Dynamic Response of Train-Long-Span Bridge Coupled System under Wave Loading: A Postprint

Authors: Luo Hao

Date: 2026-01-30T23:13:32+00:00

Abstract

Focusing on the coupling behavior of a vehicle-bridge system under wave loading, this study takes a certain cross-sea cable-stayed bridge carrying both highway and railway traffic as the research object. Based on fifth-order Stokes wave theory and the JONSWAP wave spectrum, a numerical wave flume model for deep-water conditions is established using Fluent software, and the wave loads corresponding to significant wave heights with return periods of 5, 20, 50, and 100 years are calculated. A co-simulation framework integrating Ansys and Simpack is then adopted to analyze the dynamic response of the vehicle-bridge coupling system under wave loading. The results indicate that the dynamic response of the vehicle-bridge coupling system increases progressively with increasing train speed and wave height; due to the influence of wheel-rail forces, the lateral displacement curve of the bridge under combined wave-train action exhibits a markedly different profile from that under wave action alone; under different operating conditions, the peak lateral displacement at mid-span does not occur when the train reaches the mid-span position, implying that wave loading plays a dominant role in the lateral displacement of the bridge; as the wave height increases, all dynamic indices of the train increase gradually; when the wave height reaches 8.92 m or above and the train speed reaches 250 km/h, the train dynamic response becomes more pronounced, and accidents are more likely to occur.

Full Text

Abstract

This study addresses the coupled interaction problem of train-bridge systems under wave loading, taking a highway-railway sea-crossing cable-stayed bridge as the research object. Based on fifth-order Stokes wave theory and the Jonswap spectrum, a numerical wave flume model for deep-water environments was established using Fluent software to calculate wave loads corresponding to wave heights with return periods of 5, 20, 50, and 100 years. Combined simulation using Ansys and Simpack software was employed to analyze the dynamic response of the train-bridge coupling system under wave loading. The results indicate that the dynamic response of the train-bridge coupling system gradually increases with increasing train speed and wave height parameters. Due to the influence of wheel-rail forces, significant differences exist between the lateral displacement curves of the bridge under combined wave-train action compared with those under wave action alone. Under different operating conditions, the peak lateral displacement at mid-span does not occur when the train reaches the mid-span position, indicating that wave loading plays a dominant role in bridge lateral displacement. As wave height increases, various dynamic indicators of the train gradually increase. When wave height reaches 8.92 m or above and train speed reaches 250 km/h, the train exhibits greater dynamic response and is more prone to accidents.

Keywords: wave loading; fifth-order Stokes wave; train-bridge coupling; dynamic response; safety and comfort

Introduction

With the rapid development of railway mileage in China, railway bridge construction has gradually expanded from inland areas to coastal regions, with increasingly larger spans and more complex environmental conditions, particularly the impact of wave loading. Long-span bridges often exhibit significant flexibility in design, and wave loading poses certain threats to both the normal serviceability of bridges and train operation safety. Research on the coupled interaction between train and bridge systems has become particularly important. Liu Bin [1] analyzed the dynamic response of deep-water bridges under wave action; Wu Mingjun et al. [2] studied the dynamic response of bridge piers under combined earthquake and wave action; Li Zhongxian et al. [3-4] analyzed the dynamic response of deep-water bridges and piers under wave action and combined earthquake-wave action; Wu Anjie et al. [5] analyzed the dynamic response of deep-water bridge piers under combined wave-current and earthquake action using nonlinear Morison equations; Wang Lianbin [6] analyzed the influence of water depth and pier stiffness on train-bridge system dynamic response based on Ansys; Luo Hao et al. [7-8] analyzed the dynamic response patterns of sea-crossing bridges under wave action and combined wave-earthquake action; Lei Hujun et al. [9] studied the influence of seismic hydrodynamic pressure on train-bridge system dynamics using the rigid column method with input of measured seismic waves. The aforementioned scholars primarily focused on the dynamic response of bridge substructures under wave loading, neglecting the influence of superstructure stiffness and mass on bridge dynamic response, with limited analysis of train-bridge system dynamic response under wave loading.

