

STUDY ON DYNAMIC IMPACT FACTOR OF MULTI-ARCH RIB SPECIAL-SHAPED TIED ARCH BRIDGE

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ABSTRACT

This study examines the dynamic response characteristics and impact factor (IF) of a multi-rib, special-shaped tied-arch bridge under heavy truck loads. Field traffic tests were performed, and finite element models were validated to analyze the bridge's dynamic response to moving vehicles, obstacle crossings, and emergency braking. The results show no significant correlation between the speed of moving vehicles and the bridge's IF. However, under obstacle-crossing and braking conditions, the bridge's IF increased with speed, reaching 4.2 times and 3.1 times the standard design value, respectively. Additionally, as deck roughness worsens, the IF increases, with the arch ribs being most sensitive to these changes. The study also found that an increase in the number of vehicles, when spaced appropriately, can significantly reduce the bridge's IF. The lateral position of the vehicle has a minimal effect on the IF of the main girder and arch ribs, but a notable impact on the IF of the hangers.

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1. Introduction

As society progresses and bridge construction technology advances, the demand for bridges has shifted from a sole focus on transportation to a multifaceted need that incorporates aesthetics and cultural significance. Bridges are now expected not only to offer strong spanning capacity but also to serve as iconic symbols of cities—and even nations—through their aesthetic value. Engineers must therefore balance flexibility, aesthetics, and cultural symbolism with cost-effectiveness and safety.

In urban landscape bridge engineering, tied-arch bridges with unconventional shapes—featuring inclined hangers that distort the arch axis—are gaining popularity. This design enhances structural flexibility and visual impact, thereby improving the bridge's aesthetic value. Their unique geometry and complex stress patterns make predicting the dynamic response to traffic loads difficult, complicating design, assessment, and maintenance.

The dynamic characteristics of bridges, particularly the dynamic IF and ride comfort, are significantly influenced by vehicle weight, speed, and road surface quality [1–6]. Increasing traffic volumes and larger heavy-truck fleets have heightened the importance of dynamic loading [7–10]. Huang et al. [11] quantified the dynamic effects of special vehicles on short- and medium-span bridges, providing a benchmark for performance evaluation under such loads. Han et al. [12] used real-world weigh-in-motion data to assess the effects of heavy traffic on typical T-beam bridges, offering empirical evidence for load-rating practices. Zou et al. [13] introduced an interval-analysis method to predict critical bridge responses, surpassing traditional probabilistic methods when parameters are uncertain. Xie et al. [14] developed a high-fidelity vehicle-bridge interaction model to capture localized stress concentrations, aiding in fatigue assessment and maintenance planning.

While these studies provide a solid foundation for understanding vehicle-bridge interaction in conventional bridge types, the dynamic behavior becomes more complex in non-standard or geometrically intricate structures, such as multi-rib arch bridges. In these systems, the interaction between vehicle dynamics and structural response is influenced not only by the factors mentioned above but also by spatial configuration, rib interaction, and arch-specific vibration modes.

For non-standard arch systems, Li et al. [15] and Qin et al. [16] showed that current design codes may underestimate the dynamic properties of concrete-filled steel-tube arch bridges. Sun et al. [17] found strong coupling between vehicle speed, surface roughness, and dynamic amplification in spatial Y-shaped tied-arch bridges. Walton et al. [18] emphasized the importance of soil-structure interaction in buried arch bridges through hybrid numerical-experimental work. Li et al. [19] cautioned that conventional standards may under-predict impact effects in novel bridge designs, advocating for the adoption of advanced analysis tools.

Emerging traffic modes complicate the load environment further [20]. Lin et al. [21] showed that autonomous truck platoons can create dynamic load patterns not accounted for in existing specifications. He et al. [22] proposed a sensor-free bridge weigh-in-motion system, reducing both installation cost and complexity. Robert et al. [23] introduced the operating deflection shape ratio (ODSR), allowing real-time bridge deflection mapping from vehicle responses—an advancement toward continuous health monitoring.

Despite growing interest in aesthetically driven arch bridges, systematic data on their dynamic response to heavy trucks—especially the IF—remain scarce. Sun et al. [17] examined vehicle-bridge coupling in a Y-shaped tied-arch bridge, confirming speed and roughness as key parameters. However, their study did not address spatial multi-rib interaction or local component amplification. Lin et al. [21] focused on conventional girder bridges, leaving the question of how forked ribs and asymmetric hangers affect impact patterns unanswered. In conclusion, no field-validated, systematic study exists for multi-rib, special-shaped tied-arch bridges under heavy-truck loading.

To address this gap, we combine full-scale drive-by tests with high-fidelity finite element modeling of the Qinhuai Bay Bridge. This bridge features a 157 m main span and a three-rib structure, where rib A bifurcates into a Y-shape at mid-span. Additionally, the 53 steel-strand hangers vary in length and inclination. This prototype raises two unresolved questions: (i) Do forked ribs excite unanticipated modes? (ii) Do short hangers and fork nodes experience impact amplification beyond code limits?

Parallel field tests and finite element (FE) simulations are conducted under moving-vehicle, obstacle-crossing, and braking scenarios to systematically quantify the evolution of the IF as a function of speed, roughness, vehicle number, spacing, and lane position. The results provide direct evidence for the design, maintenance, and safety assessment of similar multi-rib, special-shaped tied-arch bridges.

2. Project overview and finite element model

2.1. Project overview

The Qinhuai Bay Bridge is a multi-arch rib, special-shaped tied-arch bridge that uses a steel box structure with a total span of 157 meters. Arch ribs B and C are aligned along the bridge's central line, while arch rib A splits at the crown, forming a distinct "Y"-shaped structure. The arch axis combines both curvature and straight segments. The bridge features 53 hangers, each consisting of multiple 7-mm-diameter high-strength steel strands with a nominal tensile strength of 1,670 MPa. For clarity, the hangers are numbered: B1 to B9 and A1 to A12 on the right side, and C1 to C11 in the central area. The overall layout of the bridge is shown in Fig. 1(a), with the typical sections of the main girder and arch ribs depicted in Figs. 1(b) and 1(c), respectively.

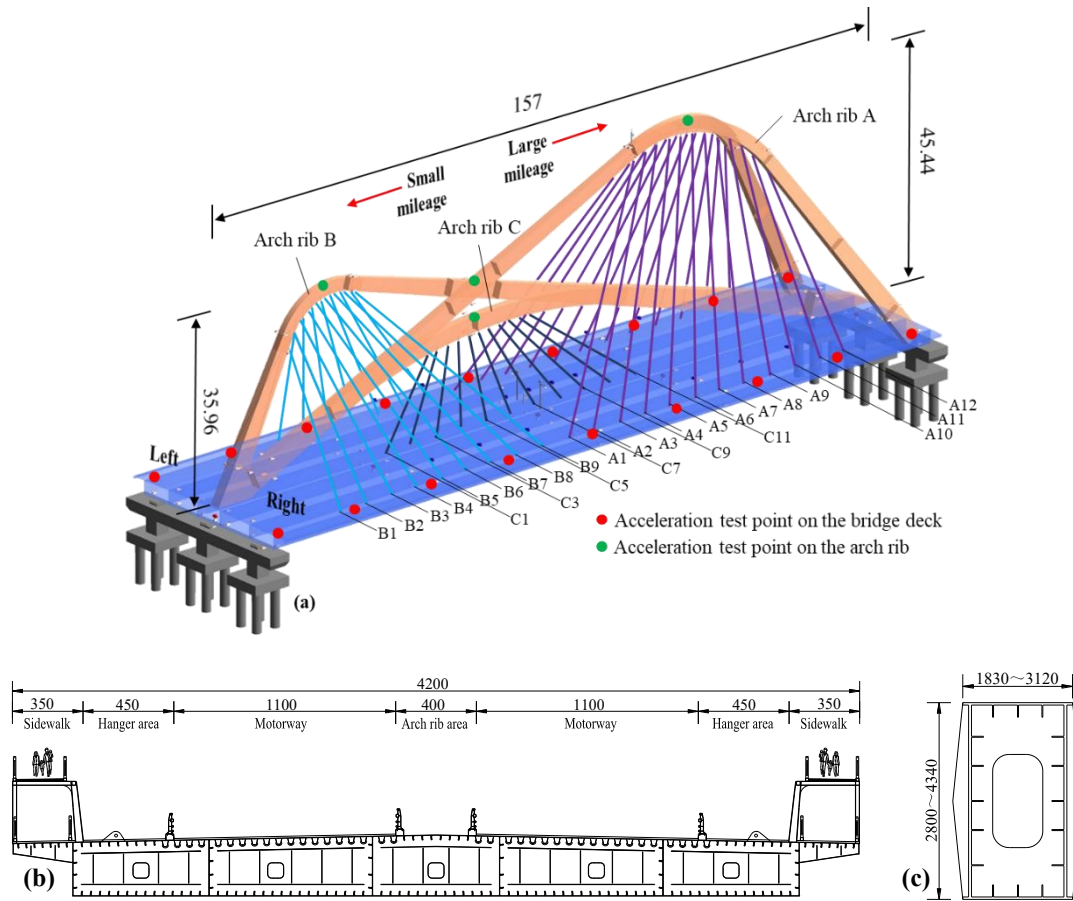


Fig. 1 Overview of Qinhuai Bay Bridge. (a) bridge component number and measuring point arrangement (unit: m), (b) typical main girder section (unit: cm), (c) typical arch rib section (unit: mm)

2.2. Bridge finite element model

Based on the stress characteristics and structural layout of each component of the Qinhuai Bay Bridge, guidelines for element selection and model simplification have been established. The main girder and arch rib are modeled using SHELL63 elements. The hangers are represented by LINK10 elements,

and hanger tension is simulated by applying initial strain. The bridge deck pavement is modeled as an equivalent mass and applied to the upper surface of the main girder using the MASS21 mass element. The ends of the hangers are connected to the arch rib and main girder through shared nodes. The finite element model of the Qinhuai Bay Bridge consists of 17,496 nodes and 29,397 elements, as shown in Fig. 2.

