

ADVANCED STEEL CONSTRUCTION

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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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Special-shaped tied-arch bridge;
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Vehicle running test

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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.

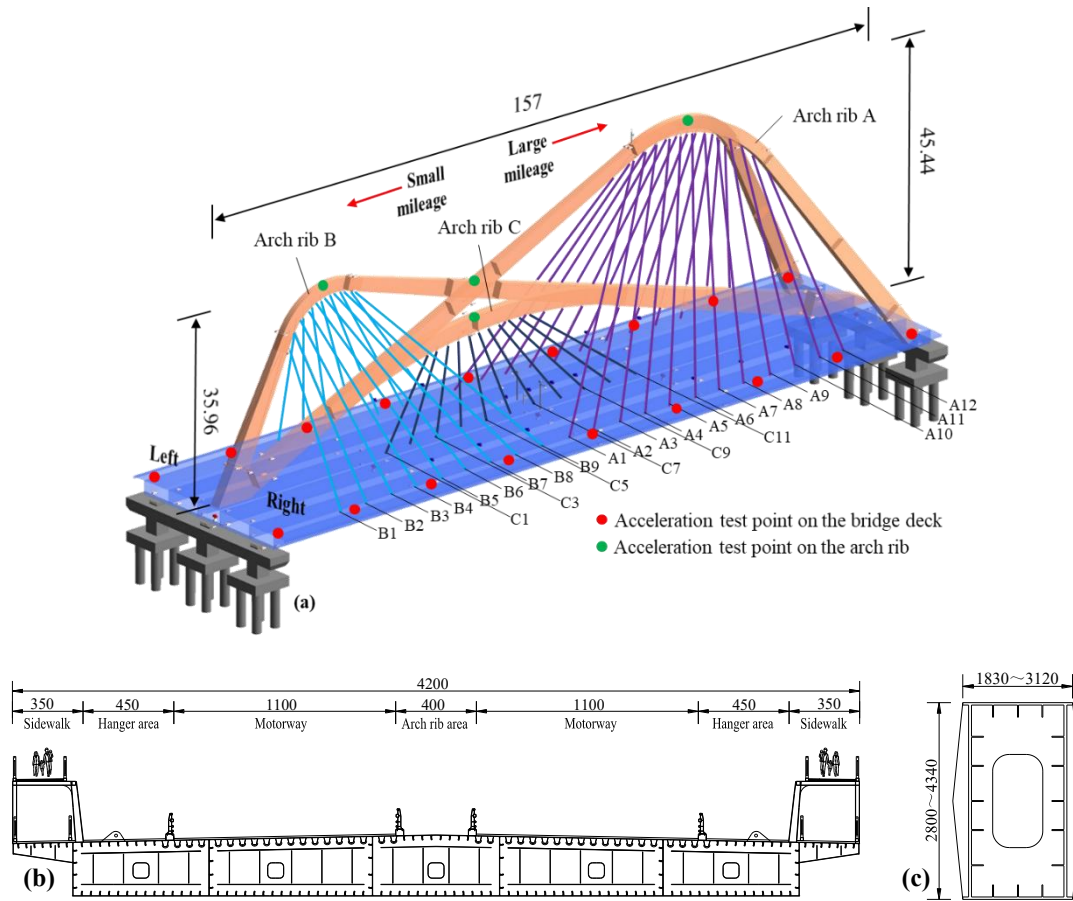


Fig. 1 Overview of Qinhuai Bay Bridge. (a) bridge component number and measuring point arrangement (unit: m), (b) typical main girder section (unit: cm), (c) typical arch rib section (unit: mm)

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.

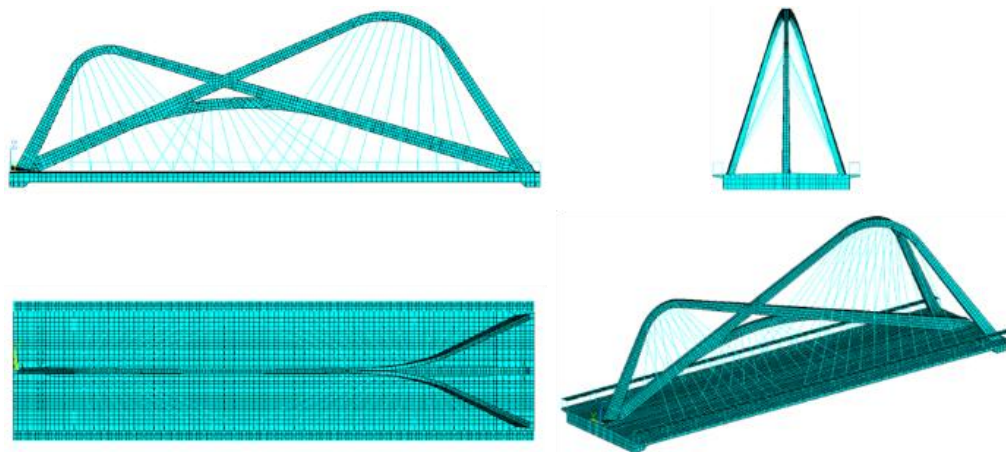


Fig. 2 Finite element model of Qinhuai Bay Bridge

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.

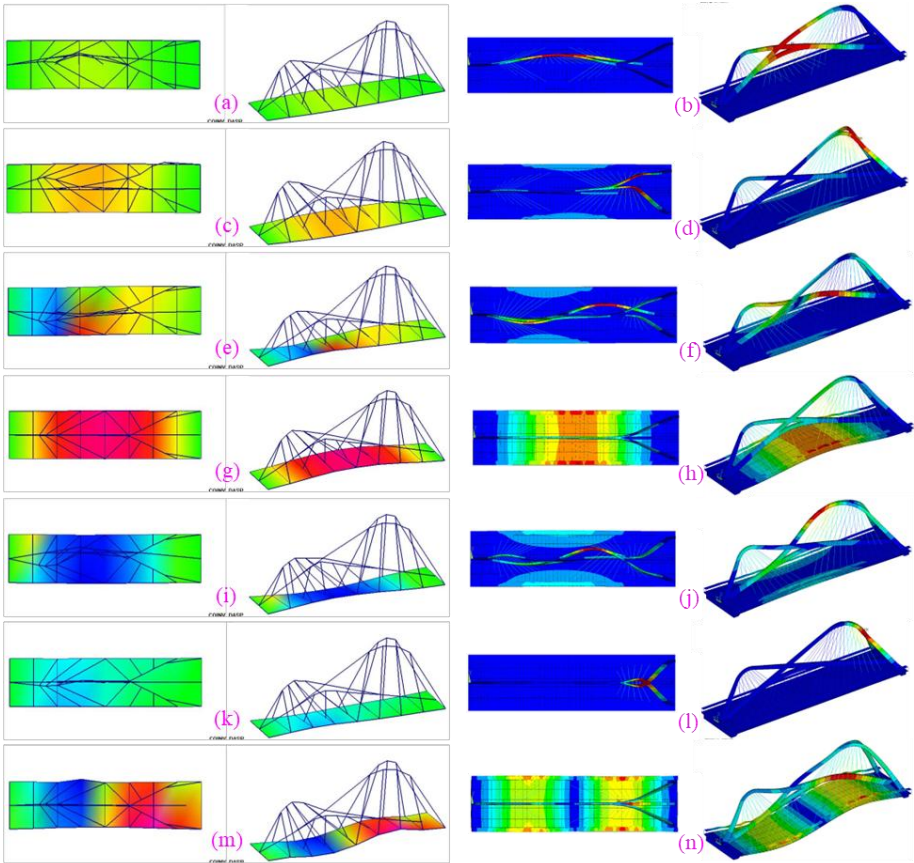


Fig. 3 Comparison of theoretical and measured vibration modes of Qinhuai Bay Bridge. (a), (c), (e), (g), (i), (k), (m) are the first 7 orders of measured vibration patterns of bridge, (b), (d), (f), (h), (j), (l), (n) are the first 7 orders of theoretical vibration patterns of the bridge

Table 2 and Fig. 3 show that the Qinhui 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 Qinhui 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.

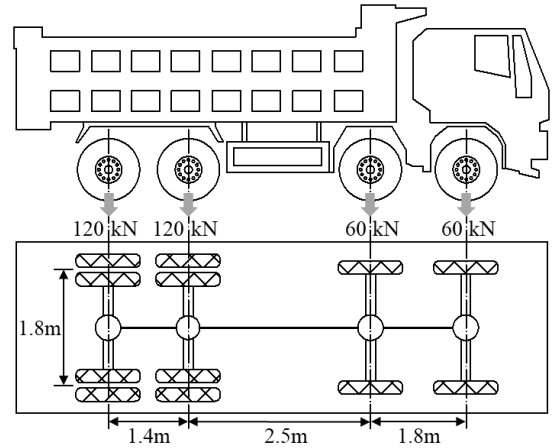


Fig. 4 Test vehicle wheelbase and axle load

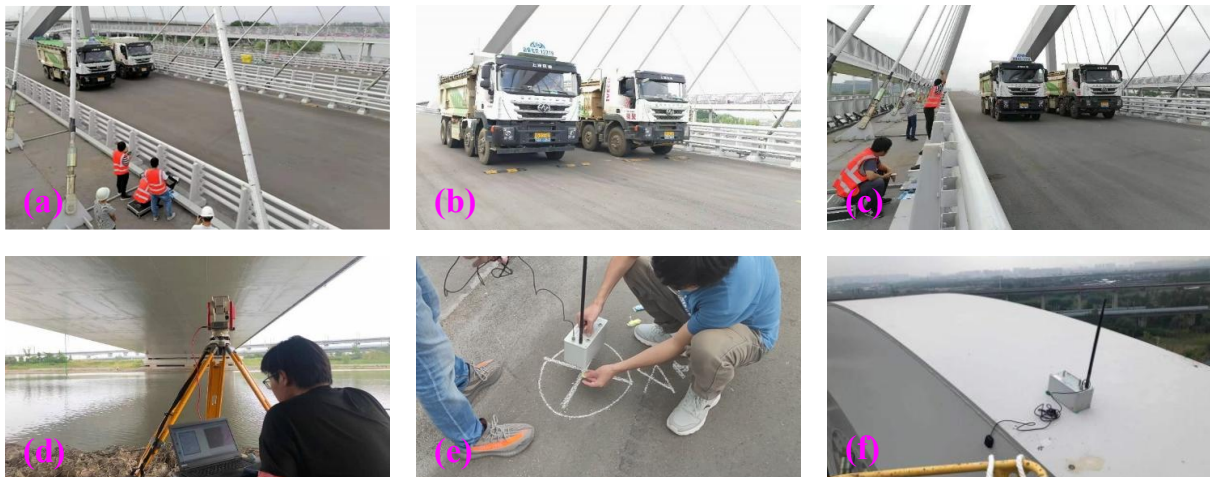


Fig. 5 Modal and IF test of Qinhui Bay Bridge. (a) moving vehicle test, (b) obstacle-crossing test, (c) braking test, (d) mid-span dynamic deflection monitoring, (e) acceleration test point on the bridge deck, (f) acceleration test point on the arch rib

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.

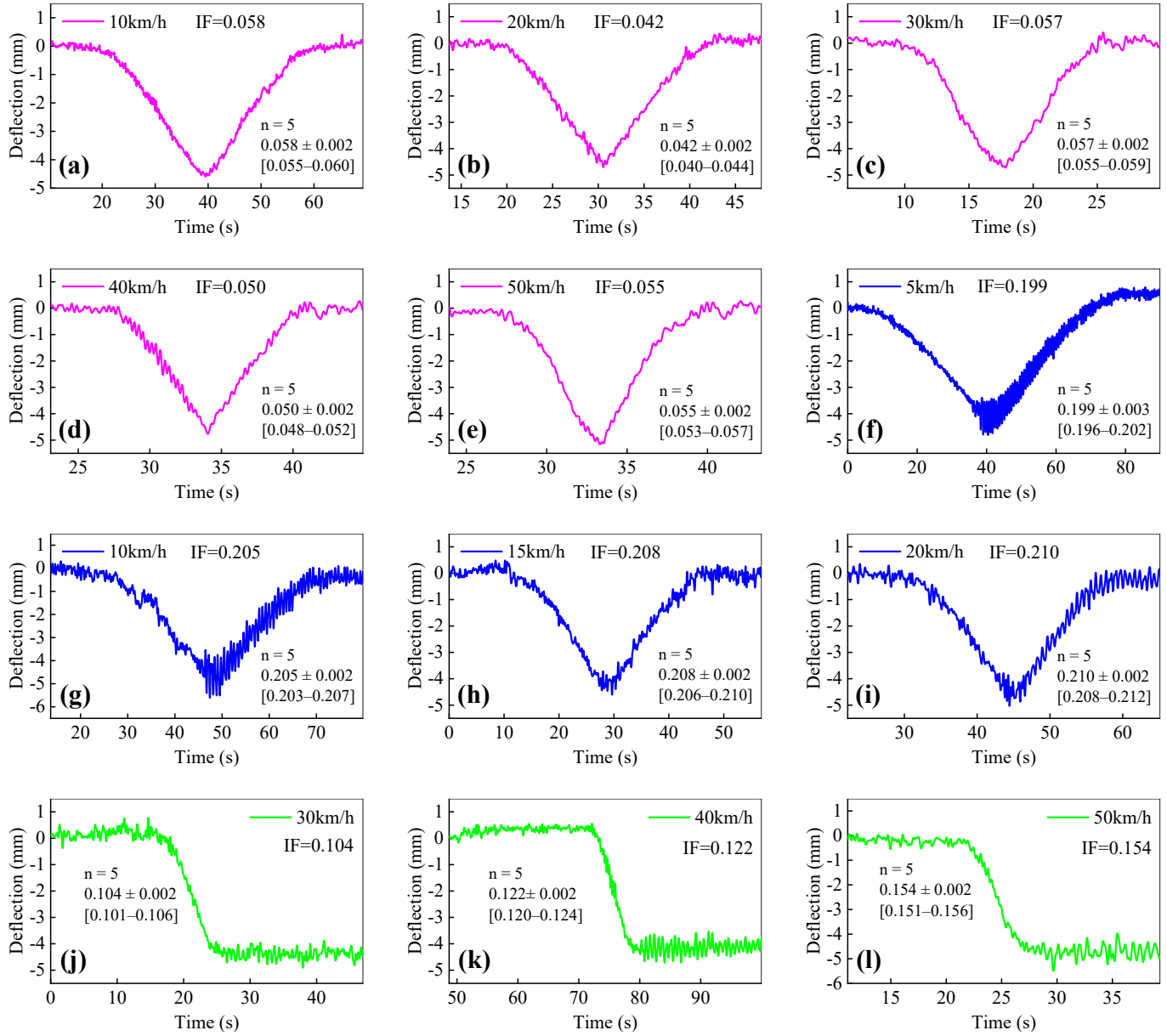


Fig. 6 Dynamic deflection and IFs of mid-span measuring points of main girder under different working conditions. (a) ~ (e) moving vehicle conditions, (f) ~ (i) obstacle-crossing conditions, (j) ~ (l) braking conditions

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.

