0.600 mm to 0.237 mm.

Conclusions

1. After laying out prestressed steel wires in the bridge, the dynamic responses of the car body did not change significantly compared with the case of no prestressed steel wire. However, the dynamic responses of the bridge changed significantly, especially the vertical acceleration responses of the bridge whose peak values reduced from 0.331 m/s² to 0.105 m/s².
2. With the increase of the prestress of the steel wires, the peak values of the vertical dynamic response at the midspan of the bridge first decreased and then increased. The peak values of the vertical dynamic responses of the bridge reached the minimum when the prestress value of the steel wires was 1300 MPa. The lateral dynamic responses of car bodies and bridges were less affected by the prestress of steel wires.
3. Compared to the design position, the vertical displacements of the bridge reduced from 1.534 mm to 1.290 mm when the steel wires were shifted upwards. The amplitude of the reduction was greater than that of the case when the steel wires were shifted downwards. The steel wires eccentricity had no obvious influence on the vertical acceleration of the bridge, the vehicle body and the dynamic responses of the transverse bridge.

4. With the increase of vehicle speed, the vertical dynamic responses at the midspan of the bridge continued to increase, and the peak values of the lateral displacement of the vehicle body decreased constantly. The lateral dynamic responses of the bridge and the acceleration responses of the vehicle body reached extreme value when the vehicle speed was 300 km/h.
5. The research results are based on one type of bridge and a certain vehicle type in the high-speed railway. For other different bridge structures, train types and line types, the influence law of the prestress on the dynamic characteristics of the vehicle-bridge coupling system can be obtained through a similar analysis.

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References

- Bonopera, M., Chang, K. C., Chen, C. C., Sung, Y. C., & Tullini, N. (2019). Experimental study on the fundamental frequency of prestressed concrete bridge beams with parabolic unbonded tendons. *Journal of Sound and Vibration*, 455, 150–160. <https://doi.org/10.1016/j.jsv.2019.04.038>
- Bruzelius, F., Nielsen, J. C. O., & Landén-Hillemyr A. (2020). Advances in Dynamics of Vehicles on Roads and Tracks. In *Proceedings of the 26th Symposium of the International Association of Vehicle System Dynamics, IAVSD 2019*, Gothenburg, Sweden. <https://www.springer.com/gp/book/9783030380762>
- Chen, A. S. (2012). *Vehicle-bridge coupling analysis of prestressed simply supported beam*. MSc Thesis, Jilin University. <https://kns.cnki.net/kcms/detail/detail.aspx?dbcode=CMFD&dbname=CMFD2012&filename=1012367986.nh&uniplatfor=m=NZKPT&v=tP4VmI4FUTyUGz6FfV8Jhn1zJjwVgjqGJSi1Vd8GDckixOY3ZPbvG0NHsHHCCp63>
- Chen, D. H. (2011). *Study on dynamic response and vibration control of vehicle-bridge coupling system under earthquake*. Dissertation, Central South University. <https://kns.cnki.net/kcms/detail/detail.aspx?dbcode=CDFD&dbname=C-DFD1214&filename=1012474095.nh&uniplatfor=m=NZKPT&v=CYQ49dWlq%25mmd2FoF1HfxmnH3KgwvyCYoN4jdwezKvpKxQ%25mmd2FxbESyiyMNRw%25mmd2F%25mmd2BL4sW%25mmd2Bijwg>