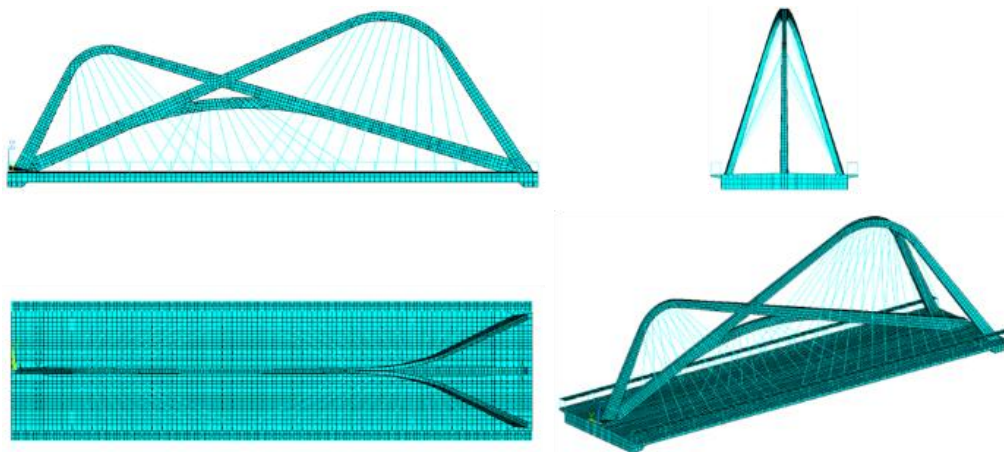


Fig. 2 Finite element model of Qinhuai Bay Bridge

2.3. Mesh sensitivity analysis

A mesh sensitivity study was conducted to verify the model's reliability and mesh adequacy. Three mesh densities—coarse, medium, and fine—were created by varying the element size, while keeping the element types and boundary conditions constant. The first-order natural frequency of the bridge and the mid-span deflection under self-weight were selected as key response quantities for assessing convergence.

The results of the mesh sensitivity analysis are shown in Table 1. As the mesh is refined from coarse to medium, noticeable changes occur in both the first natural frequency and mid-span deflection. However, the differences between the medium and fine meshes are minimal, indicating convergence. Specifically, the variation in the first natural frequency is less than 1%, and the change in mid-span deflection is within 2%. Notably, the medium-level mesh used in this analysis is the same as that in Section 2.2, confirming its adequacy for accurately and efficiently capturing the structural response.

Table 1
Mesh sensitivity analysis results

Mesh level	Total elements	First natural frequency (Hz)	Relative Change (%)	Mid-span deflection (mm)	Relative Change (%)
Coarse	15432	0.602	—	-101.7	—
Mediu	29397	0.615	2.2	-98.3	-3.3
Fine	45863	0.619	0.7	-96.7	-1.6

3. Bridge dynamic characteristics test

3.1. Natural vibration characteristic test

The results of the mesh sensitivity analysis are shown in Table 1. As the mesh is refined from coarse to medium, noticeable changes occur in both the first natural frequency and mid-span deflection. However, the differences between the medium and fine meshes are minimal, indicating convergence. Specifically, the variation in the first natural frequency is less than 1%, and the change in mid-span deflection is within 2%. Notably, the medium-level mesh used in this analysis is the same as that in Section 2.2, confirming its adequacy for accurately and efficiently capturing the structural response.

Table 2
Comparison of theoretical and measured natural frequencies of Qinhuai Bay Bridge

Rank	Natural frequency (Hz)		Deviation (%)	Measured damping ratio (%)	Characteristics of vibration mode
	Measured value	Theoretical value			
1	0.600	0.615	-2.5	0.275	First-order lateral bending of arch rib
2	1.475	1.507	-2.2	0.232	Second-order lateral bending of arch rib + first-order
3	1.525	1.551	-1.7	0.145	Third-order lateral bending of arch rib + second-
4	1.550	1.552	-0.1	0.390	First-order vertical bending of the main girder
5	1.850	1.823	1.5	0.283	Fourth-order lateral bending of arch rib + third-order
6	2.025	1.940	4.2	1.129	Fifth-order lateral bending of arch rib
7	2.100	2.096	0.2	0.840	second-order vertical bending of main girder

The natural vibration characteristics of the bridge are evaluated using the environmental random excitation method. During measurement, it is necessary to close the bridge deck to traffic. The natural vibration properties are determined by minor vibrations induced by irregular vibration sources near the bridge. These slight vibrations are mainly caused by stochastic forces such as wind pressure, ground vibrations, and water currents. Although the amplitude of the bridge’s vibrations from these sources is small, the frequency components are highly varied. This technique does not require excitation devices and is not constrained by the bridge’s structural configuration or dimensions, making it particularly suitable for assessing the natural vibration properties of the entire bridge.

During testing, data from environmental random excitations were collected using 941 B sensors, which were calibrated before the test to ensure accuracy. The sensor signals were captured by the INV3060 data acquisition system and analyzed using the DASP-V10 engineering version analysis software. The layout of the measurement points for evaluating the natural vibration characteristics of the Qinhuai Bay Bridge is shown in Fig. 1.

3.2. Bridge finite element model validation

The Block Lanczos method is used to analyze the modal properties of the bridge. A comparison of the first seven measured and theoretical modal frequencies for the Qinhuai Bay Bridge is shown in Table 2 and Fig. 3.

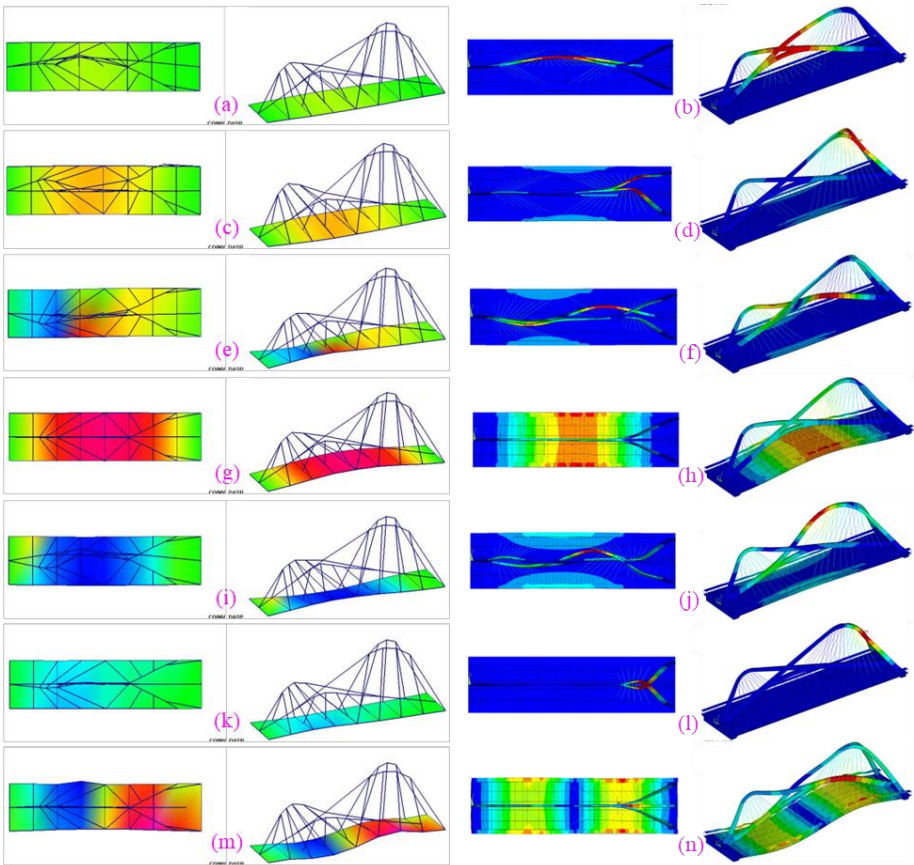


Fig. 3 Comparison of theoretical and measured vibration modes of Qinhuai Bay Bridge. (a), (c), (e), (g), (i), (k), (m) are the first 7 orders of measured vibration patterns of bridge, (b), (d), (f), (h), (j), (l), (n) are the first 7 orders of theoretical vibration patterns of the bridge

Table 2 and Fig. 3 show that the Qinhuai Bay Bridge exhibits a maximum deviation of only 4.2% between its theoretical and measured natural vibration frequencies. This indicates a high degree of agreement between the dynamic properties of the bridge's finite element model and the actual structure. The bridge primarily exhibits vibration modes of out-of-plane arch rib bending, in-plane main girder bending, and main girder torsional vibration. As the vibration mode order increases, the coupling between the arch rib's lateral bending and the main girder's torsional vibration becomes more pronounced. The fundamental frequency of the first-order lateral deflection of the arch rib is 0.6 Hz, while the first-order vertical deflection frequency of the main girder is 1.525 Hz. These results suggest that the bridge's arch rib has relatively low lateral stiffness.