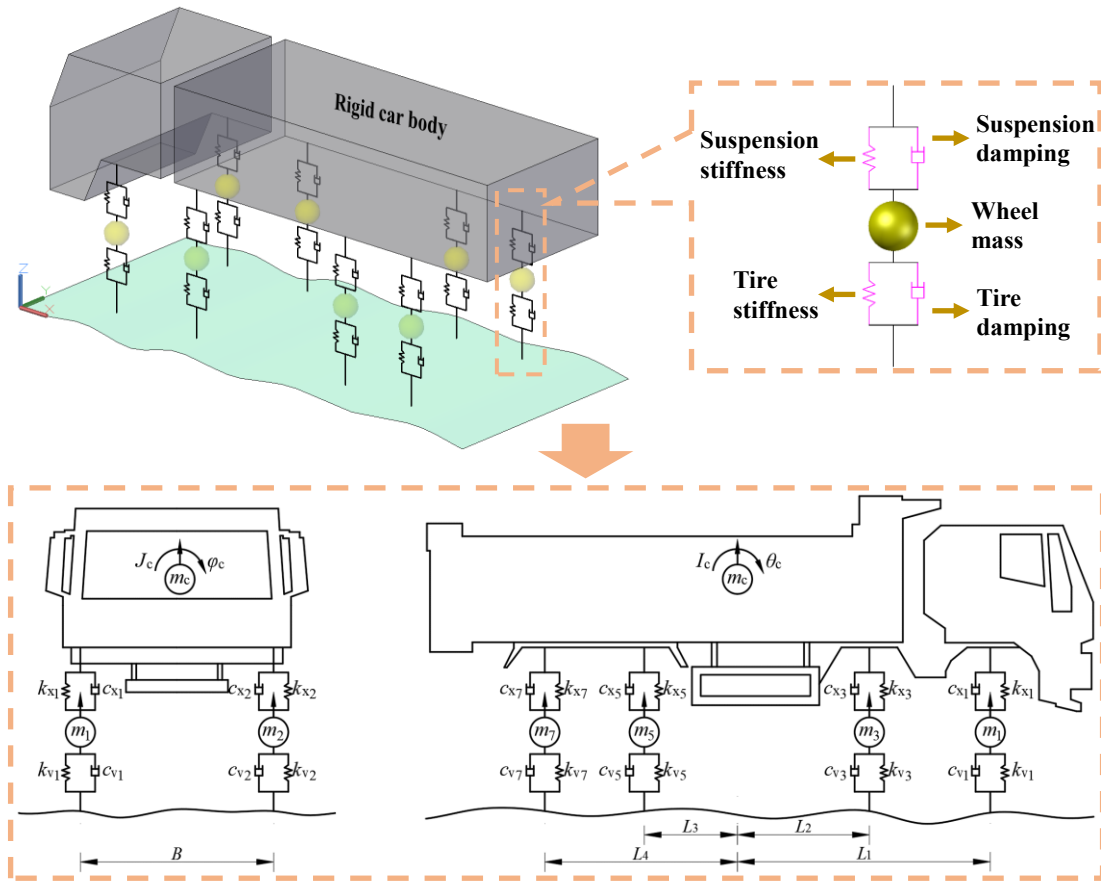


Fig. 7 Four-axle truck model

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 (m²/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: $n_d=0.01$ m⁻¹, $n_u=5$ m⁻¹, $n_0=0.1$ 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.

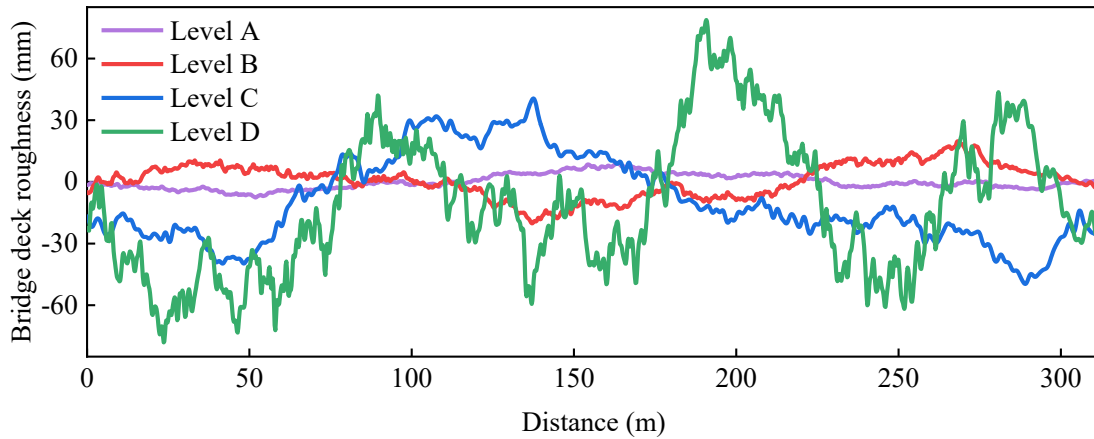


Fig. 8 A ~ D level bridge deck roughness

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.

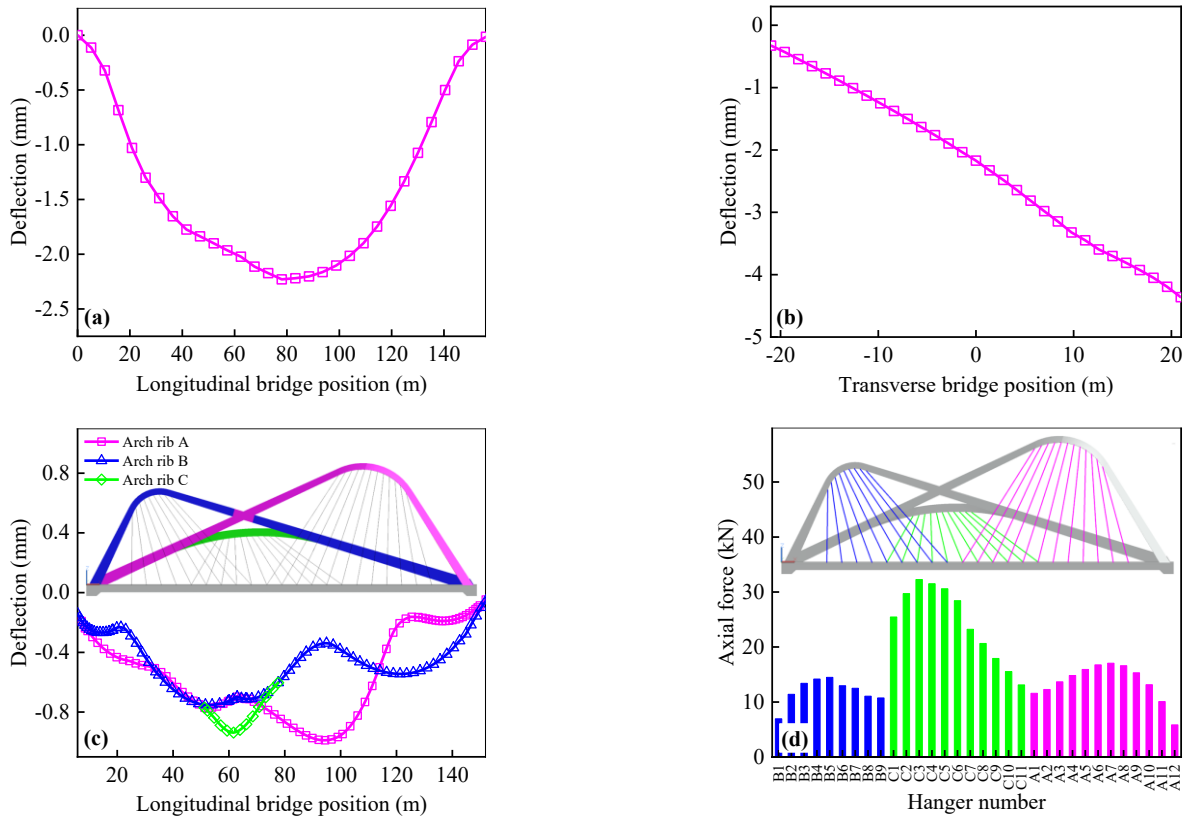


Fig. 9 Dynamic response of bridge. (a) longitudinal distribution of deflection extreme value of main girder, (b) transverse distribution of deflection extreme value of mid-span of main girder, (c) longitudinal distribution of deflection extreme value of arch rib, (d) axial force extreme value of hangers

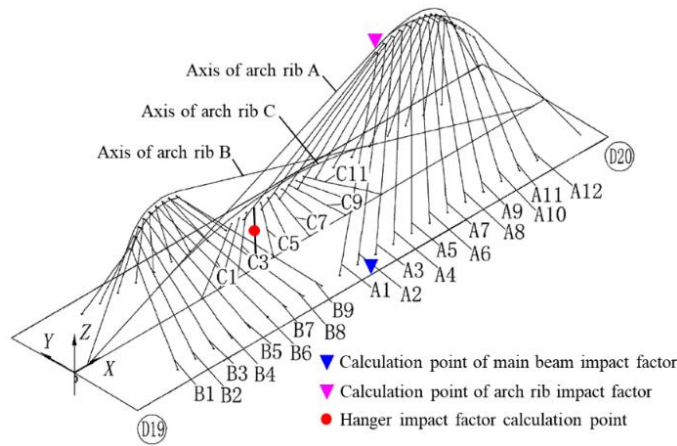


Fig. 10 IF calculation point

4.3. Bridge dynamic IF analysis

The general specification for the design of highway bridges and culverts

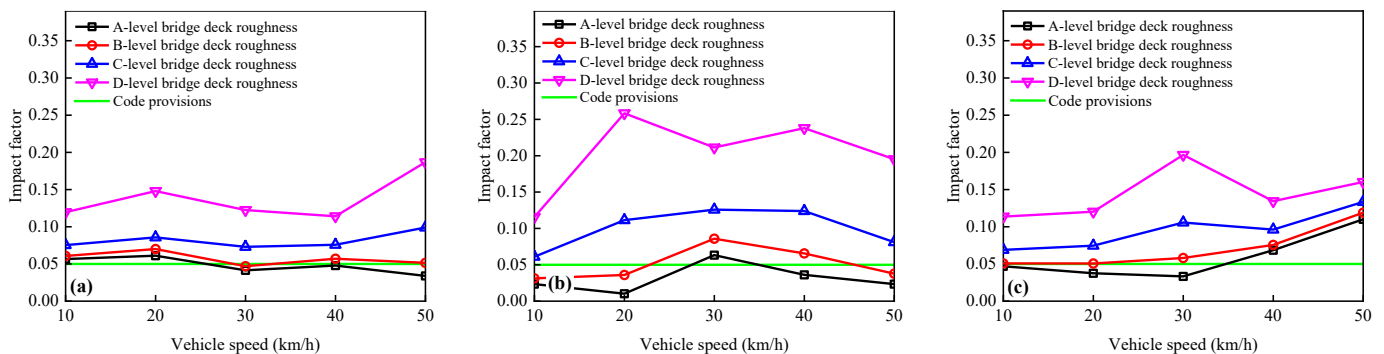


Fig. 11 Bridge IFs. (a) displacement IFs of main girder feature point, (b) displacement IFs of arch rib feature point, (c) axial force IFs of hanger C3

(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.

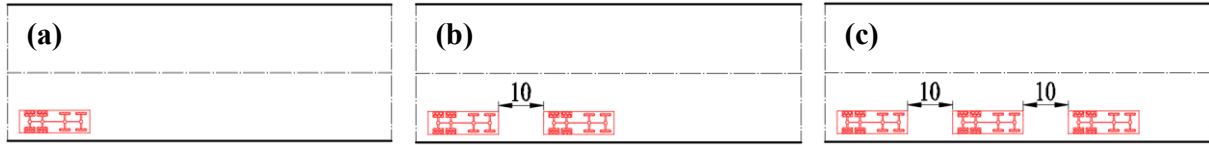


Fig. 12 Longitudinal vehicle layout (unit: m). (a) single truck, (b) two trucks, (c) three trucks

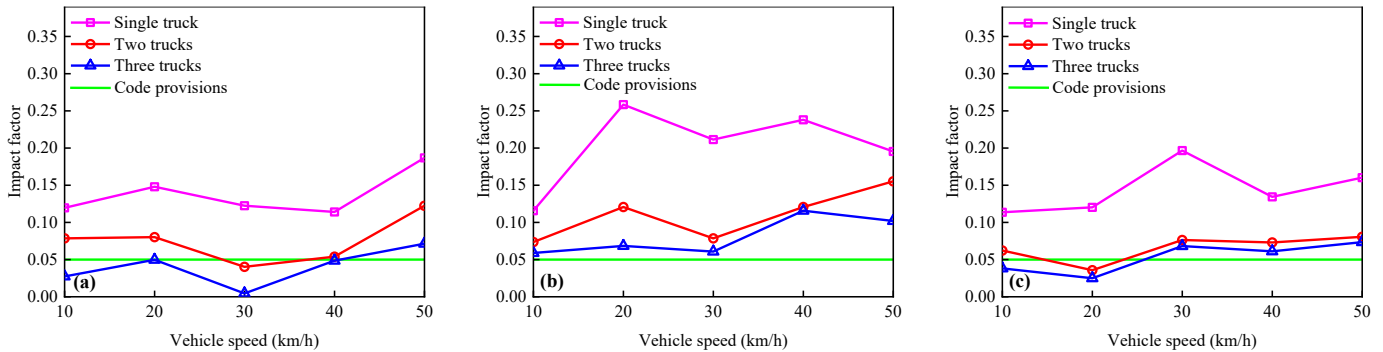


Fig. 13 Bridge IFs under different driving numbers. (a) displacement IFs of main girder feature point, (b) displacement IFs of arch rib feature point, (c) axial force IFs of hanger C3

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.

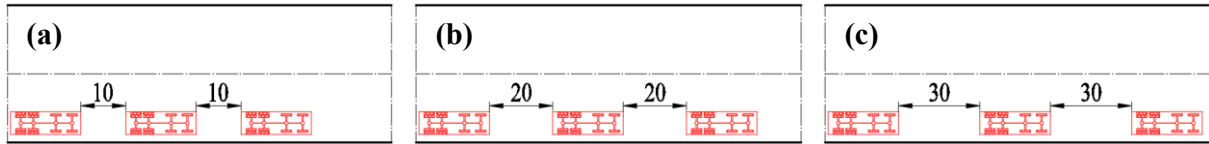


Fig. 14 Vehicle driving distance diagram (unit: m). (a) 10m spacing, (b) 20m spacing, (c) 30m spacing

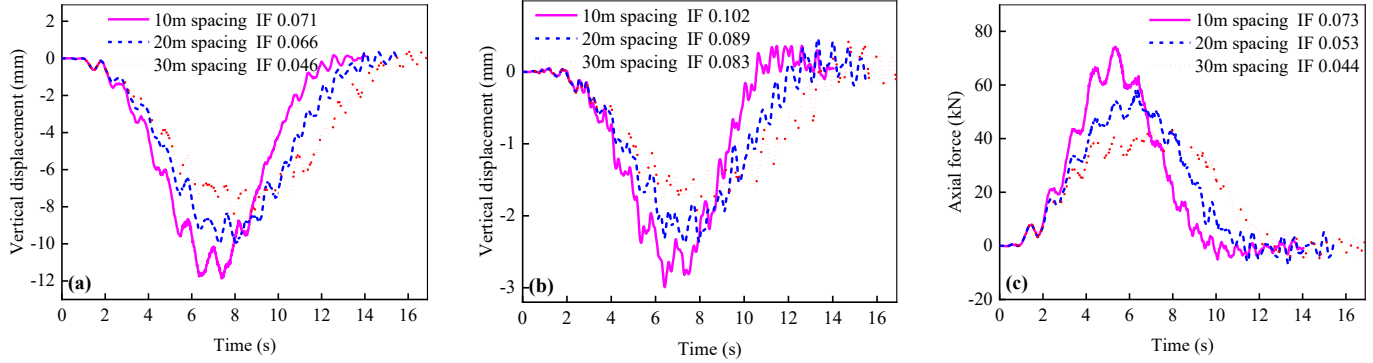


Fig. 15 Bridge IFs under different driving distances. (a) deflection time history curve of main girder feature point, (b) deflection time history curve of arch rib feature point, (c) axial force time history curve of hanger C3

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.