Therefore, further research is necessary.

1. Engineering Overview

This study takes an existing highway-railway sea-crossing cable-stayed bridge as the research object. The bridge span arrangement is 132 m + 196 m + 532 m + 196 m + 132 m, with a main girder consisting of a plate-truss composite steel truss structure with auxiliary trusses. The upper deck is a six-lane highway with a design speed of 100 km/h, while the lower deck carries a double-track railway with a design speed of 200 km/h. The bridge site is located at the boundary between subtropical and tropical monsoon zones in China, significantly affected by wind and wave climates, with a 100-year return period design wave height of 9.69 m. The overall bridge layout is shown in Figure 1 [Figure 1: see original paper].

2. Wave Conditions

2.1 Wave Condition Formulation

Based on ocean wave theory and calculation principles [10], the relationship between cumulative wave height and mean wave height, as well as the cumulative probability and mean period, can be obtained. The design of the numerical wave flume for adverse conditions [11-12] requires wave parameters. The water depth environment at the bridge location is set at 40 m. Wave parameters for different return periods are listed in Table 1 .

2.2 Wave Spectrum

The Jonswap spectrum was obtained from measurements conducted in the North Sea. The expression derived from spectral analysis is used to calculate wave power spectral densities corresponding to wave heights for return periods of 5, 20, 50, and 100 years. The power spectral density curves exhibit narrow-band distribution and increase with wave height. The cross-section of the main girder is shown in Figure 2 [Figure 2: see original paper], and the power spectral densities for different wave heights are shown in Figure 3 [Figure 3: see original paper].

3. Numerical Wave Flume

For the establishment of the numerical wave flume, the water depth environment at the bridge location is set at 40 m. The flume dimensions satisfy all working conditions, and the bridge tower geometry model is created using Ansys Workbench. The numerical wave flume is designed based on the lengths of the wave generation and absorption zones. With a design water depth of 40 m, the wave generation zone is set to be a multiple of the maximum wavelength (560 m), and the absorption zone is set to be a multiple of the maximum wavelength (125 m).

During mesh generation for the numerical wave flume, a 0.15 m mesh refinement is applied along the wave propagation direction from the starting point to the bridge tower, while vertical mesh refinement of 0.25 m is implemented from the free surface to both ends of the flume. To accurately reflect the wave forces on the tower wall surfaces, boundary layer refinement to 0.1 m is applied to the longitudinal grids at the tower flow domain, after which the mesh is converted to hexahedral structured grids.

Wave absorption employs the momentum damping method [15-16], which eliminates or counteracts wave forces by establishing a momentum source opposite to the wave propagation direction in the absorption zone at the end of the numerical wave flume. The momentum equation expression is: + u∂ + u∂ + v∂ = -1 = g -1 + v∂ + v∂) - μu) - μv, where ρ is the fluid relative density, u and v are the horizontal and vertical velocity components of wave propagation, p is the pressure on fluid mesh cells, t is the wave propagation time, g is gravitational acceleration, and μ is the absorption momentum coefficient.

To verify the correctness of wave generation and absorption in the flume and analyze wave propagation, monitoring points are established in front of the bridge tower to compare theoretical calculations of the wave surface function with simulated values. Within the first 27 seconds at the front of the bridge tower, as waves are still propagating, the wave surface reaches its peak, with the water surface in a developing state and minimal wave height attenuation. The waveforms from theoretical and numerical calculations show good agreement. The mesh generation and boundary conditions of the numerical wave flume are shown in Figure 4 [Figure 4: see original paper].

3.1 Wave Generation and Absorption Principle

This study employs the velocity inlet method for wave generation, providing water particles at the inlet boundary with horizontal and vertical velocities as well as wave surface elevation to generate periodic and uniformly distributed waves. Fifth-order Stokes wave theory is adopted as it accounts for high-order nonlinear effects and can more accurately describe water wave phenomena [13]. The wave surface elevation and velocity equations are as follows:

The wave surface elevation equation is: $\eta = 1$, where the coefficients are λ cos(kx - ωt) = λ_2B = λ_3B = λ_4B = λ_5B.