Table 1 lists the measured modal damping ratios, which range from 0.145% to 1.129% and generally increase with mode order. This scatter is common in ambient-vibration tests for three reasons:

- (i) Different energy-dissipation mechanisms—such as material internal friction, joint slippage, and micro-aerodynamic drag—are activated to varying extents by each mode shape;
- (ii) Higher-order modes concentrate strain energy in local members (e.g., short hangers or the split-arch rib), creating additional energy release paths; and
- (iii) The signal-to-noise ratio of the output-only method decreases at higher frequencies, potentially inflating the identified damping.

In the FE model, Rayleigh damping ($C = \alpha M + \beta K$) was used. Coefficients α and β were calibrated based on the first two measured frequencies and their damping ratios, and then auto-extrapolated to higher modes. Table 2 shows that the theoretical frequencies deviate by less than 5% from the measured values, and the mode shapes align well, indicating that the damping strategy is appropriate for reproducing the global dynamic behavior.

3.3. Bridge IF test

The IF represents the dynamic influence of vehicular load on a bridge structure. As a vehicle crosses the bridge at a given velocity, a coupled vibration occurs between the vehicle and the bridge. This vibration significantly increases the bridge's deflection and stress, leading to potential damage and reduced serviceability. The amplification of the dynamic response due to bridge vibration is commonly represented by the IF (μ). The dynamic amplification factor (ϕ) refers to the ratio of the bridge's dynamic response to vehicle load relative to its static response under a static load. The IF (μ) is the difference between the dynamic amplification factor and the value of one. The IF is defined as follows:

$$\mu = \frac{\delta_d}{\delta_s} - 1 \quad (3.1)$$

where:

δ_d —corresponds to the peak dynamic response experienced by the bridge due to traffic loading; δ_s —indicates the maximum static response of the bridge when subjected to static loading from the vehicle.

The Chinese standard [24] stipulates that the IF is a function associated with the fundamental frequency of the structure, and the specific formulation is as follows:

$$\mu = \begin{cases} 0.05 & f < 1.5\text{Hz} \\ 0.1767\ln f - 0.0157 & 1.5\text{Hz} \leq f \leq 14\text{Hz} \\ 0.45 & f > 14\text{Hz} \end{cases} \quad (3.2)$$

where:

f —fundamental frequency of the bridge, unit is Hz.

The bridge's IF depends on multiple parameters. In addition to the inherent natural vibration properties of the structure, it is influenced by factors such as bridge type, vehicle load, vehicle speed, and vehicle position. As a result, precisely determining the dynamic IF of a bridge is challenging. The Chinese specification "Highway Bridge Load Test Specification" [25] outlines a method for determining the IF of a bridge through load testing. This method involves deploying one or more vehicles to conduct driving tests on the bridge and capture its dynamic response to moving vehicle loads.

To investigate the impact effects of heavy trucks on the Qinhuai Bay Bridge, several experimental studies were conducted, including moving vehicle tests, obstacle-crossing tests, and braking tests. These tests were designed to replicate the vehicle operating conditions that occur during actual bridge use.

(1) Test method and working condition

Moving vehicle test: The vehicle runs at 10 km/h, 20 km/h, 30 km/h, 40 km/h and 50 km/h in the right lane.

Obstacle-crossing test: A 7 cm-high curved barrier (deceleration area) is situated in the center of the right portion of the bridge to simulate bridge deck potholes. The vehicle crosses the curved barrier at velocities of 5 km/h, 10 km/h, 15 km/h, and 20 km/h.

Braking test: The vehicle travels at 30 km/h, 40 km/h, and 50 km/h in the right lane, applying brakes upon reaching the midspan.

(2) Test equipment and measuring point arrangement

The test vehicle is two 36 t four-axle heavy trucks, as shown in Fig. 4. The HPQN-X multi-point video detection system, designed for dynamic and static analysis, is utilized to acquire the dynamic deflection data of the main girder's span and determine the IF. The corresponding test image is presented in Fig. 5.

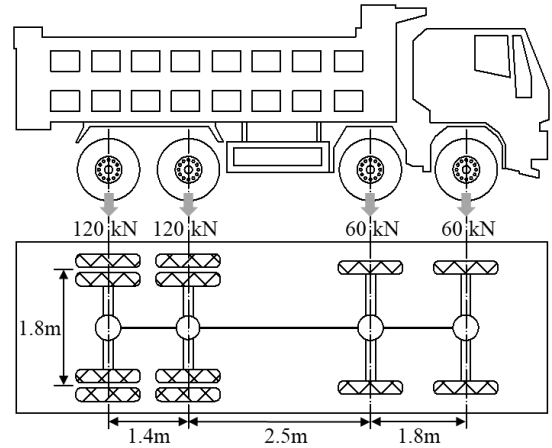


Fig. 4 Test vehicle wheelbase and axle load

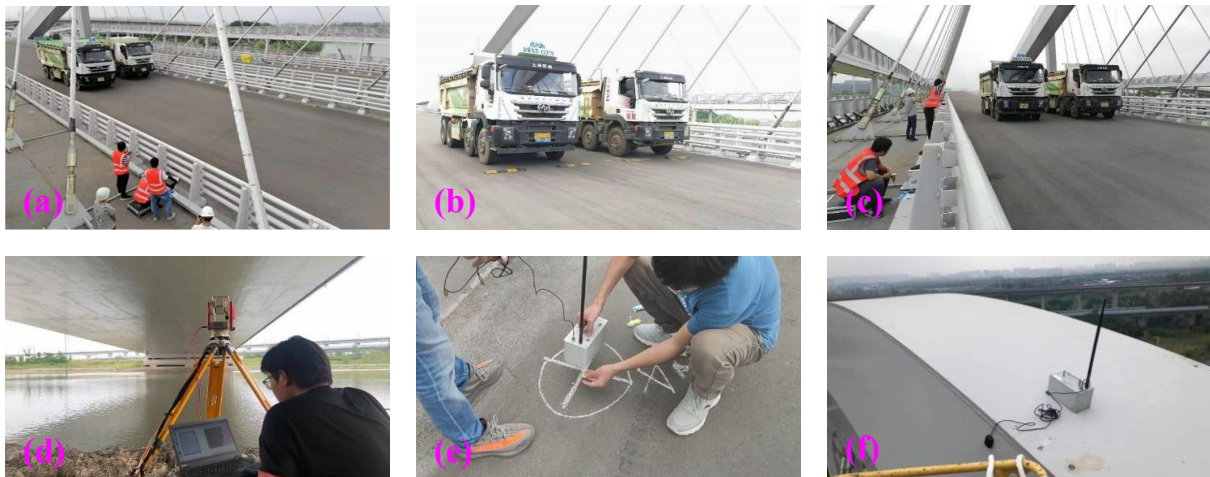


Fig. 5 Modal and IF test of Qinhuai Bay Bridge. (a) moving vehicle test, (b) obstacle-crossing test, (c) braking test, (d) mid-span dynamic deflection monitoring, (e) acceleration test point on the bridge deck, (f) acceleration test point on the arch rib

3.4. Analysis of test results

Fig. 6 shows the dynamic deflection time history curves and displacement IF of the main girder span under various driving conditions. Subfigures (a)–(e) represent moving vehicles at different speeds, (f)–(i) show obstacle-crossing

scenarios, and (j)–(l) display braking conditions. Each subfigure highlights the dynamic response characteristics under the corresponding test condition. The IF value is the mean of five independent runs ($n = 5$), with ± 1 SD and 95% CI provided below.