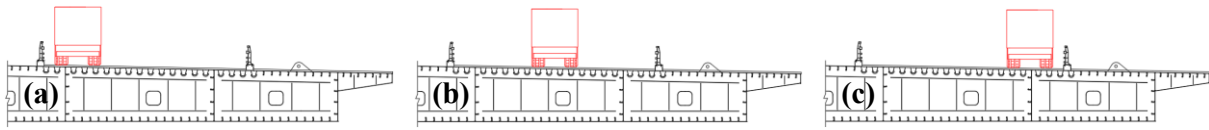


Fig. 16 Vehicle driving position diagram. (a) lane 1, (b) lane 2, (c) lane 3

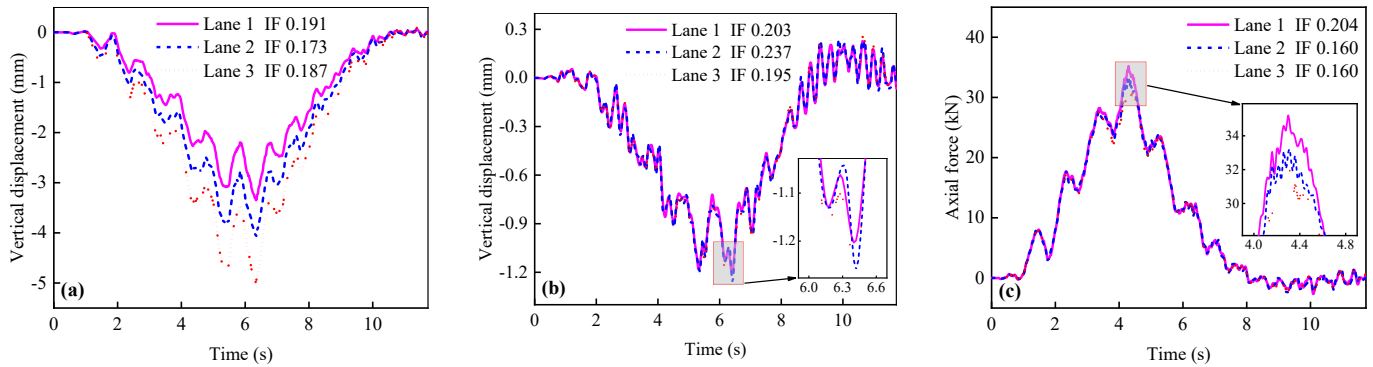


Fig. 17 Bridge IFs under different driving positions. (a) deflection time history curve of main girder feature point, (b) deflection time history curve of arch rib feature point, (c) axial force time history curve of hanger C3

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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MECHANICAL BEHAVIOR AND YIELD-LINE-BASED DESIGN OF THROUGH-CORE BOLTED HSSC ANGLE JOINTS

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ABSTRACT

Combined bolted angle beam to column joint had the advantages of high bearing capacity and considerable deformation capacity, which are desirable characteristics for semi-rigid connections considered in earthquake-resistant frame structures. This paper focuses on the fundamental bending performance and design of a novel connection type that using through-core bolts to connect the angle and hollow square section column (HSSC). The bending performance was simulated using the verified finite element model (FEM) by means of monotonic loading, and the parametric analysis included angle flange thickness, angle steel strength, bolt vertical distance, stiffening rib position, thickness, shape, and number and double web angles. The results showed that angle flange thickness, steel strength, and bolt vertical distance had significant impact on bearing capacity of joints and the parameter of stiffening rib had little effect on initial stiffness. For unstiffened angle joints, the pattern of yield line was three straight lines located at the root of angle flange, angle web, and throughout bolt holes, respectively. It can be changed to two circular arc curves by setting stiffening ribs, which increased the bearing capacity of joints obviously. The web angle only formed yield line in tension zone and the pattern was the same as top angle. Three types of design method were proposed for unstiffened angle, stiffened angle, and web angle joints, respectively, by referring to the yield line theory given in Eurocode 3 and P398. Compared with results obtained from FEM, the calculated values were highly consistent with, demonstrating the reliability of the formula.

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1. Introduction

In frame structures, beam-column joints were conventionally designed as rigid connections, wherein the beam and column members were welded to ensure sufficient structural strength. However, brittle fractures at the weld interfaces were frequently identified as a predominant failure mode. This limitation was notably exemplified by the 1994 Northridge earthquake in the United States [1] and the 1995 Hanshin earthquake in Japan [2], during which the inter-story drift angles were observed to exceed ductility limits, thereby inducing complete fracture propagation across welded zones and ultimately resulting in catastrophic structural failures. To overcome these limitations, researchers suggested using semi-rigid joints with high strength bolts, since these connections could satisfy both the strength and deformation requirements [3,4]. The behavior of semi-rigid joints was studied using moment-rotation ($M-\theta$) curves [5], where two main properties moment resistance and rotational stiffness were measured. Fig. 1 showed $M-\theta$ curves for different types of semi-rigid joints [6], among which the top and seat angles connection was commonly used in earthquake-prone areas owing to its heavy ultimate moment and large deformation [7,8].

Davor Skejic et al. [9] conducted tensile and bending tests on stiffened angle connections, through which the yield lines of stiffened angles were identified. Based on the results giving a more precise assessment than Eurocode 3 [10]. Reinoso et al. [11] performed tensile tests on five sets of stiffened angle connection based on the tests conducted by Skejic [12] and established a simplified finite element model (FEM). Based on the numerical analysis, proposed an analytical method to determine the initial tensile stiffness of the joints. Movaghati et al. [13] investigated the effect of angle deformation on the nonlinear behavior and hysteresis behavior of beam to column connections by performing monotonic and cyclic loading tests. Wang et al. evaluated the bending performance of beam to H-column major-axis [14] and minor-axis [15] connections, along with the effect of bolt patterns on failure mode [16]. Zhang et al. [17] conducted monotonic tensile tests on stiffened angle to examine the impact parameters such as stiffening rib thickness, bolt arrangement, and bolt diameter on tensile strength, giving a mechanical model describing the load-displacement behavior of stiffened angle joints. Meng et al. [18] proposed an enhanced top and seat angles joints in order to address weak robustness under progressive collapse scenarios by incorporating four bent angles within the beam flange, and the efficacy of the design was confirmed through FEM.

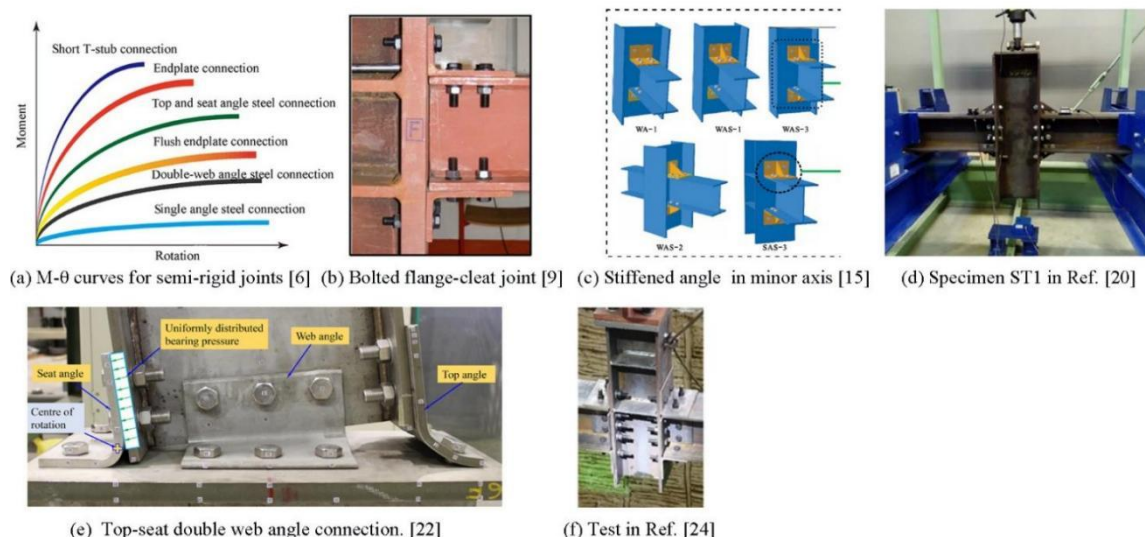


Fig. 1 Several connected types in papers

Extensive research on double web angles connections was also conducted by various scholars. The $M-\theta$ behavior of double web angles joints were examined by Kong et al. [19], who developed improved equations for initial stiffness and ultimate moment capacity based on previous research. Similarly, Reinoso et al. [20] conducted eight tests on stiffened web angle in which the rotation of the beams was monitored, based on the results a calculation method for predicting rotational stiffness was proposed using the Eurocode 3 component method. Mohammad et al. separately investigated the $M-\theta$ behavior of both top and seat angles joints [21] and double web angles joints [22] fabricated from austenitic stainless steel, with test results confirming this material's exceptional ductility. Further investigation was carried out by Alhendi et al. [23], who numerically simulated the $M-\theta$ behavior of double web angles joints with varying steel grades. In a complementary study, the fire resistance of web angle joints was modeled by Cao et al. [24]. Results indicated that while thicker angles enhanced bearing capacity at ambient temperature, thinner angles demonstrated better performance under elevated temperatures.

However, the majority of existing research, including experimental validations, has focused on connections to H-shaped columns, as shown in Fig. 1(b)–(f). The behavior of semi-rigid angle connections to HSSCs, which offer distinct advantages in terms of bi-axial stiffness and architectural potential, remains significantly understudied, particularly for through-core bolted configurations. As a fundamental step towards understanding this novel joint, this study employs a numerically-driven approach. The primary objectives are: (1) to establish a reliable finite element modeling methodology for such connections; (2) to investigate the key parameters governing their monotonic bending behavior and failure mechanisms; and (3) to propose a yield-line-based analytical model for predicting their moment capacity. It is acknowledged that the proposed design methods are derived from numerical parametric studies, and their validation is based on verifying the FEM against established experimental benchmarks for analogous connection mechanics. Subsequent cyclic testing is required to fully assess seismic performance.

2. Research strategies

The mechanical performance of the joints was simulated using ABAQUS [25], which was recognized as widely used software for large-scale general-purpose FEM analysis. The simulations were conducted under monotonic static loading. This approach is adopted to establish a clear understanding of the joint's strength hierarchy, deformation capacity, and the formation of yield line mechanisms without the added complexity of load reversal. The derived moment capacity and stiffness parameters serve as direct inputs for nonlinear static analysis of frames and form the basis for planning future cyclic testing protocols.

2.1. Validations

2.1.1. General

To assess the accuracy of the FEMs applied in this paper, four specimens in Ref. [14] were selected to verify the accuracy of the material properties, boundary conditions and contact relationships in FEM, namely, specimen “SA-1”, unstiffened angle connection, specimen “SAS-1”, stiffened top angle connection, specimen “SAS-2”, stiffened seat angle connection, and specimen “SAS-3”, stiffened top and seat angles connection. The primary aim of this validation is not to directly validate the HSSC joint configuration itself, but to rigorously confirm that the developed FEM techniques, including material laws, contact interactions, bolt pretension, and element formulation, are capable of accurately capturing the complex mechanical behavior that is fundamental to all bolted angle connections. This established modeling framework is then applied

with confidence to the parametric study of the through-core bolted HSSC joint, for which dedicated experimental data are currently scarce. In the validations, the cross-section of the H-column was $\text{HW}250 \times 255 \times 14 \times 14$ and of the I-beam was $\text{HM}194 \times 150 \times 6 \times 9$ mm, the angle was $\text{L}140 \times 90 \times 8$ mm, with a width of 150 mm. A total of 12 bolts with 10.9 grade M16 high strength were used to connect, and the preload was set to 100 kN. Detailed constructions were shown in Fig. 2.

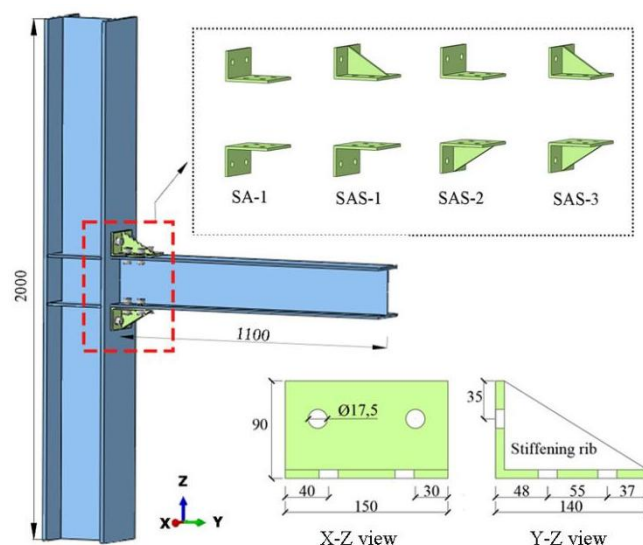


Fig. 2 Detailed constructions of test specimens in Ref. [14] (Unit: mm)

2.1.2. Finite element model

The FEM model had symmetry in geometric conditions, boundary conditions, and loading conditions, so the 1/2 model can be taken to save time and cost and improve calculation accuracy [26,27]. A simplified bilinear curves model was applied for stress-strain constitutive relationship of steel taken from Ref. [14], as shown in Table 1. All parts were modelled using 3-dimensional 8-node reduced integral units (C3D8R). A finer mesh size of 3 or 5 mm was used for the parts with large stress concentration and deformation, and a coarser mesh size of 15 or 20 mm was used where away from the core area.

The HSSC bottom surface was fully fixed in all degrees of freedom, simulating a rigid foundation. The top surface was constrained in all linear displacements but free to rotate, simulating a hinged support with the column inflection point assumed at mid-height. This boundary condition setup is representative of a typical beam-column sub-assembly test configuration and aims to isolate the joint behavior from global column bending, which is appropriate for a comparative parametric study focused on connection components. Surface-to-surface contact was defined between all interacting parts. The tangential behavior was modeled using the Coulomb friction model with a friction coefficient of 0.3. This value is specified for untreated steel surfaces in the Chinese design code GB50017-2017 [28] and aligns with common practice in analogous numerical studies of bolted connections. The “Hard” contact pressure-overclosure relationship was used for normal behavior, allowing for separation which is critical for capturing prying action. Bolt pretension was applied to the shank of each high-strength bolt to replicate the installed state. Geometric nonlinearity was included in the analysis step to capture large rotations and potential instability phenomena.

Table 1
Material properties of steel parts in Ref. [14] and mesh size

Steel parts	Thickness/mm	E/MPa	Yield strength/MPa	Ultimate strength/MPa	Elongation/%	Mesh size/mm
Column flange	14	204981	395	590	27.3	5/20
Column web	14	204981	395	590	27.3	5/20
Beam flange	9	207072	250	425	29.6	5/15
Beam web	6	207072	250	425	29.6	5/15
Angle	8	203500	245	360	30.2	5
Stiffening rib	8	199349	320	475	29.8	5
Welding plate	14	204922	245	415	28.3	10
Bolt	10.9 grade M16	206000	1020	1120	12.7	3

2.1.3. Results of validations

Fig. 3 showed the comparison of test and FEM results. For specimens SA-1 and SAS-2, the loading was expired due to the large deformation on top angle and the failure mode was the top angle tensile yielding. For specimens SAS-1 and SAS-3, the brittle fracture occurred at the welded connections between the stiffened top angle and stiffening rib which was not suitable to continue loading. It can be seen that the stiffened top angle flange significantly deformed and formed two arc curve yield lines, while the stiffened angle web formed one

straight yield line paralleled to the web, as well as stiffened seat angle and stiffening rib showed compressive buckling. In summary, the results of failure mode FEM analysis were in good accordance with the test results.

The $M-\theta$ curve was defined as shown in Fig. 4(a), and the same approach was used in Refs. [29-31]. It can be seen that the stiffened angle joints fitted the test results, while the unstiffened joints were 14.3% lower than the tests result on yield strength. In summary, FEM curves basically coincided with the tests.