The horizontal velocity is: u = c[+ λ_4B + λ_5B nk(z + d) nk(z + d)].

The vertical velocity is: u_z = c nλ sinh[nk(z + d)].

Here, ω is the angular frequency, k is the wave number, d is the water depth, and other coefficients are detailed in reference [14].

The comparison between theoretical and simulated wave surface values is shown in Figure 5 [Figure 5: see original paper]. After waves pass the bridge tower and enter the absorption zone, the wave surface gradually flattens, reaching the still water level at the end of the absorption zone, demonstrating the effective wave absorption capability. By implementing wave propagation and its interaction with the bridge tower in the numerical wave flume, wave loads under various working conditions are simulated, and the total wave force at the tower-wave interface is solved. The wave force on the bridge tower gradually increases with wave height and varies periodically with time. As wave height increases, the time for wave force to reach its peak becomes longer, and the period becomes significantly larger. The time histories of lateral wave forces under different wave heights are shown in Figure 7 [Figure 7: see original paper].

4. Train-Bridge Coupling Model Under Wave Action

The train-bridge coupling model is a rigid-flexible interaction model [17-18]. This study employs the flexible track method for rigid-flexible coupling simulation of the bridge system. To improve coupling calculation accuracy and efficiency, the following steps are implemented: establish the cable-stayed bridge and rail finite element model in Ansys, perform substructure analysis to reduce bridge degrees of freedom, generate the bridge flexible body file (∗.fbi) through the finite element interface program, then import it into Simpack; read track parameters and generate the track flexible body (∗.ftr), then connect these two flexible bodies through force elements. The train couples with the flexible track through wheel-rail relationships. The single train car model adopts the CRH2 type, with a formation consisting of 4 powered cars and 4 trailers. Red cars in the diagram are powered cars, blue cars are trailers. Wave force, as a time-varying load, is applied to the bridge tower and piers as external excitation in Simpack. Track irregularity adopts the German low-interference spectrum.

The bridge element types are Beam188 for girders, Shell181 for slabs, and Link180 for cables and hangers, as listed in Table 2 . The bridge tower and piers are modeled accordingly. The coupled train-track-bridge dynamics model under wave loading is shown in Figure 8 [Figure 8: see original paper].

4.1 Wave Force Loading Verification

To verify the correctness of wave force loading in the coupling model, the lateral displacement response at the main span mid-point of the bridge under a 9.69 m wave load is calculated separately using finite element software and multibody dynamics software Simpack. The comparison of displacement response histories between the two bridge models is shown in Figure 9 [Figure 9: see original paper]. The results demonstrate the correctness of wave force loading in Simpack, and this method can be used for subsequent train-bridge coupling dynamic response analysis.

5. Dynamic Response Analysis

5.1 Bridge Dynamic Response Under Combined Loading

To investigate the effects of combined train and wave loading on the bridge, the dynamic response of the bridge under various wave heights and train speeds of 200 km/h and 250 km/h is analyzed. The lateral dynamic response results at the main span mid-point are presented in Figure 10 [Figure 10: see original paper].

The dynamic response of the train-bridge coupling system gradually increases with increasing train speed and wave height. Due to wheel-rail forces, significant differences exist between the lateral displacement curves under combined wave-train action and those under wave action alone. The peak lateral displacement at mid-span does not occur when the train reaches the mid-span position, indicating that wave loading dominates the lateral displacement of the bridge.

Under a train speed of 200 km/h, the mid-span lateral displacement peaks for different wave heights are 0.021, 0.029, 0.036, and 0.055 m, representing increases of 38.1%, 24.1%, and 52.8% respectively compared with the case without wave loading, demonstrating that wave loading significantly increases mid-span lateral displacement at the design speed.