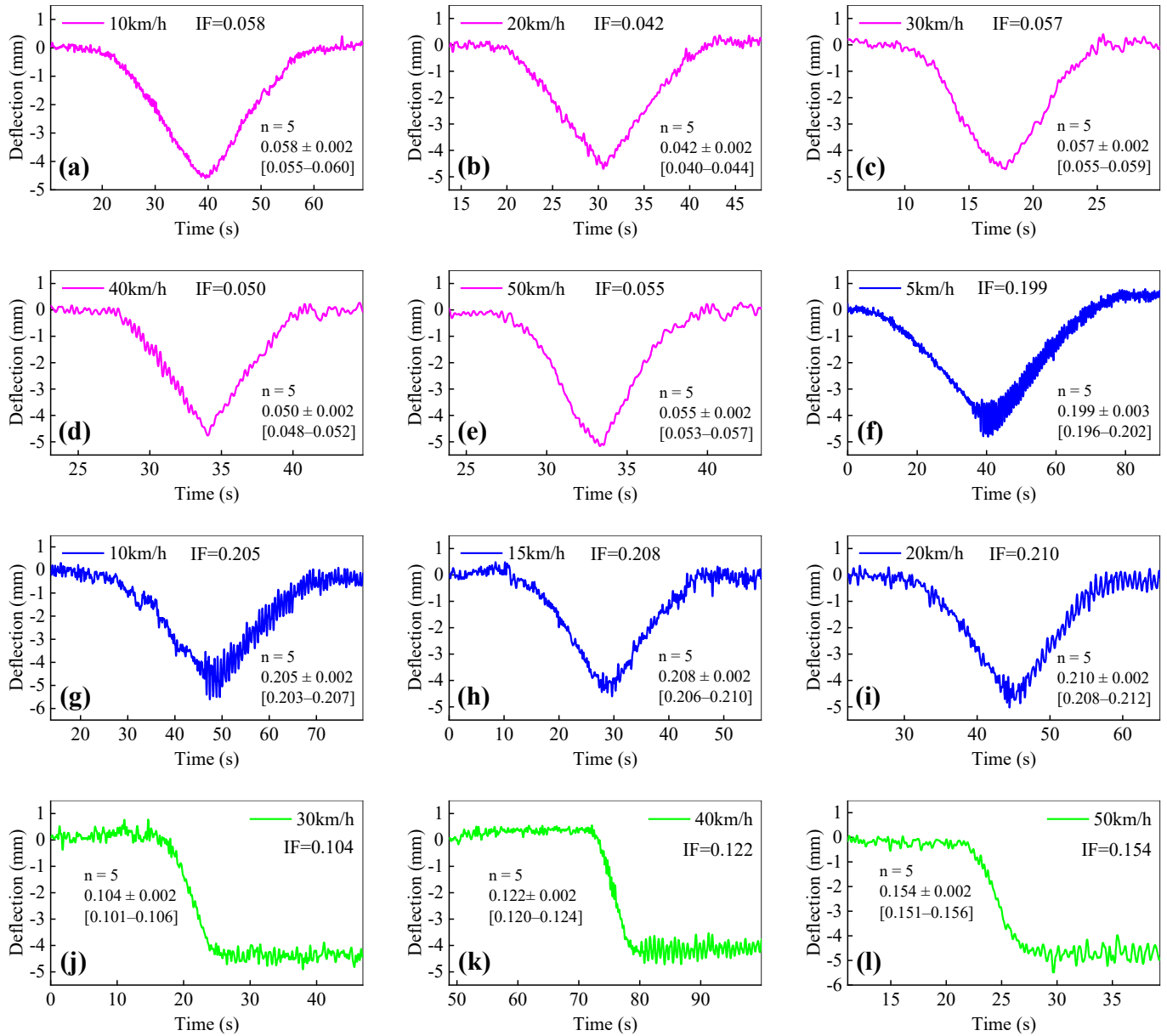


Fig. 6 Dynamic deflection and IFs of mid-span measuring points of main girder under different working conditions. (a) ~ (e) moving vehicle conditions, (f) ~ (i) obstacle-crossing conditions, (j) ~ (l) braking conditions

The IF of the Qinhuai Bay Bridge is shown in Fig. 6 (a) to (e), with values ranging from 0.042 to 0.058 under free-flow driving conditions. The average IF is slightly higher than the standard design value of 0.05. These results indicate no significant correlation between free-flow vehicle speed and the bridge's IF. Although an increase in vehicle speed causes greater vehicle vibration and higher excitation forces, the contact duration between the vehicle and the bridge is reduced. Shorter contact times limit the energy transferred to structural vibrations. As a result, the impact effect does not increase with vehicle speed.

Furthermore, Fig. 6 (f)–(i) shows the IF of the Qinhuai Bay Bridge under obstacle-crossing conditions, ranging from 0.199 to 0.210, with the maximum value being 4.2 times the standard design value. As the initial speed increases, the IF also rises. This trend is attributed to the increase in vehicle kinetic energy at higher speeds. Upon hitting the obstacle, the vehicle's surplus kinetic energy is converted into an additional impact load on the bridge. This load may cause damage to the bridge structure, particularly in the case of multiple repeated impacts, which could accelerate the aging and degradation of the bridge. More importantly, elevated IFs under obstacle conditions suggest that even localized deck defects—such as potholes or uneven expansion joints—can induce dynamic amplifications far exceeding design expectations. This underscores the

need for proactive maintenance strategies, including regular monitoring of deck surface conditions and timely repair of surface irregularities, to prevent structural overloading and fatigue accumulation in critical components, such as arch ribs and short hangers.

Ultimately, Fig. 6 (j) ~ (l) illustrates the IF of the Qinhuai Bay Bridge under braking conditions. The IF ranges from 0.104 to 0.154, with the peak value being 3.1 times the standard design value. Consistently, as the initial velocity rises, the IF also exhibits an upward trend. This can be attributed to the fact that higher vehicle speeds result in greater kinetic energy, necessitating stronger braking forces during emergency braking and consequently generating higher IFs. These findings suggest that emergency braking events, particularly at high speeds, may subject the bridge to transient dynamic loads significantly beyond codified values. Operationally, it underscores the need for speed-control measures on long-span or geometrically complex bridges—particularly multi-rib special-shaped tied arches. Moreover, current design codes that rely solely on structural frequency to determine IFs may not adequately capture such extreme dynamic responses. Therefore, incorporating scenario-specific dynamic amplification factors into future design guidelines should be considered to enhance structural robustness and safety margins.

In summary, the IFs of the Qinhui Bay Bridge under various driving conditions show significant variation, with average values exceeding the standard design values. These findings are crucial for understanding the dynamic response of special-shaped tied-arch bridges under real-world conditions and provide valuable data for the design and maintenance of similar bridges.

4. Numerical analysis

4.1. Vehicle-bridge coupling vibration analysis method

4.1.1. Vehicle model and its vibration equation

A spatial dynamic analysis framework for a four-axle truck is presented in

Fig. 7. The truck is divided into multiple rigid components, with the wheels and frame modeled as mass points. These mass points are connected by suspension springs and dampers. The mass points for each wheel are supported on the bridge deck by tires that exhibit spring-damping properties. In this study, during the vehicle model simplification, only the three degrees of freedom (DOFs) associated with the frame's vertical movement, pitch, and roll, as well as the vertical DOF for each wheel, are considered. This simplification is justified by the primary objective of capturing the bridge's vertical vibration characteristics under vehicular loading. The retained DOFs directly control the vertical forces exerted on the deck. Additionally, the numerical scenarios assume constant-speed crossing without acceleration or braking, so longitudinal forces—and thus the longitudinal DOF—have negligible influence on the vertical response. The vehicle parameters are specified in Table 3.