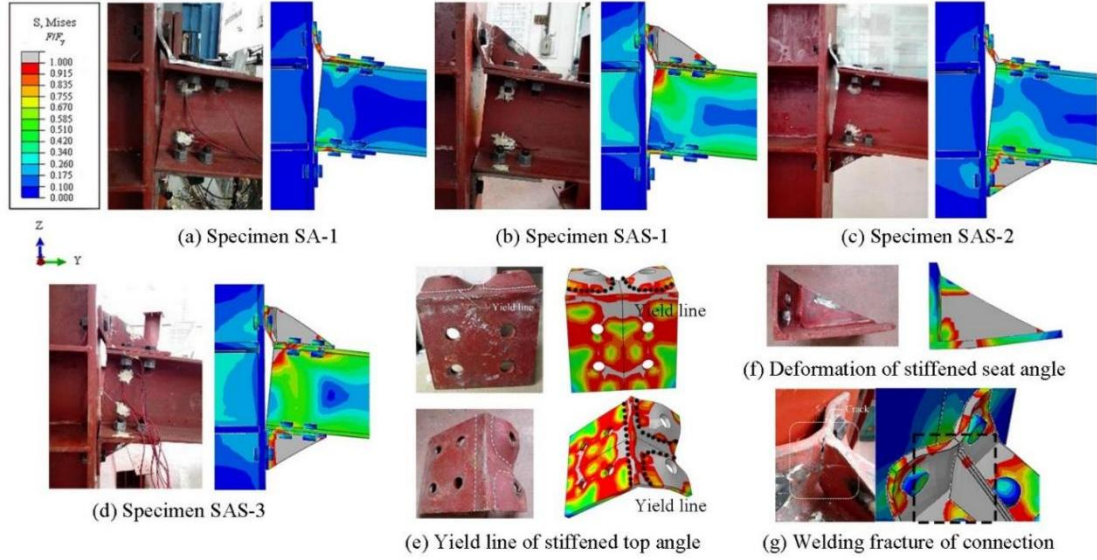


Fig. 3 Comparison of test and FEM results on deformation

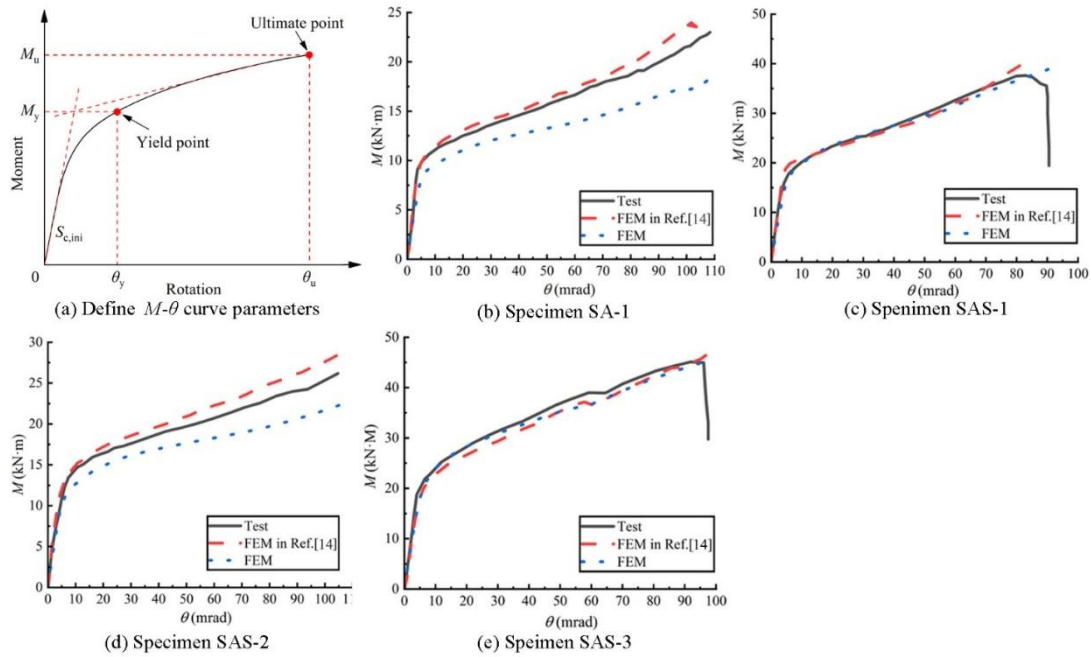


Fig. 4 Comparison of test and FEM $M-\theta$ curves

Table 2

Main results of test and FEM

Specimens	$S_{ini, test}$	$S_{ini, FEM}$	$S_{ini, test}/S_{ini, FEM}$	$M_{y, test}$	$M_{y, FEM}$	$M_{y, test}/M_{y, FEM}$	$M_{u, test}$	$M_{u, FEM}$	$M_{y, test}/M_{y, FEM}$
SA-1	2.899	2.389	1.213	11.37	9.94	1.144	22.87	18.18	1.258
SAS-1	4.866	3.605	1.349	19.09	19.11	0.992	35.45	34.62	1.024
SAS-2	3.096	2.248	1.377	15.03	13.26	1.133	26.54	21.91	1.211
SAS-3	5.030	4.192	1.199	23.63	23.68	0.997	45.70	44.78	1.021
Mean			1.284			1.066			1.128
Standard deviation			0.079			0.072			0.107

Note: S_{ini} (kN·m/mrad) denoted the initial stiffness; M_y (kN·m) and M_u (kN·m) denoted the yield moment and ultimate moment, respectively.

2.1.4. Sensitivity analysis of key modeling assumptions

Given the influence of modeling decisions on the simulated response of bolted connections, a focused sensitivity analysis was conducted on the unstiffened joint NT to assess the robustness of the primary findings. Three parameters were varied:

(1) Friction coefficient: Values of 0.2, 0.3, and 0.4 were modeled.

(2) Bolt pretension: Levels of 70 kN, 100 kN, and 130 kN were applied.

(3) Initial geometric imperfection: A local outward deviation of $L/500$ was introduced at the critical compressive zone of the seat angle web to trigger buckling modes.

The results, summarized in Table 3, indicate that while the absolute values of initial stiffness and yield moment showed some variation, the fundamental failure mechanism remained unchanged. Specifically, the formation of the three characteristic yield lines in the tension angle was consistently observed across all variations. The deformation patterns and the sequence of component yielding were not qualitatively altered. This demonstrates that the key conclusions regarding yield line patterns and the relative influence of geometric parameters are robust within a reasonable range of these modeling assumptions.

Table 3
Sensitivity analysis results for specimen NT

Model variant	Friction Coeff.	Bolt pretension (kN)	Imperfection	S_{ini} (kN-m/mrad)	M_y (kN-m)	Yield line pattern
Baseline NT	0.3	100	None	3.08	13.18	Three straight lines
Var. 1	0.2	100	None	2.95	12.85	Unchanged
Var. 2	0.4	100	None	3.18	13.42	Unchanged
Var. 3	0.3	70	None	2.92	12.91	Unchanged
Var. 4	0.3	130	None	3.21	13.52	Unchanged
Var. 5	0.3	100	$L/500$	3.05	12.95	Unchanged

2.2. Specimens design

The joint consisted of HSSC, HSS inner stiffener, I-steel beam, angles,

Table 4
FEM parameters

Specimens	Angle thickness/mm	Yield strength/MPa	p	Stiffening rib			Web angle		
				Position	Shape	Number	Thickness/mm	Angle thickness/mm	Rib thickness/mm
NT	10	245	55	null					
NT1	8	245	55	null					
NT2	12	245	55	null					
NT3	14	245	55	null					
NY1	10	355	55	null					
NY2	10	390	55	null					
NY3	10	420	55	null					
NY4	10	460	55	null					
NY5	10	500	55	null					
NP1	10	245	45	null					
NP2	10	245	50	null					
NP3	10	245	60	null					
NP4	10	245	65	null					

stiffening ribs, high strength bolts, and through-core bolts. Among them, the cross-section of the HSSC was $200 \times 200 \times 10$ mm, with a length of 1200 mm, the cross-section size of the HSS inner stiffener was $178 \times 178 \times 10$ mm, with a length of 500 mm. The purpose of the HSS inner stiffener is to provide substantial lateral restraint to the HSSC wall, thereby mitigating excessive out-of-plane deformation under bolt prying forces [32,33]. This design detail aims to ensure that the failure mechanism is governed by yielding of the angles rather than localized distortion of the column face, creating a boundary condition for the angle that is more comparable to that provided by a stiffened flange of an H-column, as used in the validation models. The detailed construction was shown in Fig. 5 and material properties were used in Table 1.

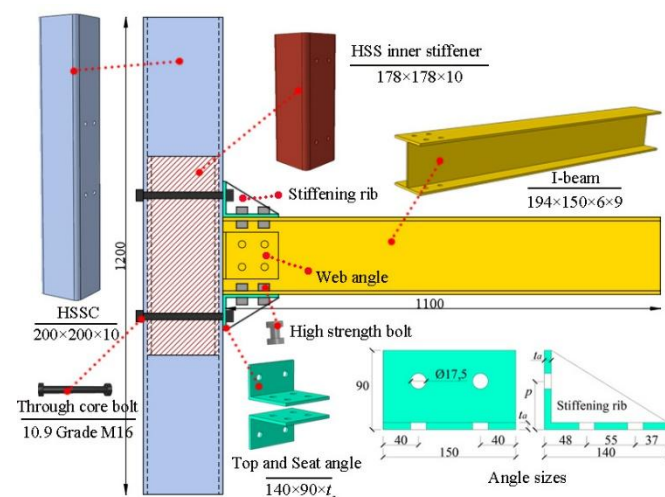


Fig. 5 Geometric construction of joints in this paper (Unit: mm)

In this paper, a total of 8 sets of FEMs were designed to study the bending performance, including angle thickness, angle steel strength, bolt vertical distance, stiffening rib position, stiffening rib shape, stiffening rib thickness, stiffening rib number, and double web angles, detailed parameters were shown in Table 4.

Specimens	Angle thickness/mm	Yield strength/MPa	p	Stiffening rib				Web angle	
				Position	Shape	Number	Thickness/mm	Angle thickness/mm	Rib thickness/mm
AR	10	245	55	top-seat	triangle	1	8		
AR1	10	245	55	seat	triangle	1	8		
AR2	10	245	55	top	triangle	1	8		
ARS1	10	245	55	top-seat	rectangle	1	8		
ARS2	10	245	55	top-seat	pentagon	1	8		
ARN1	10	245	55	top-seat	triangle	2	8		
ARN2	10	245	55	top-seat	triangle	3	8		
ART1	10	245	55	top-seat	triangle	1	4		
ART2	10	245	55	top-seat	triangle	1	6		
ART3	10	245	55	top-seat	triangle	1	10		
ART4	10	245	55	top-seat	triangle	1	12		
BR1	10	245	55	top-seat				10	
BR2	10	245	55	top-seat	triangle	1	8	10	8
BR3	10	245	55	top-seat				10	
BR4	10	245	55	top-seat	triangle	1	8	10	8

3. Parametric analysis

3.1. Parameters of angle

3.1.1. Effect of angle flange thickness

The effect of angle flange thickness on bending capacity was studied in this section and four FEMs were established to analyze. The thickness was ranged from 8 mm to 14 mm [34], and taken specimen VT (10 mm) as an example.

At the beginning of loading, the whole joint was in elastic stage. As the loading increased, the top angle flange in the tension zone gradually formed a

gap with the column and detached. Continuing to the loading, the angle bent, firstly generating a yield line at the root of the angle. Simultaneously, a yield line was also generated at the root of the angle web, as show in Fig. 6. There was a clear concave circle around the bolt hole and a yield line was formed throughout it. The seat angle flange in the compression zone had no deformation but the web generated obvious bending deformation during the rotation of the beam. The stiffness of the column was much larger than that of the angle. Therefore, in the whole loading process, the column was always in the elastic working stage, and only the stress concentration occurred around the bolt hole.

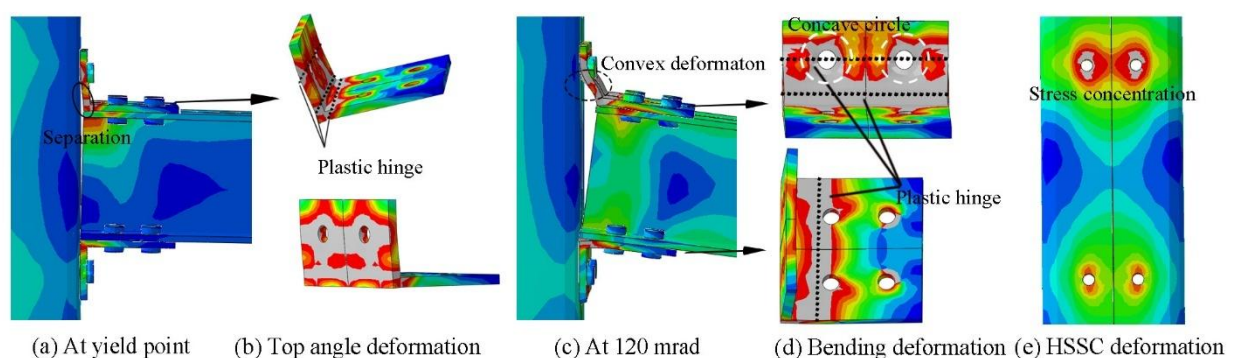


Fig. 6 Deformation of specimen VT as the loading continued

Fig. 7 and Table 5 shown the deformation, $M-\theta$ curves and main results for joints with different angle flange thickness. It can be seen that all the HSSCs were always in elastic working stage during the whole loading process at each joint. When the angle web was thinner, the beams hardly produced any deformation on specimens NT, NT1 and NT2. While the angle was thicker, the beam flanges near the rear of angle produced bending deformation as in specimens NT3 and NT4. Deformation at each joint was similar for both top

angles in tension bending as well as seat angle in compression bending. Meanwhile, three yield lines were generated at the top angle and one yield line at the seat angle.

The initial stiffness, yield moment and ultimate moment of the joints were enhanced with the increase in the angle flange thickness. This was due to the fact that as the flange thickness increased, it improved the stiffness of the angle, which allowed the joint to withstand a greater loading at the same deformation.

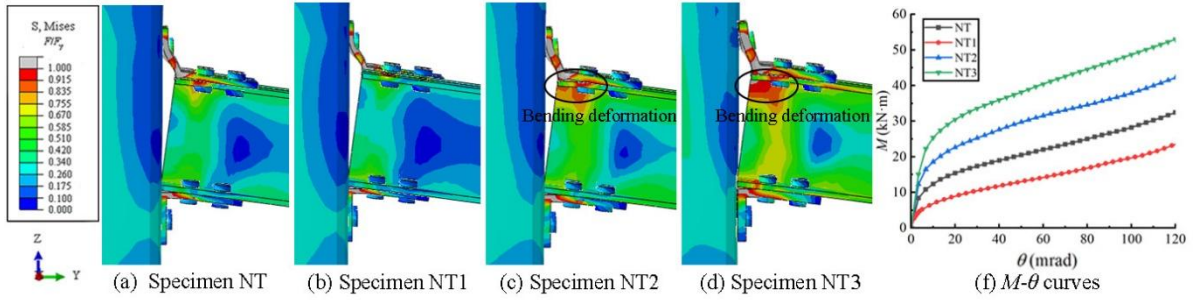


Fig. 7 Deformation of joints with different angle flange thicknesses

Table 5

Main results of the joints with different angle flange thicknesses

Specimens	Initial stiffness (kN·m/mrad)	Yield moment (kN·m)	Ultimate moment (kN·m)
NT	3.077	13.18	32.42
NT1	1.946	7.20	24.69
NT2	3.651	19.54	42.33
NT3	4.210	28.74	54.10

3.1.2. Effect of angle steel strength

The effect of angle steel strength on bearing capacity of joints was studied in this section and another five FEMs were established to analyse. The steels were taken as Table 1 Material properties of steel parts in Ref. [14] and mesh size and the nominal yield strength was taken as the yield strength according to 'GB/T19879-2023' [35]. The bilinear curve was shown in Fig. 8(a).

As can be seen from Fig. 8(b)-(f), the HSSCs and I-beams were hardly deformed and remained within the elastic state. The deformation in the joints were all taken by the angles and the angles produced similar deformation and plastic regions in each joint. Consistent with the section 3.1.1, the top angle flanges underwent bending deformation and gradually detached from the HSSC flange during the loading process. As loading continued, the area of the angle plastic region increased, forming two yield lines at the root of the flange and web, respectively. The flange stretched outward, generating a concave circle along the bolt hole and a yield line throughout the bolt hole.

From Fig. 8(g) and Table 6, it can be seen that the increase in the angle strength had little effect on the initial stiffness of the joints, but improved the yield moment and ultimate moment clearly. When the angle yield strength was increased from 245 MPa to 355 MPa, the yield moment was increased by

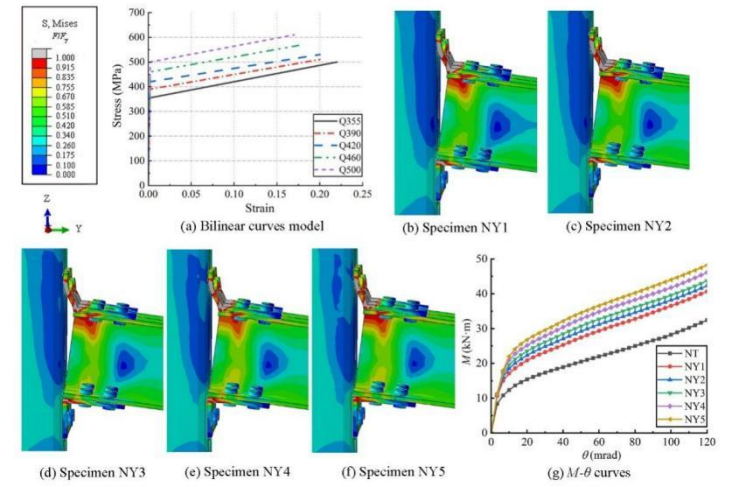


Fig. 8 Deformation of joints with different angle steel strengths

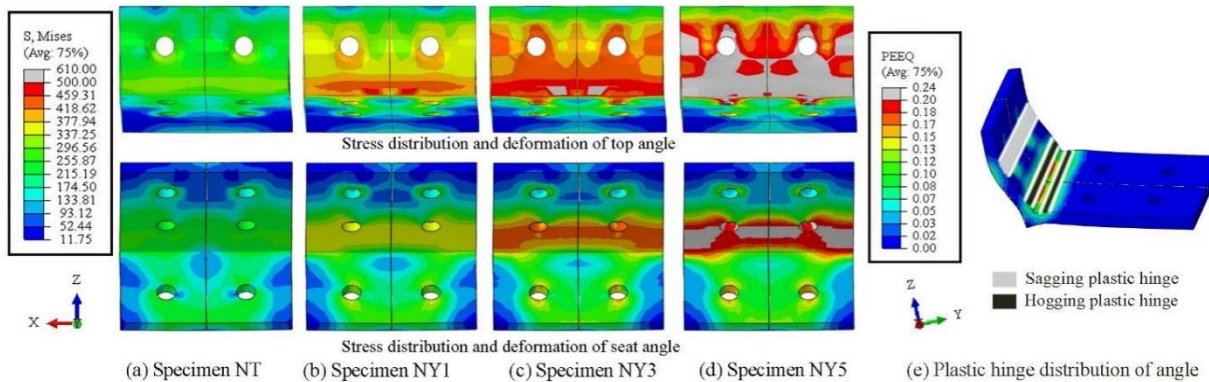


Fig. 9 Stress distribution and deformation of top and seat angles with different angle strengths

Table 6

Main results of joints with different angle strengths

Specimens	Yield moment (kN·m)	Increase over prior	Ultimate moment (kN·m)	Increase over prior
NT	13.18		32.42	
NY1	19.26	46.13%	40.65	25.38%
NY2	21.50	11.63%	42.03	3.39%
NY3	23.45	9.07%	43.74	4.06%
NY4	25.12	7.12%	46.12	5.44%
NY5	26.66	6.13%	48.20	4.51%

3.1.3. Effect of bolt vertical distance

In this section, another four FEMS were established to explore the effect of the bolt vertical distance p . The p was defined as the distance from the centre of angle flange bolt hole to the beam flange surface at intervals of 5 mm.