- China Railway Engineering Design and Consulting Group Co. (2013). Ltd. *General reference drawing for railway engineer-ing construction-350 kilometers per hour high-speed rail-way precast ballastless track post-tensioned prestressed concrete simply supported box girder (double track)*. Economic Planning and Research Institute of the Ministry of Railways (Beijing).
- Dall'Asta, A., & Dezi, L. (1996). Discussion of "Prestress force effect on vibration frequency of concrete bridges". *Journal of Structural Engineering*, 122(4), 458–458. [https://doi.org/10.1061/\(ASCE\)0733-9445\(1996\)122:4\(458\)](https://doi.org/10.1061/(ASCE)0733-9445(1996)122:4(458))
- Dong, C. J., Liu, S. Z., & Li, A. J. (2018). Influence of prestressing effect on dynamic characteristics of prestressed concrete simply supported beams. *Journal of Lanzhou Jiaotong University*, 37(05), 13–37. <https://kns.cnki.net/kcms/detail/detail.aspx?dbcode=CJFD&dbname=CJFDLAST2018&filename=LZTX201805003&v=6nrNRGmUw0x8EYos6iFM6Gv72Rj9s3xVMfk%25mmd2BulsPUHGMtPb1KEFZhqh13HkuOrJl>.
- Fang, D. P. (2020). Dynamic behavior of externally prestressed continuous beam considering second-order effect. *Mathematical Problems in Engineering*, 2020(1), 1–9. <https://doi.org/10.1155/2020/4397262>
- Guo, K. Q., Jia, Y. M., Yu, G. L., Wang, J. W., & Zhang, G. H. (2018). Study on the natural vibration frequency of a simply supported beam with linear unequal length prestressed tendons. *Journal of Vibration and Shock*, 37(17), 230–235. <https://kns.cnki.net/kcms/detail/detail.aspx?dbcode=CJFD&dbname=CJFDLAST2018&filename=ZDCJ201817033&uniplatform=NZKPT&v=qwsVlvqcjYHB%25mmd2FxC%25mmd2BfkftW%25mmd2Bvz1bfHqp9bJaj1KQH46cPbABY3E8RUkmGNoVChvDXO>
- Ghaemdoost, M. R., Wang, F., Li, S., & Yang, J. (2021). Numerical investigation on the transverse vibration of prestressed large-span beams with unbonded internal straight tendon. *Materials*, 14(9), 2273. <https://doi.org/10.3390/ma14092273>
- Li, P., Huan, S., Tan, X. Q., & Tao, W. J. (2018). Research on natural frequency of prestressed simply supported beam. *J. Guangzhou Univ., Nat. Sci. Ed.*, 17(02), 55–60. <https://kns.cnki.net/kcms/detail/detail.aspx?dbcode=CJFD&dbname=CJFDL-AST2018&filename=GUDZ201802010&v=CA5xBAx2h8DFEC1XXag1VoMWh0v0y9x3JdqJXhcKsmvu2eCRblw1i8nIS-44dqwCy>
- Noble, D., Nogal, M., O'Connor, A. J., & Pakrashi, V. (2014). The effect of prestress force magnitude on the natural bending frequencies of prestressed concrete structures. In *Proceedings of the 23rd Australasian Conference on the Mechanics of Structures and Materials (ACMSM23)*, Byron Bay, Australia.
- Peng, L., & Wang, Y. (2021). Differential quadrature method for vibration analysis of prestressed beams. *E3S Web of Conferences*, 237, 03029. <https://doi.org/10.1051/e3sconf/202123703029>
- Ren, X., Tan, X., Qi, H. P., & Ma, M. (2018). Analysis of dynamic characteristics and influencing factors of prestressed concrete continuous beam bridge. *J. Chongqing Jiaotong Univ., Nat. Sci.*, 37(10), 8–12. <https://doi.org/10.3969/j.issn.1674-0696.2018.10.-02>
- Saiidi, M., Douglas, B., & Feng, S. (1994). Prestress force effect on vibration frequency of concrete bridges. *Journal of Structural Engineering*, 120(7), 2233–2241. [https://doi.org/10.1061/\(ASCE\)0733-9445\(1994\)120:7\(2233\)](https://doi.org/10.1061/(ASCE)0733-9445(1994)120:7(2233))

- Toyota, Y., Hirose, T., Ono, S., & Shidara, K. (2017). Experimental study on vibration characteristics of prestressed concrete beam. *Procedia Engineering*, 171, 1165–1172. <https://doi.org/10.1016/j.proeng.2017.01.483>
- Wang, X.M. (2007). *ANSYS Engineering Structure Numerical Analysis*. Beijing: China Communication Press.
- Xia, H., Zhang, N., & Guo, W. W. (2018). *Dynamic Interaction of Train-Bridge Systems in High-Speed Railways*. Springer.
<http://doi.org/10.1007/978-3-662-54871-4>
- Xiao, Y. (2018). Analysis of natural vibration characteristics of typical bridge structures on existing heavy haul railway. *J. East China Inst. Technol., Nat. Sci.*, 41(03), 282–288. <https://doi.org/10.3969/j.issn.1674-3504.2018.03.013>
- Xiang, C. Q. (2015). *Study on the dynamic response and optimal wind barrier parameters of the following vehicle-track-bridge coupling system under strong crosswind*. Dissertation, Central South University.
- Xiang, Y. Q., Qiu, Z., & Bishnu, G. G. (2017). Natural frequency analysis of externally prestressed steel-concrete composite beams considering slippage effect. *J. Southeast Univ. (Chin. Ed.)*, 47(1), 107–111.
<https://doi.org/10.3969/j.issn.1001-0505.2017.01.019>
- Xu, Y., Yang, Z. H., Shen, P., & Wang, X. B. (2019). Influence of different eccentric distance and prestress level on fundamental frequency of simply supported beam. *J. Highw. Transp. Res. Dev. (Chin. Ed.)*, 15(07), 237–238.
- Yu, X. Z. (2012). *Natural frequency analysis of prestressed simply supported beam*. MSc Thesis, Dalian Maritime University. (In Chinese) <https://kns.cnki.net/kcms/detail/detail.aspx?dbcode=CMFD&dbname=CMFD2012&filename=1012342031.nh&-uniplatform=NZKPT&v=P0mTXkP70mFPLJKf4N9IAZG6Pk1PG08Ox9nxbF9d4qu7VH1toKd%25mmd2F6gMypIga9tw>
- Zhao, Y., Jia, R. M., Wang, R., & Wang, J. W. (2019). Analysis of influencing factors of natural vibration frequency of three-span prestressed continuous steel box girder. *Steel Construction (Chinese & English)*, 34(07), 75–79.
<https://doi.org/10.13206/j.gjg201907015>
- Zhong, C. L., Liang, D., Zhang, Y. L., & Wang, J. (2020). Calculation of natural vibration frequency of externally prestressed steel-concrete composite simply supported beam. *J. Jilin Univ., Eng. Technol. Ed.*, 50(6), 2159–2166. (In Chinese) <https://doi.org/10.13229/j.cnki.jdxbgxb20191001>

Notations

Abbreviations

PC – Prestressed Concrete

FEA – Finite Element Analysis