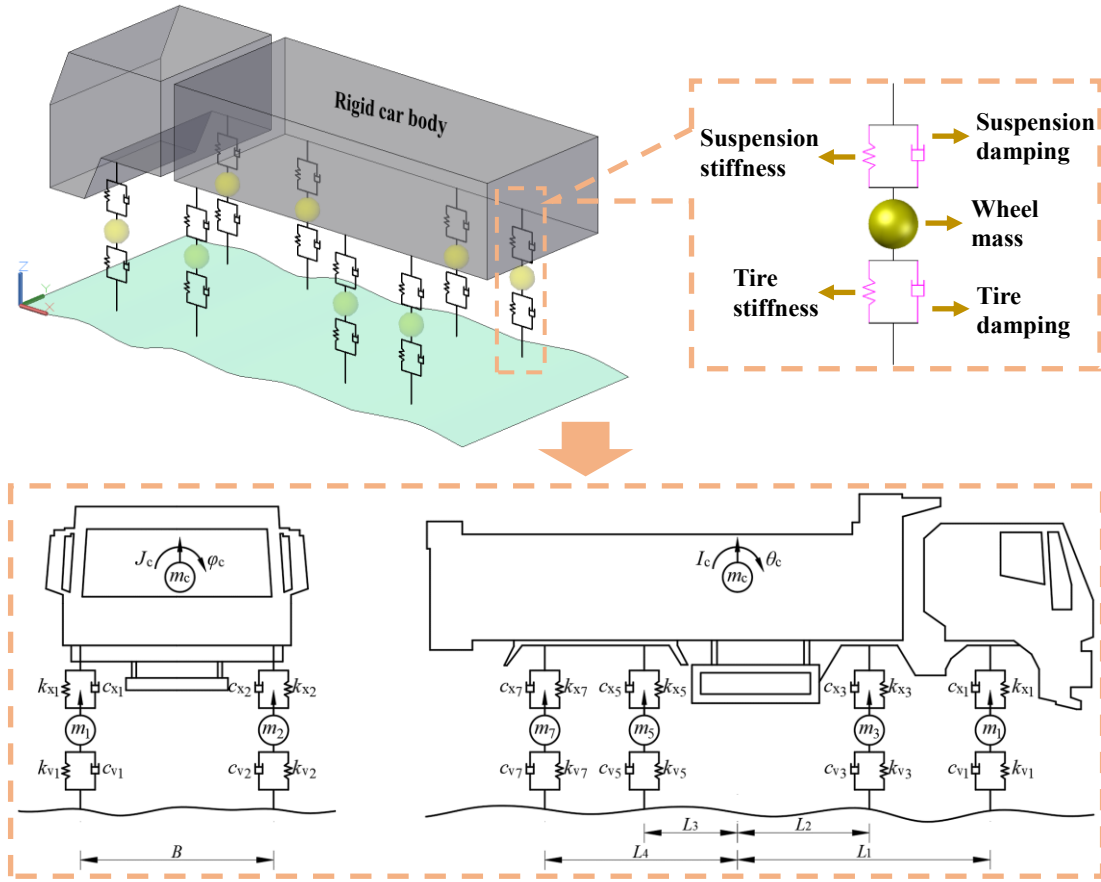


Fig. 7 Four-axle truck model

Table 3

Vehicle parameters

Parameters	Numerical value	Unit	Parameters	Numerical value	Unit
mc	32948	kg	kx1-kx2、kx7-kx8	3146	kN/m
Ic	40000	kg·m ²	kx3-kx6	2362	kN/m
Jc	10000	kg·m ²	cx1-cx2	13.3	kN·s/m
m1-m4	297	kg	cx3-cx8	5	kN·s/m
m5-m8	466	kg	kv1-kv2、kv7-kv8	1577	kN/m
L1	3.36	m	kv3-kv6	2362	kN/m
L2	1.56	m	cv1-cv2、cv7-cv8	11.2	kN·s/m
L3	1.14	m	cv3-cv6	16.7	kN·s/m
L4	2.54	m	B	1.8	m

The vibration equation for the four-axle vehicle model has been derived by applying the D'Alembert principle [26].

$$M_t \ddot{Z} + C_t \dot{Z} + K_t Z = F_G + F_t \quad (4.1)$$

where:

M_t 、 C_t 、 K_t —the mass matrix, damping matrix and stiffness matrix of the vehicle system; Z —vehicle degree of freedom column vector; F_G —gravity vector of vehicle; F_t —wheel-road contact force vector acting on the vehicle.

4.1.2. Bridge vibration equation

In line with the finite element theory, the equation governing bridge vibration is [26].

$$M_b \ddot{Y} + C_b \dot{Y} + K_b Y = F_b \quad (4.2)$$

In this context: M_b , C_b , and K_b represent the mass, damping, and stiffness matrices of the bridge structure, respectively. Y , $\dot{Y}$, and $\ddot{Y}$ denote the displacement, velocity, and acceleration vectors of the element nodes. F_b signifies the vector of contact forces between the wheels and the road acting on the bridge.

The damping ratio of bridge structures is typically determined through testing. In the absence of measured data, the values recommended in the specification [27] can be used. A damping ratio of 5% is recommended for concrete structures, while 3% is suggested for steel structures. The damping ratio for the bridge in this study has been determined and is shown in Table 2.

4.1.3. Vehicle-bridge coupling relationship

While the vehicle crosses the bridge, it is assumed that the wheel remains in steady contact with the bridge surface, avoiding any bouncing. The displacement of the wheel at any given moment is reflected by the displacement of the bridge, indicating a coordinated displacement relationship. The vehicle-bridge interaction vibration equation can then be formulated by considering the interaction force between the vehicle and bridge components at the contact interface [26].

$$\begin{bmatrix} M_b \\ M_t \end{bmatrix} \begin{Bmatrix} \ddot{Y} \\ \ddot{Z} \end{Bmatrix} + \begin{bmatrix} C_b + C_{bb} & C_{bt} \\ C_{tb} & C_t \end{bmatrix} \begin{Bmatrix} \dot{Y} \\ \dot{Z} \end{Bmatrix} + \begin{bmatrix} K_b + K_{bb} & K_{bt} \\ K_{tb} & K_t \end{bmatrix} \begin{Bmatrix} Y \\ Z \end{Bmatrix} = \begin{Bmatrix} F_{br} \\ F_G + F_{tr} \end{Bmatrix} \quad (4.3)$$

where:

C_{bb} , C_{bt} , C_{tb} , K_{bb} , K_{bt} , K_{tb} , F_{br} , F_{tr} —the time-dependent elements associated with the wheel-road contact force will vary as the vehicle travels along the bridge.

4.1.4. Bridge deck roughness simulation

Bridge deck roughness is typically evaluated using a frequency-based approach characterized by the power spectral density function [28].

$$G_q(n) = G_q(n_0) \left(\frac{n}{n_0} \right)^{-w} \quad (4.4)$$

where:

n_0 —the spatial reference frequency equals 0.1 m^{-1} ; $G_q(n_0)$ —coefficient of road surface roughness (m^2/m^{-1}); w —frequency index, commonly set at 2; n —the

spatial frequency, measured in inverse meters (m^{-1}), corresponds to the reciprocal of the wavelength.

The triangular series method, combined with the power spectral density function for bridge deck roughness, generates samples representing deck irregularities. The equation is as follows:

$$r(x) = \sum_{j=1}^m G(n_j) \cos(2\pi x n_j + \theta_j) \quad (4.5)$$

$$G(n_j)^2 = 4G_q(n_j)\Delta n \quad (4.6)$$

$$n_j = n_d + (j-1/2)\Delta n \quad (j=1, 2, 3, \dots, m) \quad (4.7)$$

$$\Delta n = (n_u - n_d) / m \quad (4.8)$$

where:

x —longitudinal bridge displacement; m —the number of segments of spatial frequency division; $G(n_j)$ —vibration amplitude; n_u , n_d —represent the upper and lower bounds of effective spatial frequency; θ_j —random variables uniformly distributed on $[0, 2\pi]$.

In this study, the computational parameters for the bridge deck roughness specimen are as follows: $n_d=0.01 \text{ m}^{-1}$, $n_u=5 \text{ m}^{-1}$, $n_0=0.1 \text{ m}^{-1}$. The value of $G_q(n_0)$ is determined based on the roughness grade and geometric mean specified in the "Method for Representing Road Surface Flatness for Vehicle Vibration Input" (GB/T 7031-2005) [29]. Fig. 8 shows the simulated roughness profile.

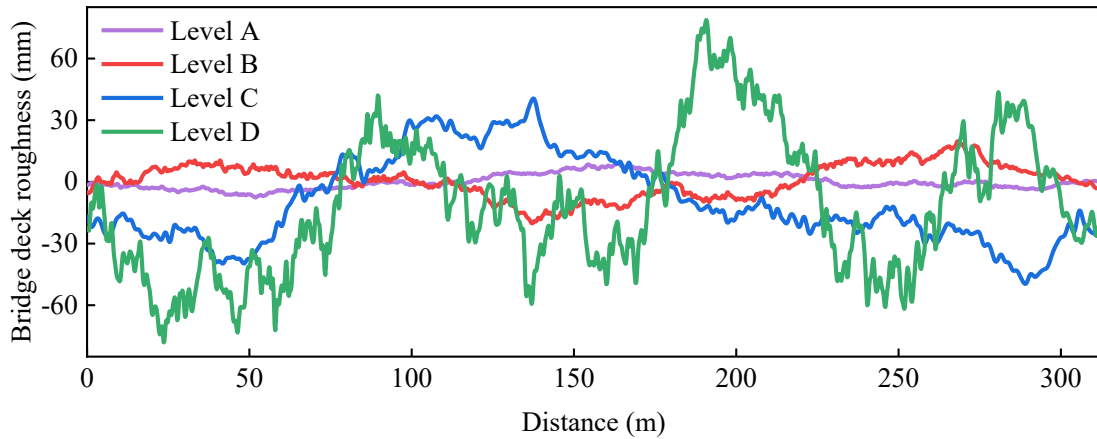


Fig. 8 A ~ D level bridge deck roughness

4.2. Analysis of dynamic behavior characteristics of bridges under the action of heavy trucks

Heavy trucks typically travel in the outermost lane of the bridge, reflecting the bridge's actual operational conditions. To examine the dynamic behavior of the Qinhuai Bay Bridge under the influence of heavy trucks, a numerical simulation method is used to model the vehicle's movement across the bridge. The bridge deck roughness is assumed to be Grade A, the vehicle speed is set at 50 km/h, and the heavy truck operates in the outermost lane on the right side of the bridge. The bridge's dynamic response is shown in Fig. 9.

Fig. 9 (a) shows that the peak dynamic deflection response of the primary beam occurs at the midspan location. As shown in Fig. 9 (b), the maximum dynamic deflection at the midspan of the primary beam is 4.4 mm on the sidewalk adjacent to the truck, while the minimum dynamic deflection is 0.3 mm on the opposite sidewalk. Therefore, in subsequent research, the deflection response on the right sidewalk of the main girder will be used to represent the overall IF of the bridge.

The Qinhuai Bay Bridge features a multi-arch rib structure. For clarity, the arch ribs are designated as Arch Rib A, Arch Rib B, and Arch Rib C. As shown in Fig. 9 (c), there is significant variation in the vertical stiffness among the arch ribs. The maximum dynamic deflection of Arch Rib A is 0.99 mm, while Arch

Rib B and Arch Rib C have maximum dynamic deflections of 0.75 mm and 0.94 mm, respectively. It is evident that Arch Rib B exhibits the highest vertical stiffness, followed by Arch Rib C, with Arch Rib A having the lowest stiffness. The peak dynamic deflection of Arch Rib A in the longitudinal direction is 94.4 mm, making it the focus of further examination.