It can be seen from the Fig. 10(a)-(d) that the influence of the p on deformation and stress of the components of the joints except for the angles was

small. Among all the joints, still only the angles underwent large deformation. The values of p had a greater effect on the stress distribution of angles. When the p was smaller, the plastic region was mainly concentrated in the lower flange. While the p was larger, the plastic region was divided into two portions, one was concentrated at the rear of the flange and the other was concentrated around the bolt holes, as shown in Fig. 10(e) and (f).

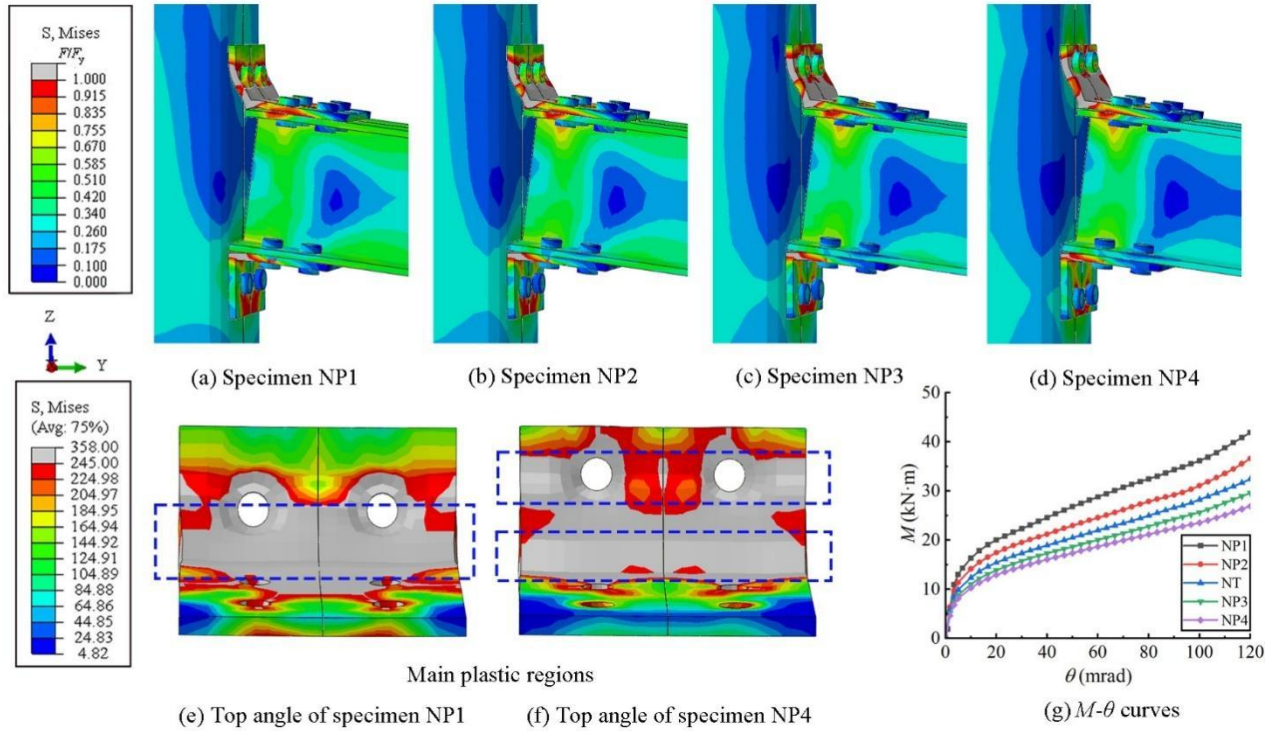


Fig. 10 Deformation of joints with different bolt vertical distance

The Fig. 10(g) and Table 7 illustrated the $M-\theta$ curves and main results of the joints. It can be seen that both the initial stiffness and ultimate moment decreased with the increased of the bolt vertical distance. When the p was larger, the deformation capacity of the angle was fully utilized, which was manifested in lower initial stiffness. With smaller p , the yield moment was elevated, attributed to the smaller distance between the two yield lines when the angle yielded.

Table 7
Main results of joints with different bolt vertical distance

Specimens	Initial stiffness (kN·m/mrad)	Yield moment (kN·m)	Ultimate moment (kN·m)
NP1	3.801	18.05	41.92
NP2	3.665	15.90	36.61
NT	3.077	13.18	32.42
NP3	2.891	12.55	29.59
NP4	2.751	11.79	26.89

3.2. Parameters of stiffening rib

3.2.1. Effect of stiffening rib position

The study on the position of stiffening rib included three specimens, namely, specimen AR, with both top and seat stiffening rib; specimen AR1, with seat stiffening rib; and specimen AR2, with top stiffening rib. The detailed construction of the joints was shown in Fig. 11(a) and (b).

From Fig. 11(c) and (d), the specimen AR1 with seat stiffening rib was top

angle yield firstly and generated yield lines at the root of the flange and throughout bolt holes. At this time, the seat angle did not deform significantly while the seat stiffening rib formed a plastic region parallel to the hypotenuse and expanded inward. Until the loading ended, the top angle was subject to tensile bending and the seat angle and seat stiffening rib were compressive bending. Overall, the deformation of specimen AR1 was the same as specimen NT.

Unlike specimen AR1, the specimen AR2 was stiffening rib yield firstly. It can be seen from Fig. 11(c) and (d) that the top angle web did not deform significantly at this time, but the vertical middle of the flange deformed forward under the tension of the stiffening rib and separated from the column, forming two circular arc yield lines around the bolt holes. As loading continued, the top angle gradually separated from the column wall, with only the periphery of bolt holes remaining in contact with column under the bolt anchorage. The stiffening rib enhanced the stiffness of the angle, resulting a plastic region in the beam web. The seat angle deformed in the same as specimen NT.

Similarly, specimen AR was top stiffening rib yielded firstly. The deformation of the top angle was the same as that of specimen AR2 and the deformation of the seat angle and stiffening rib was the same as specimen AR1. A larger plastic hinge was formed at the beam web, as shown in Fig. 11(g) and (h).

The $M-\theta$ curves of four specimens apparently showed two groups and exhibited a large difference in both stiffness and ultimate moment. It can be found that the setting of the stiffening rib in the tension zone acted as an obvious role than that of in the compression zone, as shown in Table 8. That was due to the altered form of the yield line of the tensile angle. For the unstiffened top angle joints including specimens NT and AR1, the yield lines were along the width orientation, whereas the yield lines of stiffened joints including specimens AR and AR2 were two circular arcs around the bolt hole.

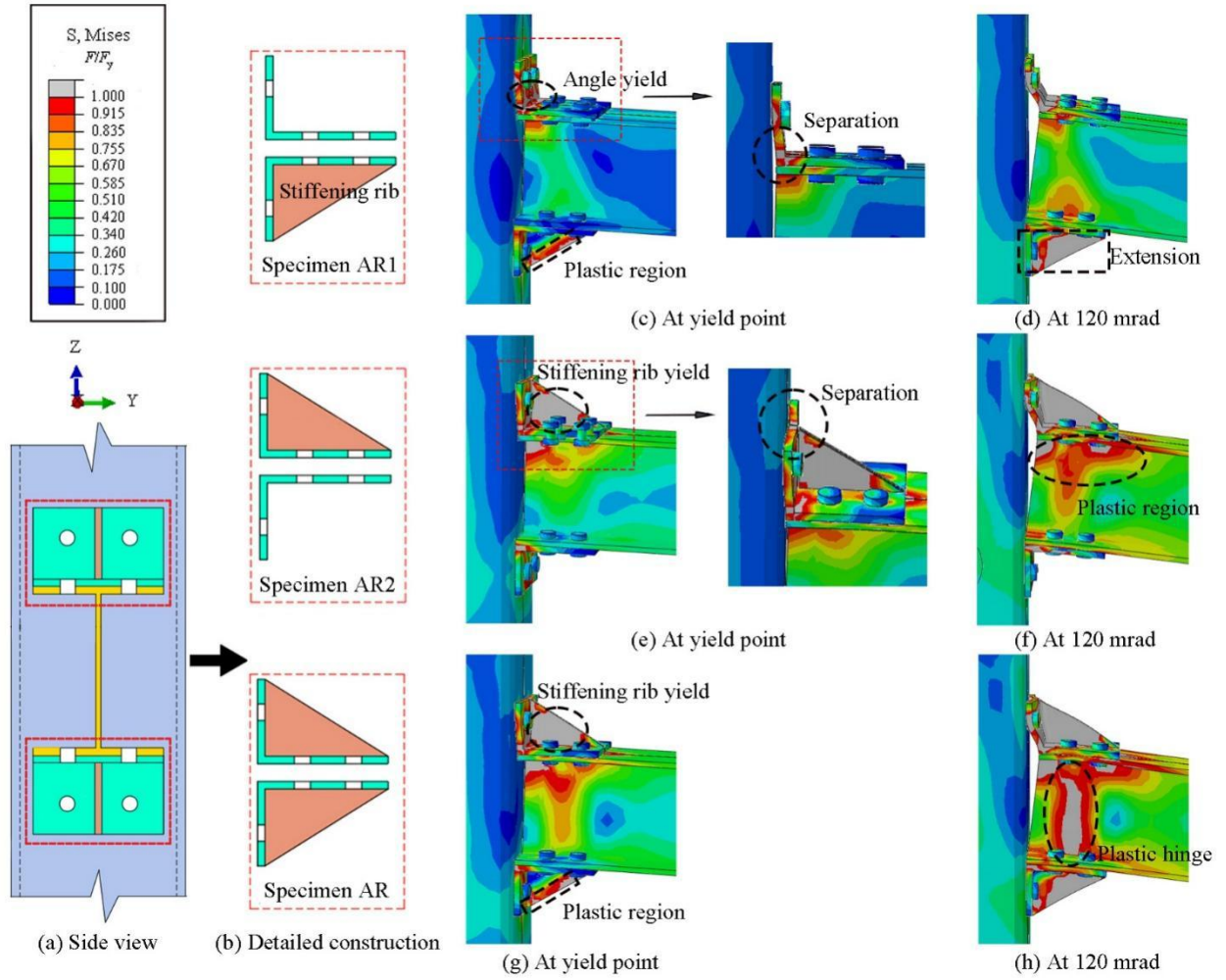


Fig. 11 Constructional details and deformation of joints with different stiffening rib position

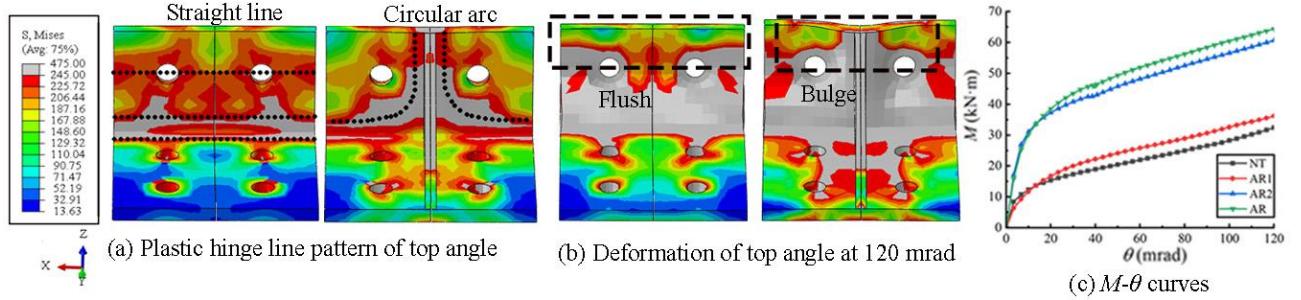


Fig. 12 Deformation of angles and $M-\theta$ curves

Table 8

Main results of joints with different stiffening rib position

Specimens	Initial stiffness ($\text{kN}\cdot\text{m}/\text{mrad}$)	Yield moment ($\text{kN}\cdot\text{m}$)	Ultimate moment ($\text{kN}\cdot\text{m}$)
NT	3.077	13.18	32.42
AR1	2.895	15.14	36.23
AR2	4.874	35.59	60.69
AR	4.711	38.14	64.39

3.2.2. Effect of stiffening rib thickness

In this section, the effect of stiffening rib thickness on bearing capacity of the joints was investigated. Fig. 13 showed the deformation of the joints with different stiffening rib thickness at the yield point and 120 mrad, respectively. The specimen ART1 with thickness of 4 mm was top stiffening rib yielding completely first and the top angle did not deform significantly. The column and I-beam remained in elastic stage. Specimen ART2 with thickness of 6 mm was the same as stiffening rib yielding completely first. At this time the angle still had no significant deformation, only in the vertical middle of the flange there

was a bending forward tendency. Specimen ART3 with thickness of 10 mm was the top angle and stiffening rib yielding almost simultaneously. A clear plastic region appeared at the end of the beam. Specimen ART4 with thickness of 12 mm was top angle yielding first with plastic region formed at the stiffening rib. The plastic region at the beam end was further enlarged. In all four specimens the angle yield pattern was two circular arcs around bolt holes.

As the loading continued, the angle, stiffening rib, and beam underwent bending deformation. It was found that the stiffness of the stiffening rib increased as the thickness growth, resulting in a decrease tendency of deformation, and the top angle web bent upward, creating a gap with the beam. Moreover, the yielding of beams increased significantly. Of 4 mm, the beam showed hardly any deformation, and of 10 and 12 mm, the beam generated a plastic hinge.

Fig. 14 showed the $M-\theta$ curves of the joints with different thicknesses of stiffening ribs. It can be seen that the thickness of the stiffening rib had little effect on the initial stiffness, but increased the yield moment, yield angle rotation and ultimate moment. This was due to the expansion of the angle yield region, which resulted in the translation of the yield line towards the vertical middle of the angle flange, increasing the effective length.