As wave height increases, the amplification of mid-span lateral displacement under wave loading gradually increases. The amplification is most pronounced under combined loading with a train speed of 250 km/h. The maximum lateral displacement response occurs under combined loading with a wave height of 8.92 m. For the 9.69 m wave height case, the deflection-to-span ratio is 1/7093, which is less than the code limit of 1/4000.

Under a train speed of 200 km/h, the mid-span lateral acceleration peaks for different wave heights are 0.039, 0.050, 0.080, and 0.117 m/s^2, representing increases of 28.2%, 37.5%, and 46.25% respectively, indicating that mid-span lateral acceleration peaks increase approximately linearly with wave height at the design speed. Under a train speed of 250 km/h and wave height of 5.72 m, the mid-span lateral acceleration shows the maximum amplification. For the 9.69 m wave height case, the mid-span lateral acceleration peak reaches 0.130 m/s^2.

5.2 Train Dynamic Response Under Combined Loading

To investigate train dynamic response under combined wave and train loading, the dynamic response of train cars traveling at 200 km/h and 250 km/h under different wave height conditions is calculated. The dynamic responses of the train subsystems are presented in Table 3 .

Under the same train speed, all dynamic indicators of the train increase with wave height. At 200 km/h, the derailment coefficient for powered cars increases from 0.62 to 0.72, while for trailers it increases from 0.54 to 0.6. At 250 km/h, the powered car derailment coefficient increases to 0.8. According to the High-Speed Railway Design Code (TB 10621—2014), the derailment coefficient and wheel load reduction rate should be less than 0.8 and 0.6, respectively. Clearly, the train derailment coefficients satisfy requirements in all cases. However, the wheel load reduction rate for powered cars exceeds the code limit at a wave height of 9.69 m and train speed of 250 km/h, which will affect operational safety.

According to the Railway Vehicle Dynamic Performance Evaluation and Test Specification (GB/T 5599—2019), the limits for train car body lateral and vertical accelerations are 1.0 and 1.3 m/s^2, respectively. Comfort evaluation indicators are 2.75 (good) and 3.0 (qualified). Train car body accelerations gradually increase with wave height, and under the same conditions, powered car accelerations are greater than trailer accelerations. Without wave action, train comfort is minimally affected by speed, with all cases rated as good or better. As wave height increases to 7.8 m, train comfort decreases. The increase in train speed significantly affects comfort, particularly at a wave height of 9.69 m and speed of 250 km/h, where vertical comfort indicators for both powered and trailer cars fail to meet the qualified standard.

Wave loading dominates the lateral displacement of the bridge. As wave height increases, train safety and comfort decrease, and all train dynamic indicators gradually increase. The dynamic indicators for powered cars increase more significantly than for trailers, especially when train speed exceeds the design speed with a wave height of 8.92 m, where the train exhibits greater dynamic response and is more prone to accidents. For sea-crossing bridges, bridge design should reasonably consider the magnitude of wave loads at the bridge site and control train speed appropriately to ensure operational safety under extreme conditions. The train car body vibration accelerations under wave loading are shown in Figure 11 [Figure 11: see original paper].

6. Conclusions

This study investigates an existing highway-railway sea-crossing cable-stayed bridge. Using Fluent software based on fifth-order Stokes wave theory and the Jonswap spectrum, a numerical wave flume is established to simulate wave loading. Combined simulation using Ansys and Simpack software creates a train-bridge coupling model to calculate the dynamic response of the train-bridge system under different train speeds and wave conditions. The following conclusions are obtained:

1. Based on numerical wave flume simulation, the total wave force at the tower-wave interface is calculated. The wave force on the bridge tower gradually increases with wave height and varies periodically with time. As wave height increases, the wave period becomes significantly larger.
2. As train speed and wave height increase, the dynamic response of the train-bridge coupling system gradually increases. Significant differences

exist between the lateral displacement curves under combined train-wave action and those under wave action alone.

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Note: Figure translations are in progress. See original paper for figures.