The distribution of the peak axial force in the hangers is shown in Fig. 9 (d). Typically, the axial force in the hangers along each arch rib is highest at the center and decreases towards the ends. The axial force in hangers at different positions varies significantly. The maximum axial force in hanger C3 reaches 32.3 kN, which is 1.9 times higher than that of A7 (17.1 kN) and 2.2 times higher than that of B5 (14.5 kN). This is due to the unique arrangement of the hangers on the bridge. The hanger on arch rib C is positioned at the middle of the span, where the bridge's dynamic response is more pronounced. This is also related to the relative length of the hanger. The lengths of hangers A7, B5, and C3 are 46.5 m, 38.4 m, and 17.6 m, respectively. The shorter hanger C3 exhibits significant rigidity and a high natural frequency, making the impact effect induced by moving vehicles more pronounced. Therefore, during the operation and maintenance of the bridge, special attention should be given to the hanger at Arch Rib C, with hanger C3 selected as the representative element for further analysis.

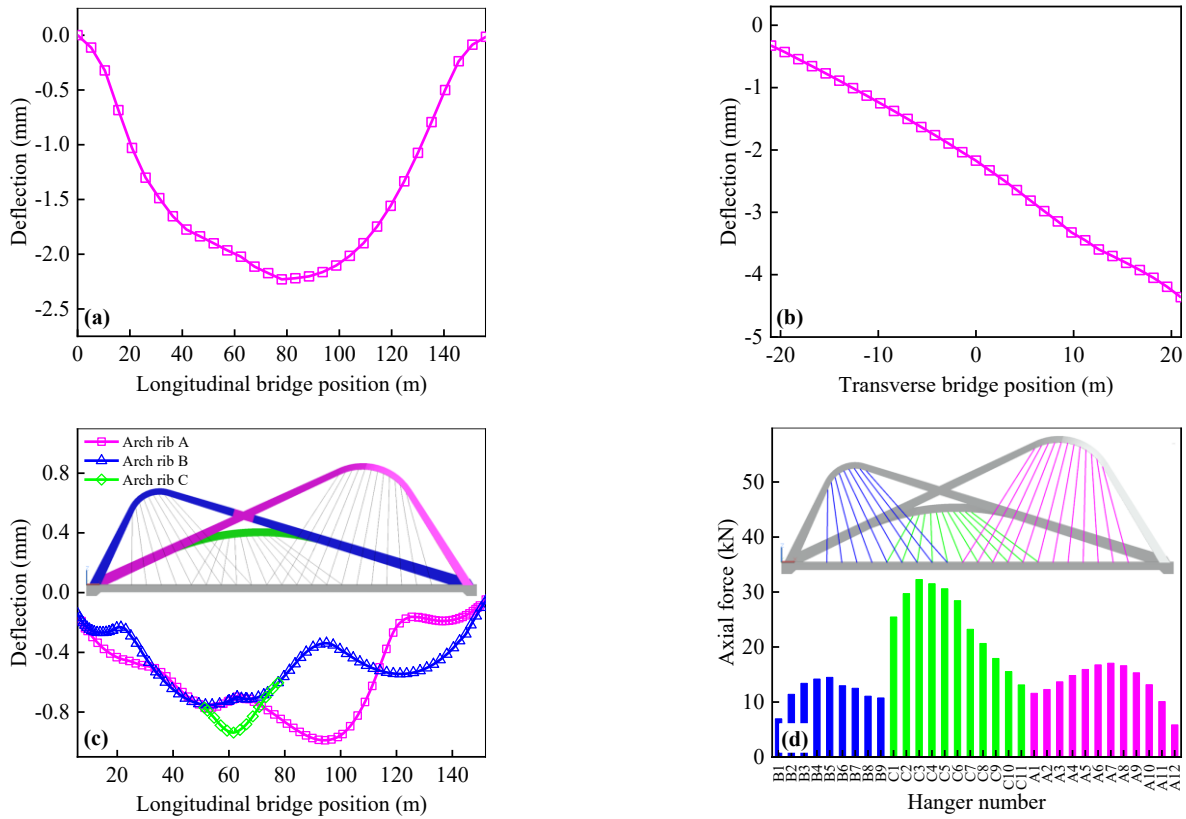


Fig. 9 Dynamic response of bridge. (a) longitudinal distribution of deflection extreme value of main girder, (b) transverse distribution of deflection extreme value of mid-span of main girder, (c) longitudinal distribution of deflection extreme value of arch rib, (d) axial force extreme value of hangers

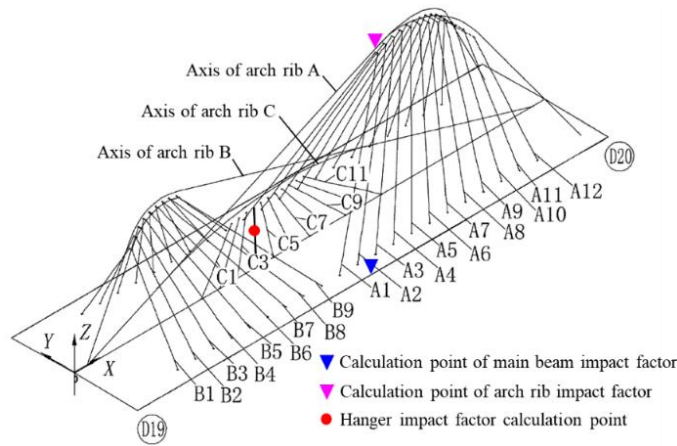


Fig. 10 IF calculation point

4.3. Bridge dynamic IF analysis

The general specification for the design of highway bridges and culverts

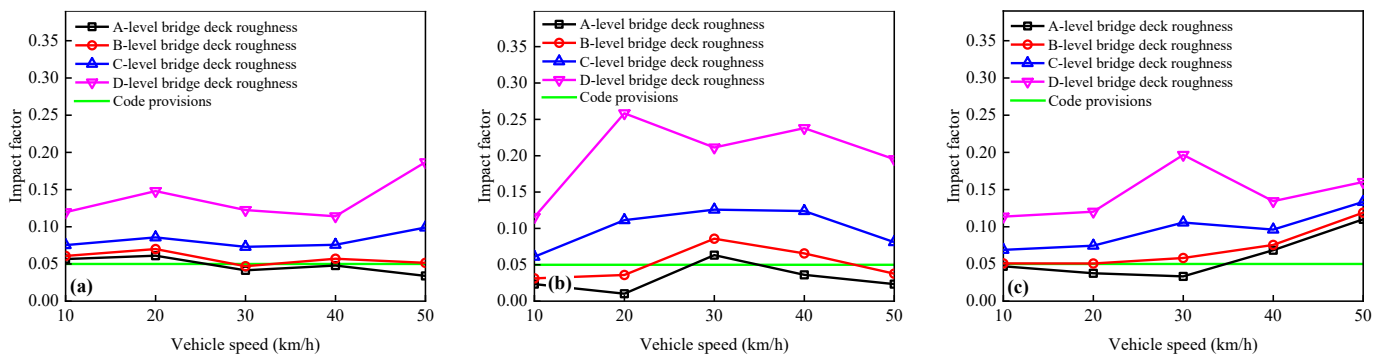


Fig. 11 Bridge IFs. (a) displacement IFs of main girder feature point, (b) displacement IFs of arch rib feature point, (c) axial force IFs of hanger C3

(JTG D60-2015) [24] defines the impact factor (IF) as a function of the structure's fundamental frequency. However, test results show that the IF of the bridge is influenced by several factors. Therefore, this section uses numerical methods to investigate the effects of vehicle speed, bridge deck roughness, the number of vehicles, spacing, location, and other factors on the IF of multi-arch rib special-shaped tied-arch bridges. Based on the calculation results from Section 4.2, the IF calculation points are selected. Measurement points on the main girder are used to determine the overall IF of the bridge, while measurement points on the arch ribs and hangers are employed to calculate the local IF, as shown in Fig. 10.

4.3.1. Effect of vehicle speed and bridge deck roughness

The deck surfacing of the bridge is a critical component directly exposed to vehicular loads and environmental factors. Over time, repeated loading gradually reduces the smoothness of the deck. This degradation not only reduces the bridge's efficiency but may also amplify the impact loads experienced as vehicles pass. To examine the effects of vehicle speed and bridge deck roughness on the impact response of key components of the Qinhuai Bay Bridge, vehicle speeds are set between 10 and 50 km/h. The bridge deck roughness is categorized into four levels, labeled A to D. Focusing on a single vehicle traveling in the outermost right lane, the displacement IFs at the mid-span sidewalk of the main girder, at the critical section of arch rib A, and the axial force IF of hanger C3 are calculated. The results are shown in Fig. 11.

As shown in Fig. 11, there is no significant correlation between the bridge's IF and vehicle speed, consistent with previous test results. When the bridge deck roughness is in good condition, the IF of the main girder is approximately 0.05, closely matching the value used in standard design. This suggests that under ideal conditions, the bridge's design aligns with its actual performance.