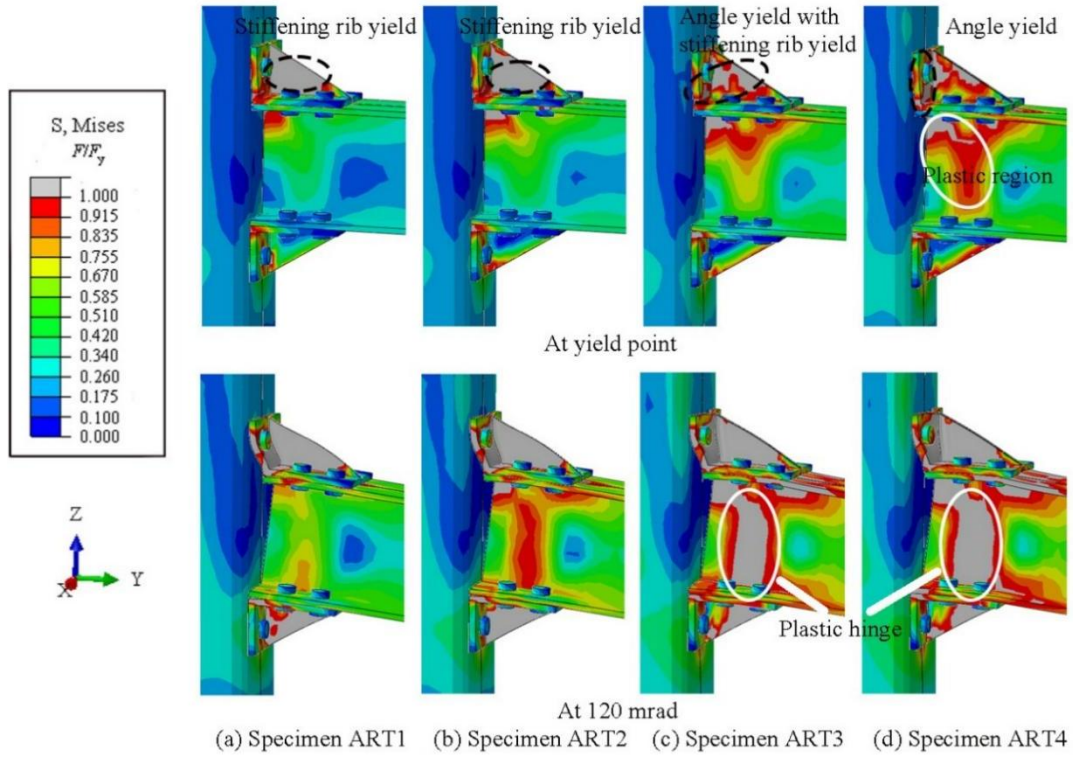
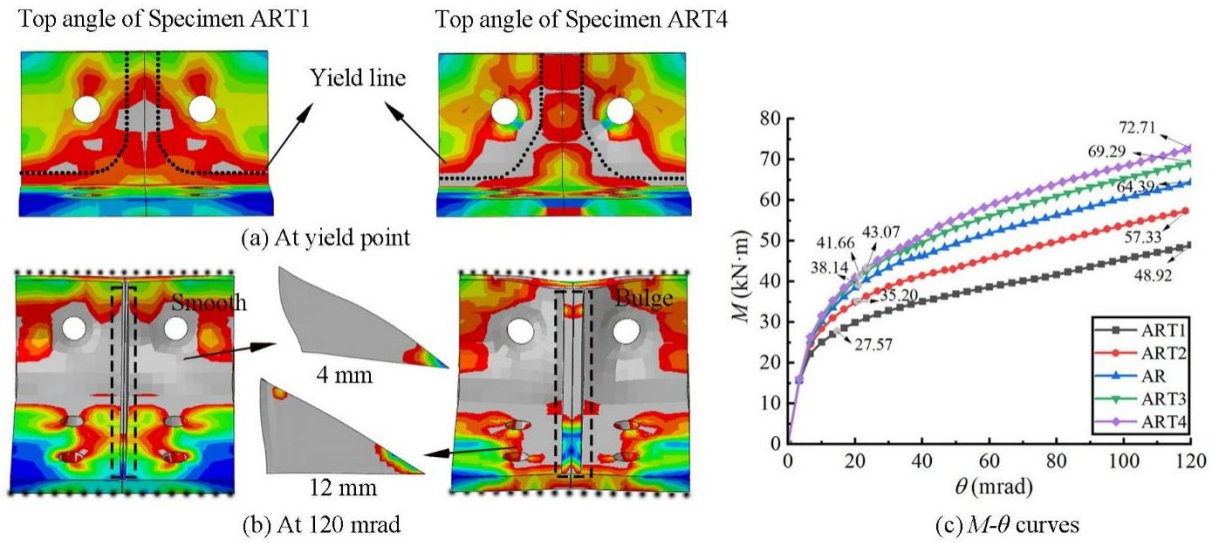
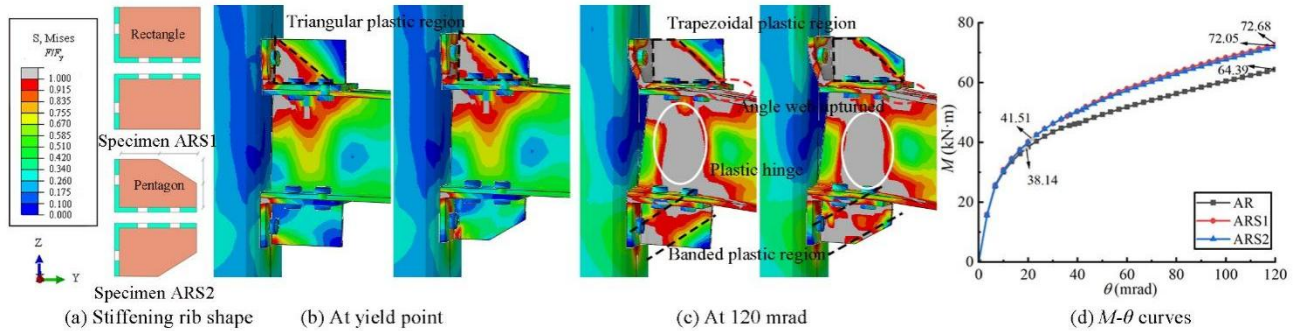


Fig. 13 Deformation of joints with different stiffening rib thicknesses

Fig. 14 Deformation of top angles and $M-\theta$ curvesFig. 15 Deformation and $M-\theta$ curves of joints with different stiffening rib shape

3.2.3. Effect of stiffening rib shape

In this section, the effect of stiffening rib shape was investigated, including specimen ARS1 with rectangular rib and specimen ARS2 with pentagonal rib, as shown in Fig. 15(a).

Similar to the specimen AR with triangular rib, the specimen ARS1 and ARS2 were top stiffening rib yielded before the angle. As the joints reached at

the yield point, the top angle had the same deformation and the top stiffening rib showed a triangular yield region. The seat stiffening rib was almost in an elastic state.

As loading continued, the beam showed significant bending deformation and formed plastic hinges. At 120 mrad, compared to specimen AR, the stiffening rib of specimens ARS1 and ARS2 had less deformation and almost

the same plastic region with a trapezium. The seat stiffening rib remained in its original shape and the plastic region was ribbon-like, as shown in Fig. 15(b) and (c).

There was no effect on the initial stiffness of joints by changing the stiffening rib shape. The $M-\theta$ curves of specimens ARS1 and ARS2 were shown in Fig. 15(d) almost coincided and revealed the same mechanical relationship. Moreover, the stress area of the stiffening ribs was increased, which led to an increase in both yield strength and ultimate moment.

3.2.4. Effect of stiffening rib number

In this section, two specimens were established to analyse the effect of the number of stiffening ribs, i.e., specimen ARN1 with two stiffening ribs arranged on each side of the angle and specimen ARN2 with three stiffening ribs arranged on each side and the middle of the angle, as shown in Fig. 16(a).

The specimen ARN1 was top angle yielding before other parts and the plastic region was mainly concentrated on the top angle when the joint reached at the yield point. Under the action of the stiffening rib, the flange and web of the angle bent along the vertical centreline and separated from the column and

beam, respectively, meanwhile the stiffening rib bent inward. The seat angle and stiffening rib produced no deformation. When the angle yielded, the yield line pattern changed to a straight line through the bolt hole and a circular arc around the bolt hole.

The specimen ARN2 was beam yielding first and produced a plastic hinge, which was different from all the joints aforementioned. The top and seat angles and stiffening ribs did not develop any deformation, but formed different plastic regions respectively.

As the loading continued, the top angle flange of specimen ARN1 was bending forward on two sides, and the web was bending upward on two sides, characterized as high sides and low centre, while the top stiffening ribs were concave to the inside. The top angle flange of specimen ARN2 were bending forward in the middle and on both sides, with the shape of a wave, and the web were bending upward to different extents. The seat angle of specimens ARN1 and ARN2 were bending downward and the seat stiffening ribs of specimen ARN1 produced significant outward bending deformations than specimen ARN2. Compared to specimen ARN1, the specimen ARN2's beam produced larger bending deformation and plastic hinge.

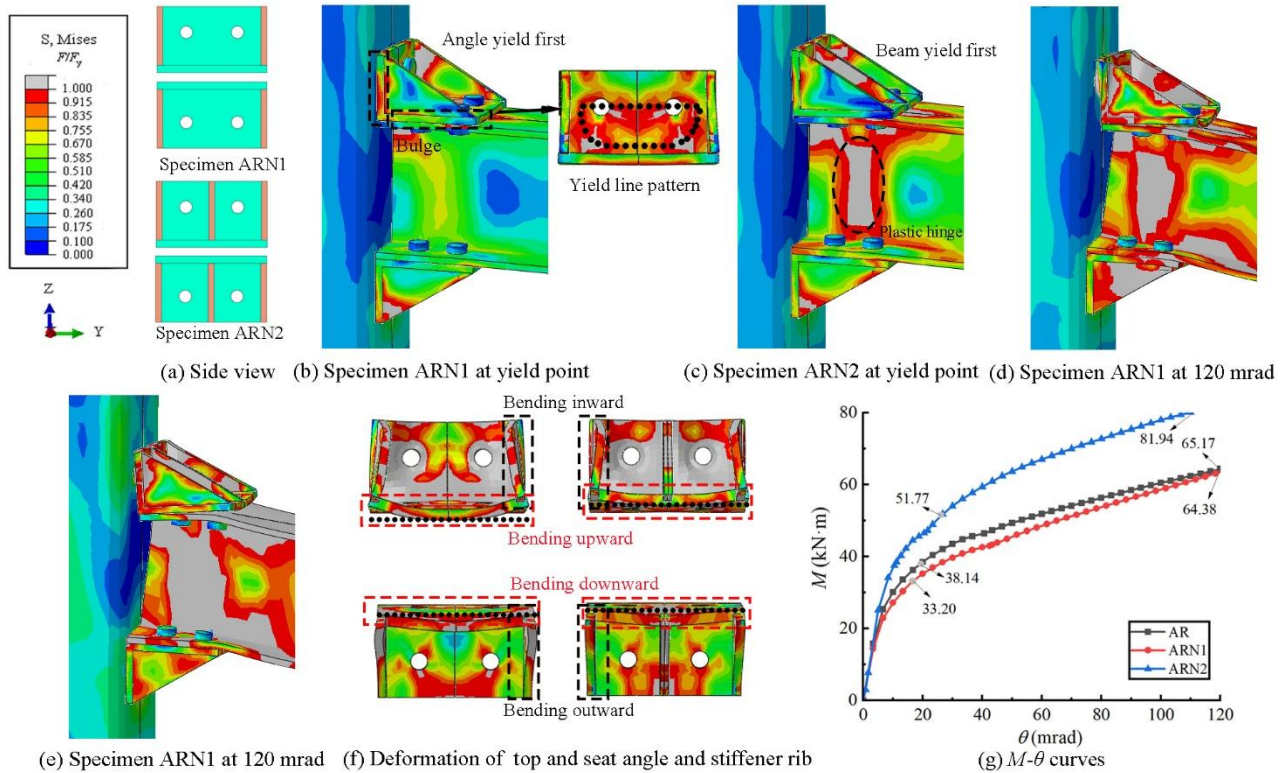


Fig. 16 Deformation and $M-\theta$ curves of joints with different stiffening rib number

3.3. Effect of double web angles

In this section, the effect of the double web angles on the bending performance of the joints was investigated. Four specimens were conducted

with web angles or stiffened web angles, respectively. The web angle was the same size as the top and seat angles, and the detailed construction was shown in Fig. 17.

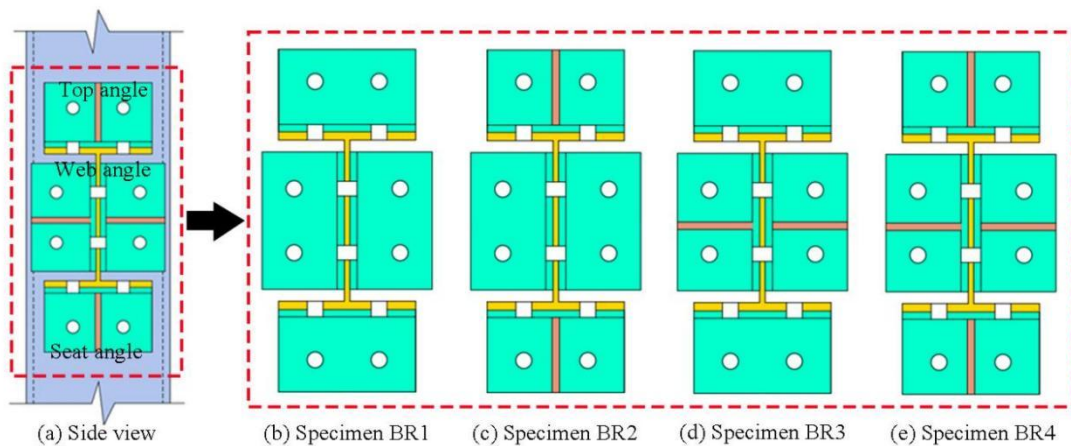


Fig. 17 Detailed constructions of double web angles joints

From Fig. 18, it can be seen that when the joints yielded, the columns and beams were still in the elastic stage. The force and deformation of the joints were mainly borne by the angles and stiffening ribs. For unstiffened web angle specimens BR1 and BR2 and the stiffened web angle specimens BR3 and BR4, the deformation and stress distribution of the top and seat angles were similar to those of without web angle joints, and the pattern of the yield lines was not changed. The web angle joints formed plastic hinges only in the tension zone of the flange, and the difference was in the form of the yield line. For unstiffened web angle joint specimen BR1, the yield line was linear, while a circular arc for the stiffened web angle.

Fig. 19 showed the joints at 120 mrad. The flange of the web angle in the tension zone was obviously detached from the column, while the flange in the

compression zone was adhered to the column with the web angle twisted. Due to the top and seat stiffening rib, the deformation and plastic region of the web stiffening ribs of specimen BR4 were smaller than that of specimen BR2. However, the beams were on the contrary, the beam flange of specimen BR1 was flat with almost no deformation, the specimens BR2 and BR3 produced bending deformation, and the plastic hinge region of specimen BR3 was larger, and even local buckling occurred at specimen BR4.

Fig. 20 and Table 9 showed the $M-\theta$ curves and main results of the web angle joints. It can be seen that the web angle, by assuming the forces and deformations, resulted in an increase in both the yield strength and ultimate moment of the joints.

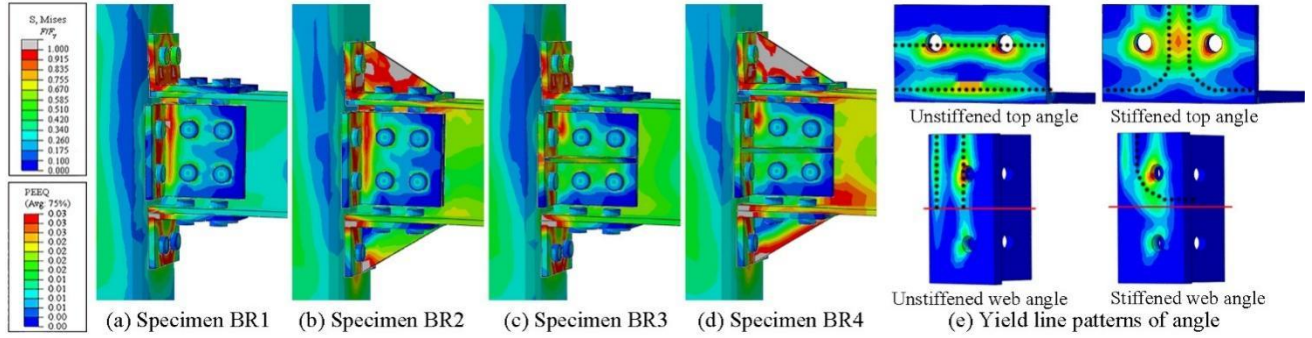


Fig. 18 Deformation of joints at yield point with double web angles

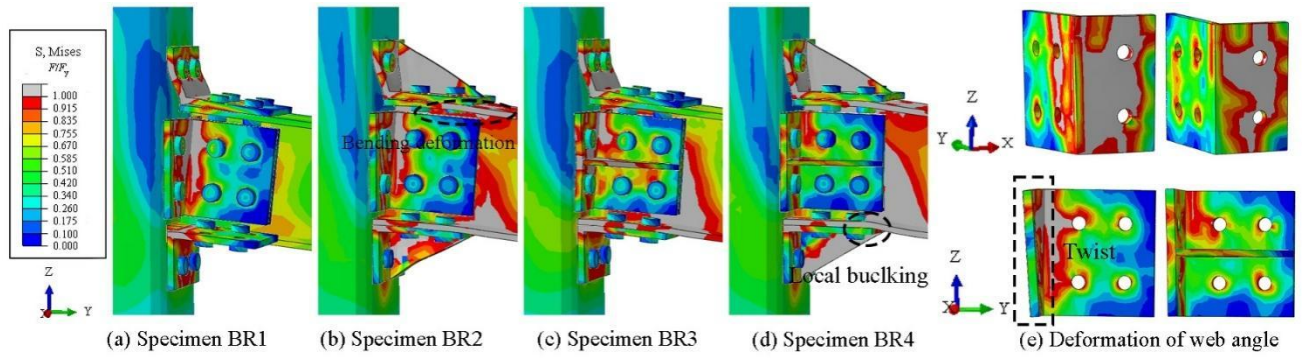


Fig. 19 Deformation of joints at 120 mrad with double web angles

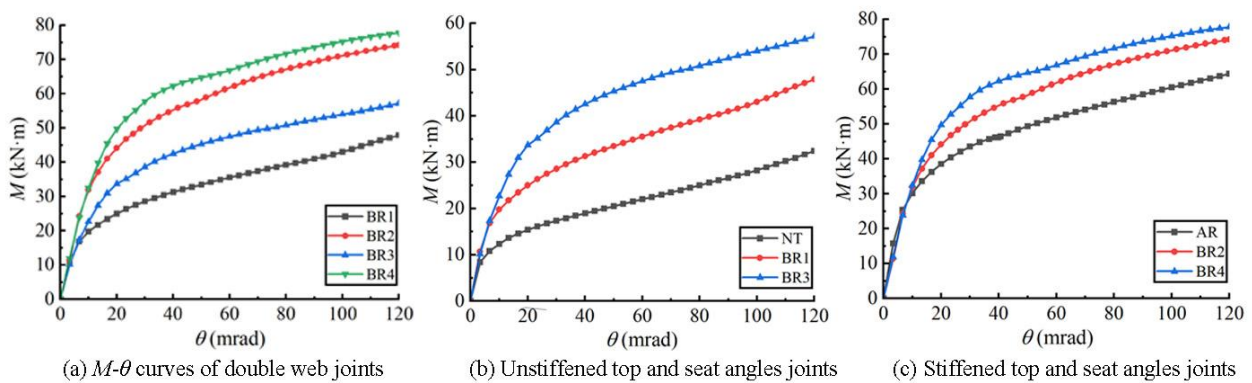


Fig. 20 $M-\theta$ curves of joints with double web angles joints

Table 9

Main results of joints with double web angles joints

Specimens	Initial stiffness (kN·m/mrad)	Yield moment (kN·m)	Ultimate moment (kN·m)
BR1	3.255	24.08	47.85
BR2	4.304	48.53	74.03
BR3	3.171	35.84	57.18
BR4	4.395	59.49	77.83

4. Calculation of bending capacity of angle joints

The mechanical performance of combined bolted angle beam to HSSC connection joint under monotonic loading was revealed by means of FEM in terms of stress distribution, deformation and bearing capacity. In order to be applied in the design work, the calculation method was proposed in this section. The yield line theory was widely used to calculate the bending capacity, which was recommended by Eurocode 3 and P398 [36].

The yield line models and the accompanying design equations proposed in the following subsections were developed and validated within the parameter ranges investigated in the numerical parametric study. The applicability of this method is therefore bounded by the following limits, which define the database

from which it was derived:

(1) Angle geometry: Leg length of 140 mm × 90 mm; thickness $t_a = 8\text{mm}$ –14 mm.

(2) Bolt layout: Two bolts in a vertical line in the outstanding leg; bolt gauge $g = 70\text{ mm}$; vertical distance from beam flange $p = 45\text{mm}$ –65 mm.

(3) Stiffener geometry: Triangular, rectangular, or pentagonal shape attached to the angle leg; stiffener thickness $t_s = 4\text{mm}$ –12 mm.

(4) Material: Structural steel with nominal yield strength $f_y = 245\text{MPa}$ –500 MPa, exhibiting ductile behavior.

The equations calculate the characteristic bending resistance M_{Rk} of the joint components. For design purposes, this resistance must be divided by the appropriate partial safety factor γ_{M2} as stipulated by Eurocode 3-1-8 [37] to obtain the design bending resistance M_{Rd} .

4.1. Unstiffened top and seat angles joints

For unstiffened top and seat angles joints, two yield lines were generated when the angle yielded. One yield line was at the root of the flange and the other yield line passed through the bolt hole. The tensile capacity of the angle flange yielded can be calculated using follows:

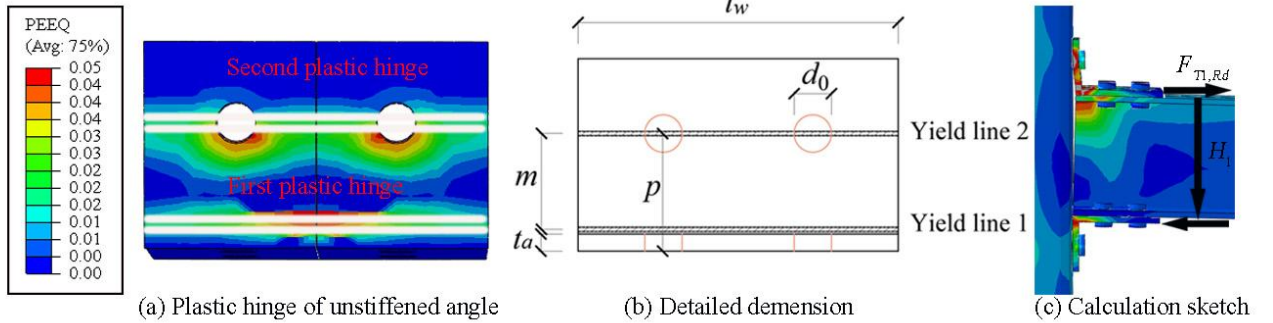


Fig. 21 Simplified diagrams for calculating unstiffened angle yield capacity

Table 10

Comparison of calculation and FEM of unstiffened angle joints

Specimens	t_a	f_y	Σl_{eff}	p	m	M_{cal}	M_{FEM}	M_{cal}/M_{FEM}
NT	10	245	265	55	45	15.44	13.18	1.171
NT1	8	245	265	55	47	9.28	7.2	1.289
NT2	12	245	265	55	43	23.70	19.54	1.213
NT3	14	245	265	55	41	34.45	28.74	1.199
NY1	10	355	265	55	45	22.37	19.26	1.161
NY2	10	390	265	55	45	24.57	21.5	1.143
NY3	10	420	265	55	45	26.46	23.45	1.128
NY4	10	460	265	55	45	28.99	25.12	1.154
NY5	10	500	265	55	45	31.51	26.66	1.182
NP1	10	245	265	45	35	19.85	18.05	1.110
NP2	10	245	265	50	40	17.37	15.9	1.092
NP3	10	245	265	60	50	13.89	12.55	1.107
NP4	10	245	265	65	55	12.63	11.79	1.071
Mean								1.155
Standard deviation								0.056

The predictive capability of Eqs. (1)–(5) is evaluated by direct comparison with the FEM yield moments M_{FEM} from the parametric study, as summarized in Table 10. No empirical calibration factors are applied to the FEM results. The calculated yield moments M_{cal} show a consistent, slight overestimation compared to the FEM benchmarks. The mean ratio M_{cal}/M_{FEM} for all 13 unstiffened specimens is 1.155 with a standard deviation of 0.057, indicating that the proposed yield line model predicts, on average, a yield strength approximately 15.5% higher than the finite element simulation.

This conservative bias is rationally explained by the idealizations inherent

$$F_{T1,Rd} = \frac{2M_{pl1,Rd}}{m} \quad (1)$$

$$M_{pl1,Rd} = 0.25 \sum l_{eff,un} t_a^2 f_y \quad (2)$$

$$m = p - t_a \quad (3)$$

$$\sum l_{eff,ns} = l_{eff,1} + l_{eff,2} = 2t_w - 2d_0 \quad (4)$$

where, t_a was the thickness of angle flange and web, t_w was the width of angle, f_y was the angle steel yield strength, d_0 was the diameter of bolt hole, and m was the distance between two yield lines, as shown in Fig. 21. $M_{pl1,Rd}$ was each yield line by the external moment and $\sum l_{eff,un}$ was the sum of effective length of unstiffened angle yield lines.

Therefore, the characteristic bending resistance of an unstiffened angle joint component can be calculated as:

$$M_{T1,Rk} = F_{T1,Rk} H_1 \quad (5)$$

$$M_{T1,Rd} = M_{T1,Rk} / \gamma_{M2}$$

in the yield line theory. The analytical model assumes the formation of discrete, perfect plastic hinges, whereas the FEM captures a more realistic gradual spread of plasticity, stress concentrations around bolt holes, and detailed deformation compatibility. The close agreement across a wide range of independent parameters demonstrates the robustness and predictive validity of the formulation within the investigated parameter space. The consistent overestimation renders the method inherently conservative for strength prediction.

4.2. Stiffened top and seat angles joints

For stiffened top and seat angles joints, the pattern of yield lines was two circular arcs as the stiffened angle yielded. The tensile capacity of the stiffened angle flange yielded can be calculated using follows:

$$F_{T2,Rd} = \frac{2M_{pl2,Rd}}{m_1} \quad (6)$$

$$M_{pl2,Rd} = 0.25 \sum l_{eff,st} t_a^2 f_y \quad (7)$$

$$m_1 = p - t_a \quad (8)$$

$$m_2 = 0.25g + 0.6t_s \quad (9)$$

$$\sum l_{eff,st} = (6m_2 - 0.625n_2 + n_1) * 2 \quad (10)$$

for specimen ARN1:

$$\sum l_{eff,st} = (6n_2 - 0.625m_2) * 2 + g - d_0 \quad (11)$$

where, n_1 and n_2 were the distance from the bolt hole to the free edge of the angle, respectively. m_1 and m_2 were the distance from the bolt hole to the yield line, respectively. t_s was the thickness of stiffening rib, g was the gauge of the

bolt, as shown in Fig. 22. $M_{pl2,Rd}$ was each yield line by the external moment and $\sum l_{eff,st}$ was the sum of effective length of stiffened angle yield lines.

Therefore, the yield bending capacity of stiffened angle joints can be calculated using the following formula:

$$\begin{aligned} M_{T2,Rk} &= F_{T2,Rk} H_1 \\ M_{T2,Rd} &= M_{T2,Rk} / \gamma_{M2} \end{aligned} \quad (12)$$

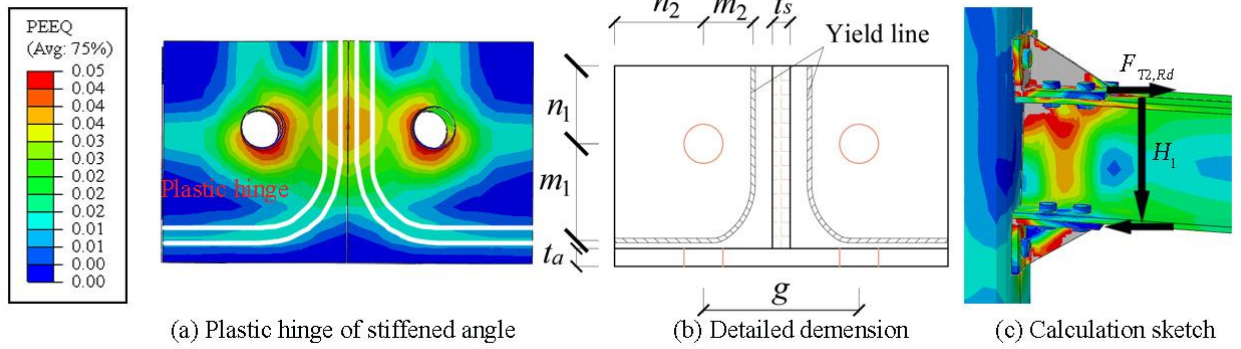


Fig. 22 Simplified diagrams for calculating stiffened angle yield capacity

Table 11

Comparison of calculation and FEM of stiffened angle joints

Specimens	n_1	n_2	m_1	m_2	t_s	g	$l_{eff,st}$	M_{cal}	M_{FEM}	M_{cal}/M_{FEM}
AR	35	40	45	22.3	8	70	287.6	33.51	38.14	0.879
ART1	35	40	45	19.9	4	70	258.8	30.15	27.57	1.094
ART2	35	40	45	21.1	6	70	273.2	31.83	35.2	0.904
ART3	35	40	45	23.5	10	70	302	35.19	41.66	0.845
ART4	35	40	45	24.7	12	70	316.4	36.86	43.07	0.856
ARS1	35	40	45	22.3	8	70	287.6	33.51	41.51	0.807
ARS2	35	40	45	22.3	8	70	287.6	33.51	41.51	0.807
ARN1	35	35	45	24.8	8	70	288.8	33.65	33.20	1.014
ARN2								beam yield	51.77	
Mean										0.901
Standard deviation										0.096

4.3. Double web angles joints

For double web angles joints, the calculation of the bending capacity can be carried out directly on the basis of the unstiffened top and seat angles joints or stiffened top and seat angles joints. The pattern of the web angle plastic hinge was the same as that of the top and seat angles, with the difference that the plastic hinge was generated only in the tension zone. The tensile capacity of the web angle flange yielded can be calculated using follows:

$$F_{T3,Rd} = \frac{2M_{pl3,Rd}}{m_x} \quad (13)$$

$$M_{pl3,Rd} = 0.25 \sum l_{eff,web} t_a^2 f_y \quad (14)$$

$$m_x = p - t_a \quad (15)$$

$$m = 0.25g + 0.6t_s \quad (16)$$

$$\sum l_{eff,web} = \begin{cases} \sum l_{eff1,web} = t_w - d_0 & \text{unstiffened web angle} \\ \sum l_{eff2,web} = 6m - 0.625e + e_x & \text{stiffened web angle} \end{cases} \quad (17)$$

The calculations given by Eqs. (6)-(12) were shown in Table 11 and compared with the results obtained by FEMs in Section 3. It was clear that the majority of calculated values are 10%-20% lower than the FEM results. This indicates that the proposed model for stiffened angles provides a built-in safety margin for design within the studied parameter range.

where, $M_{pl3,Rd}$ was each yield line by the external moment and $\sum l_{eff,web}$ was the sum of effective length of double web angles yield lines. Other parameters were shown in Fig. 23.

Therefore, the yield bending capacity of web angle joints can be calculated using the following formula:

$$\begin{aligned} M_{T3,Rk} &= F_{T(1,2),Rk} H_1 + 2F_{T3,Rk} H_2 \\ M_{T3,Rd} &= M_{T3,Rk} / \gamma_{M2} \end{aligned} \quad (18)$$

where, $F_{T(1,2),Rd}$ indicated that the unstiffened top and seat angles joints were taken as $F_{T1,Rd}$ while the stiffened top and seat angles joints were taken as $F_{T2,Rd}$. H_1 and H_2 were the distance between the rotation centre of the tension zone and the compression zone of top angle and web angle, respectively.

The calculations given by Eqs. (13)-(18) were shown in Table 12 and compared with the results obtained by FEMs in Section 3. It was observed that the majority of computational errors fall within the range of 10%. In conclusion, these equations were suitable for calculating the bending capacity of double web angles joints.

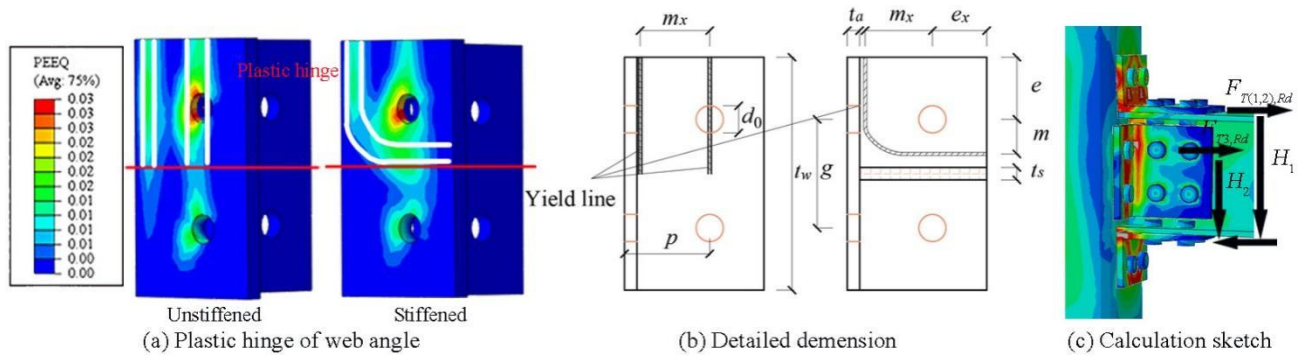


Fig. 23 Simplified diagrams for calculating double web angles yield capacity

Table 12

Comparison of calculation and FEM of double web angles joints

Specimens	e_x	e	m_x	m	t_s	g	$l_{eff,web}$	M_{cal}	M_{FEM}	M_{cal}/M_{FEM}
BR1	35	40	45			70	132.5	25.68	24.08	1.066
BR2	35	40	45			70	132.5	43.75	48.53	0.902
BR3	35	40	45	22.3	8	70	143.8	37.67	35.84	1.051
BR4	35	40	45	22.3	8	70	143.8	55.74	59.49	0.937
Mean										0.989
Standard deviation										0.071

5. Conclusions

In this paper, the mechanical performance of the combined bolted angle HSSC to I-beam connection joints was carried out by the verified FEM, and the impact of parameters on the bending capacity of the angle was investigated. The yield capacity had been calculated by applying the yield line theory. Based on the research in this paper, the following conclusions can be drawn:

(1) The unstiffened top angle generated three yield lines in tension bending, all of which were straight, located at the root of the web, at the root of the flange and through the bolt holes, respectively. The unstiffened seat angle generated only one yield line at the rear of web.