A thorough investigation into the impact of bridge-deck roughness on the IF reveals a clear trend: the IF of all critical components increases as the deck condition deteriorates, with the arch rib exhibiting the most significant increase—much larger than that of the main girder or hangers. This behavior can be explained from a system perspective: (i) the curved arch rib has much lower out-of-plane stiffness than the in-plane flexural stiffness of the main girder, making it more sensitive to wheel-induced excitation; (ii) impact forces from deck irregularities are transmitted to the arch rib through the hangers, and because the arch rib has low stiffness and damping, these irregularities are amplified; (iii) the main girder, with its continuously distributed mass and higher vertical stiffness, acts as a low-pass filter for local roughness, resulting in a relatively modest increase in the IF.

When the bridge deck roughness reaches level D, the peak IF of the arch rib may be up to five times greater than the standard design value. This suggests that, in extreme cases, the impact on the arch ribs could significantly exceed design expectations, posing a serious challenge to the bridge's safety and durability. Given the sensitivity of the arch rib IF to deck roughness and its importance within the bridge structure, it is more appropriate to adopt it as the representative IF value for the Qinhuai Bay Bridge.

Field tests and numerical simulations show that, under realistic events such

as obstacle crossing or emergency braking, the IFs of multi-rib special-shaped tied-arch bridges can significantly exceed the values prescribed in China's current JTG D60-2015 code, which bases IF solely on the fundamental frequency. For example, with deck roughness at Level D and a vehicle speed of 50 km/h, the arch-rib IF can reach five times the code limit. These results demonstrate that a frequency-only approach fails to capture the dynamic amplification experienced by complex bridges in actual traffic. Therefore, it is recommended that future revisions of JTG D60-2015 incorporate multi-parameter or scenario-specific IFs, explicitly accounting for bridge geometry, deck condition, and vehicle actions (e.g., braking and obstacle negotiation), and introduce local IF amplifications for critical components such as short hangers and forked arch ribs to improve robustness and safety margins.

4.3.2. Effect of number of vehicles

In real-world road transportation, heavy trucks often travel in convoy formations. This chapter examines the dynamic behavior of a bridge under various driving scenarios, including single, double, and triple vehicle formations, as shown in Fig. 12. These scenarios aim to replicate driving patterns observed in actual traffic, with the objective of assessing how different traffic densities affect the dynamic response of the bridge structure. By analyzing the IFs in critical components such as main girders, arch ribs, and hangers, we can better understand how different vehicle configurations influence bridge performance, providing a scientific basis for bridge design. The effect of varying vehicle numbers on the bridge's IFs is shown in Fig. 13.

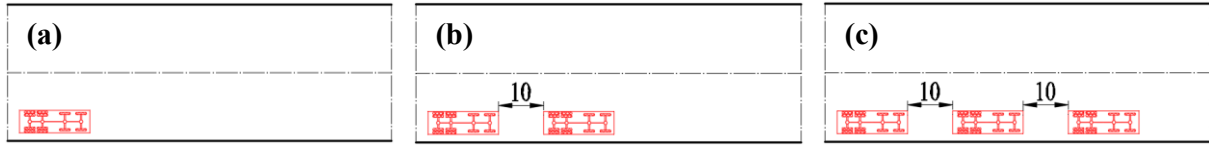


Fig. 12 Longitudinal vehicle layout (unit: m). (a) single truck, (b) two trucks, (c) three trucks

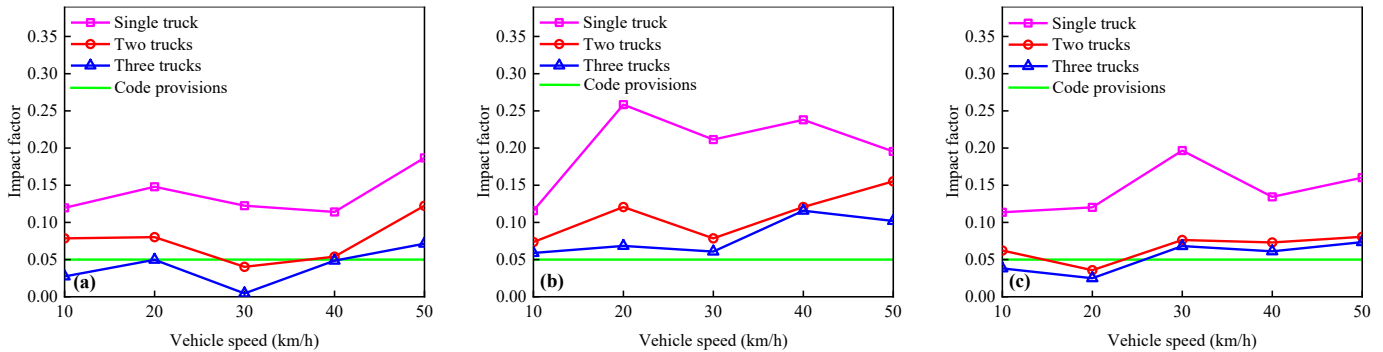


Fig. 13 Bridge IFs under different driving numbers. (a) displacement IFs of main girder feature point, (b) displacement IFs of arch rib feature point, (c) axial force IFs of hanger C3

Fig. 12 shows that as the number of vehicles in the fleet increases, the IFs at each feature point of the bridge exhibit a marked decline. This suggests that the number of vehicles in the fleet significantly affects the bridge's IF. The reason is that an increase in the number of vehicles leads to a more even distribution of load across the bridge, reducing severe vibrations caused by concentrated loads and, consequently, lowering the bridge's IF.

Even on a Class-D deck, the main girder's IF meets code limits when three vehicles are spaced appropriately. Additionally, comparing the IFs at different feature points shows that the arch rib generally has a higher IF than the main girder and hangers. This finding is consistent with the analysis in Section 4.3.1. As a critical component of the bridge structure, the arch rib's high IF may be linked to its load characteristics and structural form. When the arch rib is subjected to load, its unique geometric shape and force distribution may result in larger amplitudes, leading to a higher IF.

In conclusion, increasing the number of vehicles in the fleet load helps reduce the bridge's IF. The differences in the IFs at various feature points highlight the complexity of the bridge structure under load. These insights provide valuable guidance for bridge design and maintenance, helping engineers optimize bridge structures and improve their durability and safety.

4.3.3. Effect of driving distance

The interaction between vehicles and the vibration coupling of bridge components are key factors influencing the bridge's IF. In real traffic scenarios, the distance between vehicles fluctuates due to factors such as traffic density,

driving behavior, and road conditions. Vehicle spacing is crucial for both bridge safety and traffic flow efficiency. Excessively small spacing can pose safety risks, while overly large spacing can reduce traffic efficiency. Therefore, investigating the bridge's IF under various vehicle distances helps identify the optimal balance between safety and efficiency. To assess the effects of different driving intervals on the dynamic behavior of the Qinhuai Bay Bridge, three cases were established with intervals of 10 m, 20 m, and 30 m, as shown in Fig. 14. The platoon speed was set at 50 km/h, and the bridge deck roughness was classified as Level D. The IFs for the bridge's key components under these conditions are shown in Fig. 15.

Fig. 15 shows that as the inter-vehicle distance increases, the displacement response of the main girder and arch rib, as well as the axial force response of the hanger, tends to decrease. This suggests that greater vehicle spacing reduces the impact of vehicles on the bridge structure. For example, as the gap between vehicles increases from 10 meters to 30 meters, the dynamic response of the main girder decreases by 29.1%. The dynamic response of the arch rib drops by 32.9%, with the most significant reduction observed in hanger C3, which experiences a 41.6% decrease. Wider spacing reduces dynamic load, stress, and deformation.

Additionally, the IF of the bridge decreases as vehicle spacing increases. Specifically, when the vehicle spacing reaches 30 meters, the IFs of the girder, arch rib, and hanger C3 are reduced by 35.2%, 18.6%, and 39.7%, respectively, compared to a 10-meter spacing. The reduction in IF due to increased vehicle spacing positively impacts the long-term stability and safety of the bridge.

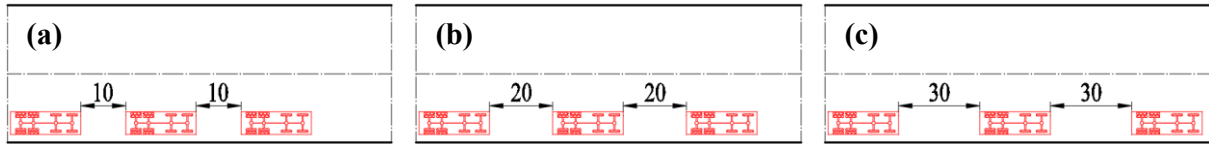


Fig. 14 Vehicle driving distance diagram (unit: m). (a) 10m spacing, (b) 20m spacing, (c) 30m spacing

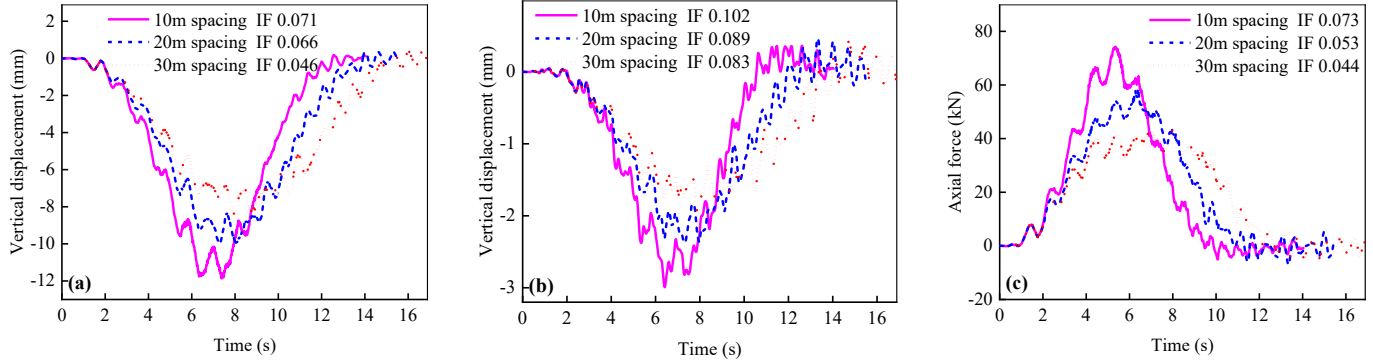


Fig. 15 Bridge IFs under different driving distances. (a) deflection time history curve of main girder feature point, (b) deflection time history curve of arch rib feature point, (c) axial force time history curve of hanger C3

4.3.4. Effect of driving position

When evaluating the IF of a bridge, the lateral positioning of heavy trucks must be considered due to its influence on the bridge structure. The load distribution and its effects on the bridge vary depending on the vehicle's position on the bridge. For example, the Qinhui Bay Bridge has a main girder width of 42 meters and is a six-lane, two-way road. To simulate and analyze how changes in vehicle load and lateral position affect the bridge's IF, three scenarios were

established: a single truck traveling in the right inner lane, the middle lane, and the outer lane. These scenarios, shown in Fig. 16, represent potential vehicle travel paths in real traffic conditions and are crucial for assessing the bridge's safety and durability. It was assumed that the vehicle traveled at a speed of 50 km/h, with the bridge deck roughness classified as Level D. Fig. 17 shows the IFs of the bridge's critical components under each scenario.

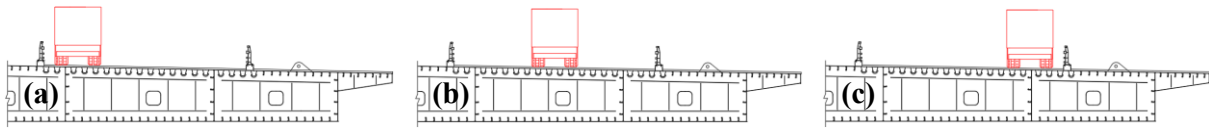


Fig. 16 Vehicle driving position diagram. (a) lane 1, (b) lane 2, (c) lane 3

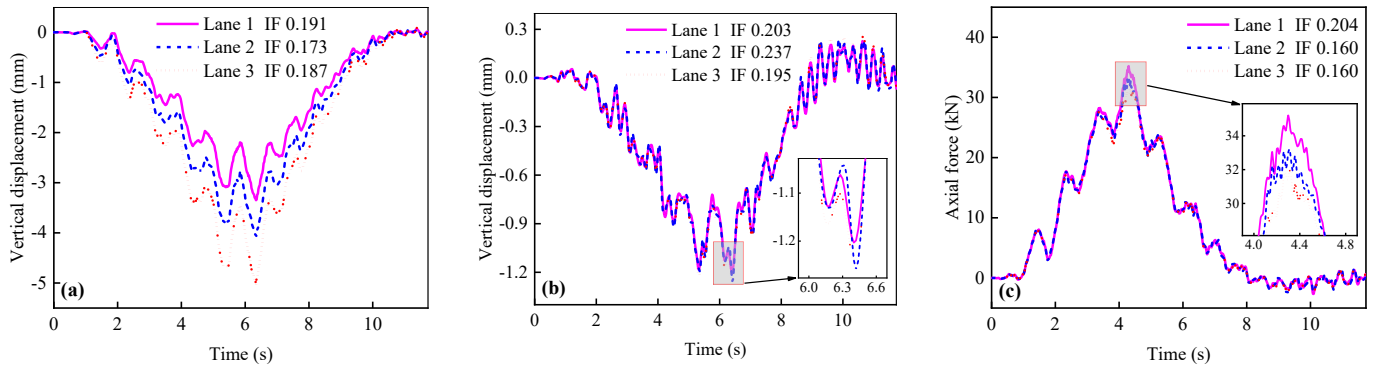


Fig. 17 Bridge IFs under different driving positions. (a) deflection time history curve of main girder feature point, (b) deflection time history curve of arch rib feature point, (c) axial force time history curve of hanger C3

As shown in Fig. 17, the lateral positioning of the vehicle during motion significantly affects the dynamic behavior of the main girder. In contrast, its impact on the arch ribs and hangers is relatively minor. For example, when the vehicle is in lane 1, lane 2, and lane 3, the maximum dynamic deflection of the main girder is 3.1 mm, 4.1 mm, and 5.1 mm, respectively. This suggests that as the vehicle moves closer to the outer edge of the bridge, the dynamic deflection of the main girder increases, largely due to the eccentric loading effect. However, vehicle lateral position does not show a discernible trend in the IFs of the main girder or arch ribs.

For hanger C3, when the vehicle is in lane 1, its axial force response is the largest, reaching 35.2 kN. This value is 6.1% higher than in lane 2 and 9.1% higher than in lane 3. This change is closely related to the relative distance

between the vehicle and hanger C3. When the vehicle is closer to hanger C3, the axial force response increases. Conversely, when the vehicle is farther from hanger C3, the axial force response decreases. Additionally, when the vehicle is in lane 1, the axial force IF of hanger C3 is also the highest, approximately 1.3 times that in lanes 2 and 3. This phenomenon is linked to the unique hanger layout of the Qinhui Bay Bridge. Hanger C3 is located near the midpoint of the bridge span, an area where the dynamic response of the main girder is most significant. Furthermore, due to its relatively short length, hanger C3 has high stiffness and a high natural frequency. Under the action of moving vehicles, the impact effect is more pronounced.

5. Conclusions

Using field tests and numerical analysis of the Qinhuai Bay Bridge, we quantified the dynamic response and IFs of a multi-rib, special-shaped tied-arch bridge under heavy truck loading. The key findings are as follows:

(1) Deck roughness is the primary cause of excessive IFs in special-shaped tied-arch bridges, with arch ribs being the most sensitive component. Maintaining deck smoothness at Grade A or better should therefore be a mandatory maintenance criterion.

(2) The arch-rib IF can reach 3 to 4 times the code value when heavy trucks encounter bumps or brake suddenly. In addition to speed limits, warning signs and brake-relief strips should be installed to mitigate these extreme transient impacts.

(3) Platooning heavy trucks with ≥ 30 m spacing significantly reduces IFs. Short hangers near mid-span exhibit the strongest dynamic amplification, so their prestress and fatigue status should be monitored frequently throughout the bridge's service life.

6. Limitations and future work

While this study offers valuable insights into the dynamic IF of multi-arch rib, special-shaped tied-arch bridges under heavy truck loads, several limitations must be acknowledged. First, the field tests were conducted on a single bridge (Qinhuai Bay Bridge), which may limit the applicability of the findings to other bridges with different geometries or material properties. Second, the numerical simulations focused primarily on vertical vehicle-bridge interaction, without fully accounting for lateral or longitudinal dynamic effects, such as centrifugal forces or braking-induced horizontal forces. Additionally, the influence of environmental factors (e.g., wind, temperature) on dynamic responses was not addressed.

Future research could expand on this work in the following ways:

(1) Conduct similar studies on multiple special-shaped tied-arch bridges to validate and generalize the findings;

(2) Investigate the dynamic response under more complex loading scenarios, such as multi-directional seismic excitation or combined vehicle-wind loading;

(3) Incorporate long-term structural health monitoring data to assess the fatigue performance and service life of critical components such as arch ribs and short hangers;

(4) Develop improved design guidelines or IF formulas specifically tailored for non-conventional arch bridges, considering real-world traffic and road conditions.

Interest statement

The author(s) stated that there were no potential conflicts of interest regarding the research, writing, and/or dissemination of this article.

Data availability statement

The corresponding author can provide relevant data, models, or code that underpin the results of this research upon receipt of a reasonable request.

CRedit authorship contribution statement

Tailei Chen: Conceptualization, Methodology, Software, Validation, Formal analysis, Investigation, Data Curation, Writing - Original Draft, Visualization, Supervision, Project administration. **Shiling Xing:** Methodology, Validation, Formal analysis, Data Curation, Writing - Review & Editing. **Liming Zhu:** Conceptualization, Resources, Supervision, Writing - Review & Editing, Funding acquisition, Project administration. **Wei Li:** Investigation, Data Curation, Writing - Review & Editing.

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