(2) Increasing the thickness and strength of the angle, as well as decreasing the bolt vertical distance can improve the bearing capacity of the joints, with changing the flange thickness had the most significant effect. Adjusting the flange thickness and bolt vertical distance can change the initial stiffness of the joints, attributed to changing the angle cross sectional shape.

(3) The initial stiffness and bearing capacity of the joints can be significantly increased by setting stiffening ribs due to the change in the yield line pattern of the angle, which effectively increased the effective length of the yield line and generated a plastic hinge in the beam. Compared to triangular stiffeners, rectangular and pentagonal ribs increased the bearing capacity by enlarging the force region.

(4) The thickness of the stiffening ribs had a great influence on the deformation of the joints. The deformation of 4 mm stiffening ribs was large while the beam did not deform and the deformation of 12 mm stiffening ribs was small but the beam generated plastic hinge. Setting the stiffening ribs on angle sides reduced the bearing capacity, and setting three stiffening ribs caused the beam to yield before the angle.

(5) For web angle joints, the forces and deformations of top and seat angles were the same as in joints without web angles. The web angle underwent torsional deformation and the yield line pattern was the same as that of the top angle but formed only in the tension zone.

(6) Based on the yield line theory, design equations were proposed to calculate the characteristic bending capacity of unstiffened, stiffened, and web angle joints. The equations demonstrate predictive capability when directly compared against FEM results across a wide parameter range, showing a consistent and conservative bias without the need for empirical calibration. The calculated values show satisfactory agreement with numerical benchmarks, with errors generally within an acceptable range for practical design.

(7) The design equations and conclusions presented in this study are primarily derived from FEM parametric analyses. While the FEM methodology was carefully validated against physical tests for the core connection mechanics, direct experimental validation of the through-core bolted HSSC angle joint under monotonic and cyclic loading remains an essential next step. Furthermore, the applicability of the yield line patterns and formulas is confined to the

parameter ranges investigated. Future work should encompass experimental testing to confirm the predictions, investigation under cyclic loading to evaluate seismic performance metrics, and expansion of the parametric scope to include a wider variety of geometric configurations.

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STRUCTURAL PERFORMANCE AND DESIGN OF HYBRID STAINLESS STEEL GIRDERS UNDER LOCALISED FORCES

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ABSTRACT

A novel generation of hybrid stainless steel girders, featuring high-strength stainless steel flanges and conventional stainless steel webs, has been developed and is increasingly used in marine structures. This hybrid configuration provides enhanced flexural capacity while simultaneously reducing material consumption and structural weight compared with conventional homogeneous girders. However, no established design rules currently exist for such hybrid steel girders. As a result, the resistance and design of such hybrid stainless steel girders under localised forces should be carefully evaluated. This study aims to fill the gap by expanding on previous experimental work and proposing simplified design methods for predicting the resistance of hybrid stainless steel girders under localised forces using numerical methods. A robust finite element (FE) model, previously developed and validated by the authors, was utilized to conduct an extensive parametric study, generating a total of 1248 FE results. This study focused on key parameters, including the bearing width ratio, web aspect ratio, web slenderness, and local geometric imperfection. The numerical results were analysed to assess the reliability of existing design guidelines, namely GB 50017-2017, EN 1993-1-5, and ANSI/AISC 360-22. The analysis revealed that the predicted values from GB 50017-2017 were unconservative, with an average deviation of 16.4%. In contrast, EN 1993-1-5 (2006) and ANSI/AISC 360-22 (2022) provided conservative estimates, with average deviations of 44.7% and 19.8%, respectively. To address this issue, new design proposals were formulated, incorporating a modified coefficient within the framework of existing design rules.

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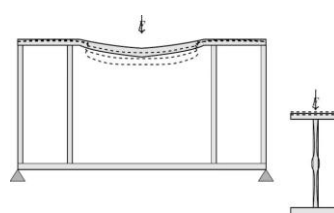
Hybrid stainless steel girders;
Localised force, numerical method;
New design proposals;
Parametric study

1. Introduction

A novel generation of hybrid stainless steel girders, featuring high-strength stainless steel flanges and conventional stainless steel webs, has been developed and is increasingly used in marine structures. Stainless steel is attractive for structural applications in aggressive marine environments [1]. This hybrid configuration offers clear structural advantages by providing enhanced flexural capacity while simultaneously reducing material consumption and structural weight compared with fully homogeneous girders [2]. However, hybrid-configured girders may experience buckling failure due to patch loading when exposed to moving loads, in which a concentrated force applied perpendicular to the flange leads to localized web failure [3-8], as illustrated in Fig. 1. The issue is especially common with crane girders and beam-column couplings. Therefore, a comprehensive assessment of the buckling behavior of hybrid stainless steel girders under localized bearing force is required.



(a) Steel girders used in marine structures



(b) Schematic diagram

Fig. 1 The practical application and loading cases of steel girders

The structural behavior of hybrid carbon steel members has been the subject of numerous investigations in the literature [9–13], revealing notable performance distinctions compared to their homogeneous counterparts. Nevertheless, studies on hybrid steel members incorporating conventional stainless steel remain relatively scarce. Lalthazuala and Singh [14–16] performed a series of numerical investigations to evaluate the structural performance and design behavior of hybrid-configured girders, with particular emphasis on bending, axial compression, and shear capacity. Based on their numerical results, refined design formulations tailored to hybrid members were proposed. In parallel, Bock et al. [17] carried out numerical analyses on hybrid stainless steel girders subjected to compression and bending. However, the validation of their numerical models was based exclusively on experimental results from homogeneous welded I-sections, rather than on specimens with hybrid material configurations. The lack of direct experimental evidence for

hybrid-configured girders therefore limits the reliability and general applicability.

To date, no studies have addressed the resistance of hybrid stainless steel girders under localized forces. While the authors' prior research [18-22] examined the structural behavior and design of homogeneous stainless steel girders, these investigations focused exclusively on patch loading and shear resistance in homogeneous members, which exhibit distinct characteristics compared to hybrid configurations. The variations in material composition, bonding mechanisms, and load distribution between hybrid and homogeneous girders highlight the need for a more comprehensive analysis of their performance under concentrated loads.

Furthermore, homogeneous girders are the main focus of the current design requirements for predicting girder resistance under localized loads [23-25]. Existing design standards, such as GB 50017 [26], EN 1993-1-5 [27], and ANSI/AISC 360-22 [28], are fundamentally developed based on the assumption of material homogeneity, which may not be fully applicable to hybrid stainless steel members. They do not consider the unique characteristics and performance variations inherent in hybrid-configured girders, which combine different material grades in their construction. Consequently, their applicability to hybrid-configured girders requires thorough evaluation in this study.

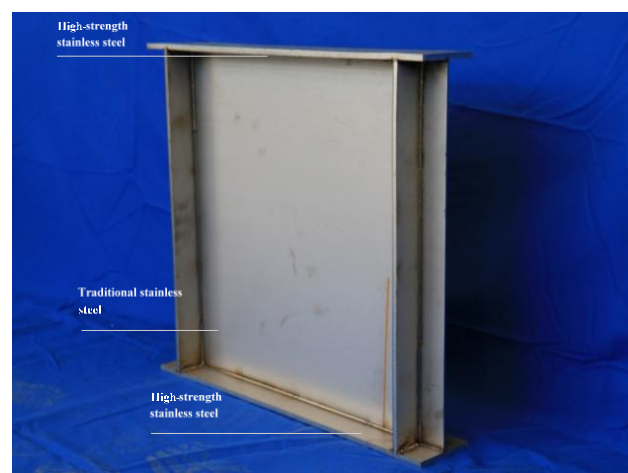


Fig. 2 Photograph of test specimens investigated in this study [29]

To address the aforementioned research gap, the present study builds upon

the authors' previous experimental investigations [29] and proposes simplified design approaches for evaluating the structural performance of hybrid stainless steel girders subjected to concentrated loading. The experimental specimens consist of high-strength stainless steel flanges combined with conventional stainless steel webs, as illustrated in Fig. 2. A previously developed and experimentally validated finite element (FE) model was subsequently employed to conduct an extensive parametric study, comprising a total of 1,248 numerical simulations. Through this comprehensive analysis, the influences of key parameters, including the bearing width ratio, web aspect ratio, web slenderness, and local geometric imperfections, were systematically investigated. Subsequently, the numerical predictions were systematically assessed to examine the suitability and predictive accuracy of the existing design provisions prescribed in GB 50017 [26], EN 1993-1-5 [27], and ANSI/AISC 360-22 [28]. Based on the observed discrepancies, calibration factors were introduced into the existing design formulations, and alternative design approaches were further proposed.

2. Numerical modelling

2.1. General

Numerical simulations using the ABAQUS program were performed to examine the structural response of hybrid-configured girders under localized bearing force [30]. The computational models underwent thorough validation through comparison with experimental data from the authors' prior studies [29],

incorporating considerations of material nonlinear behavior and initial geometric deviations. A detailed comparison of numerical predictions with experimental findings confirmed the accuracy of models in predicting the strength of such hybrid-configured girders. Further numerical modelling approaches can be found in the authors' recent studies [18,19].

2.2. Summary of test data for validation purposes

The authors' prior experimental program [29] involved testing 10 hybrid-configured girders under localized forces, as summarized in Table 1, with the data serving as a basis for subsequent validation. The flanges were fabricated from either QN1803 or S32001 high-strength stainless steel, while the webs were constructed using EN 1.4301 (S30408) or EN 1.4462 (S2205) conventional stainless steel.

A total of 22 tensile test specimens were extracted from specified locations on both the web and flange components, with testing procedures conducted in accordance with the British Standard for metallic material testing (ISO 6892-1) [31]. The corresponding tensile properties are summarized in Table 2. For the concentrated load testing, a 2000 kN MTS hydraulic press was utilized, as illustrated in Figs. 3 and 4. The investigation primarily examined failure behavior under an internal one-flange (IOF) loading condition applied at the beam's midspan. The girders were simply supported at both ends, with the concentrated force transmitted through a bearing plate. Displacement measurements were recorded using three LVDTs positioned at both ends and midpoint, as illustrated in Fig. 3.

Table 1
Summary of patch-loading test results reported by previous study [29]

Specimen ID	Flange material	Web material	L/mm	h_w/mm	a/mm	t_w/mm	t_f/mm	S_s/mm	w_0/mm	S_s/h_w	a/h_w	$P_{u,Test}/kN$
F1803W304-T3.0-ad1.0-S50-1	QN1803	EN 1.4301	600	500	500	2.95	7.77	50	0.5	0.10	1.0	99.7
F1803W304-T3.0-ad1.0-S50-2	QN1803	EN 1.4301	600	500	500	2.95	7.77	50	0.5	0.10	1.0	94.8
F1803W304-T3.0-ad1.0-S100-1	QN1803	EN 1.4301	600	500	500	2.94	7.75	100	0.3	0.20	1.0	108.6
F1803W304-T3.0-ad1.0-S100-2	QN1803	EN 1.4301	600	500	500	2.94	7.75	100	0.3	0.20	1.0	110.4
F32001W2205-T3.0-ad1.0-S50-1	S32001	EN 1.4462	600	500	500	3.00	7.78	50	1.0	0.10	1.0	164.2
F32001W2205-T3.0-ad1.0-S50-2	S32001	EN 1.4462	600	500	500	3.00	7.78	50	1.0	0.10	1.0	163.6
F32001W2205-T3.0-ad1.0-S100-1	S32001	EN 1.4462	600	500	500	3.00	7.75	100	0.6	0.20	1.0	173.9
F32001W2205-T3.0-ad1.0-S100-2	S32001	EN 1.4462	600	500	500	3.00	7.75	100	0.6	0.20	1.0	173.1
F32001W2205-T3.0-ad1.5-S100-1	S32001	EN 1.4462	850	500	750	2.98	7.76	100	1.2	0.20	1.5	167.9
F32001W2205-T3.0-ad1.5-S100-2	S32001	EN 1.4462	850	500	750	2.98	7.76	100	1.2	0.20	1.5	164.5

Table 2
Summary of material properties of hybrid stainless steel girders [20,24,37]

Grades	Coupon ID	E_0/GPa	$\sigma_{0.2}/MPa$	σ_u/MPa
QN1803	1803-T3-1	193.4	445.1	763.6
	1803-T3-2	191.3	437.5	749.1
	1803-T3-3	193.7	423.6	752.6
	1803-T8-1	187.1	415.6	703.5
	1803-T8-2	185.7	413.3	714.7
	1803-T8-3	182.4	406.9	708.8
S32001	32001-T3-1	187.9	475.1	674.6
	32001-T3-2	187.5	479.7	654.9
	32001-T3-3	180.8	476.9	666.8
	32001-T8-1	200.7	478.1	673.9
	32001-T8-2	191.1	481.5	680.8
	32001-T8-3	198.9	464.3	677.1
S30408	S30408-T3-1	187.5	306.7	716.5
	S30408-T3-2	180.2	300.7	714.3

EN 1.4301	S30408-T8-1	180.8	332.5	715.4
	S30408-T8-2	182.8	321.8	689.5
	S30408-T8-3	184.7	324.2	727.1
EN 1.4462	2205-T3-1	198.6	593.1	788.0
	2205-T3-2	202.2	598.5	795.4
	2205-T3-3	200.1	619.7	789.7
	2205-T8-1	202.7	559.1	776.7
	2205-T8-2	200.8	552.9	753.6

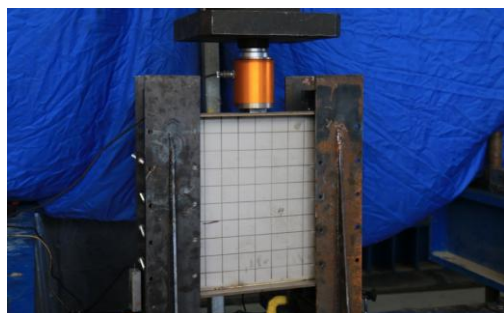


Fig. 3 Experimental setup from the authors [29]

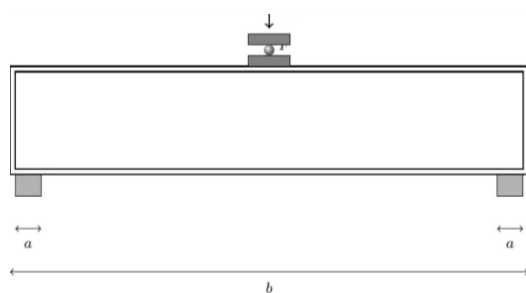


Fig. 4 Schematic diagram of the testing setup

Each girder specimen was tested following a two-stage loading protocol to fully exploit its structural capacity. In the first stage, the specimen was loaded in its as-fabricated orientation without any prior damage. Taking specimen F1803W304-T3.0-ad1.0-S50-1 as a representative example, the initial test was conducted until the peak load was attained. After completion of the first loading stage, the specimen was unloaded and prepared for a second test. For the subsequent test, the girder was inverted to reverse the loading configuration. Specifically, the flange that originally served as the bottom flange was repositioned at the top, while the flange previously subjected to loading was relocated to the bottom. Following this reconfiguration, the specimen was relabeled as F1803W304-T3.0-ad1.0-S50-2 and reloaded under the same experimental conditions. Additional information regarding the test setup, boundary conditions, loading scheme, and instrumentation can be found in Ref. [29].

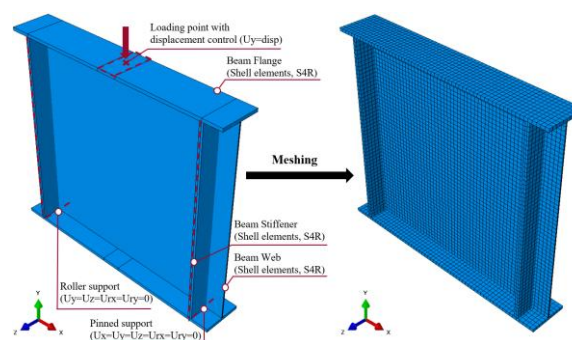
2.3. Summary of numerical modelling approaches

The yield strength, ultimate tensile strength, and elongation obtained from the coupon tests were systematically incorporated into the finite element (FE) modeling framework to ensure an accurate representation of the structural response under various loading conditions. Owing to its proven effectiveness in structural analyses where the plate thickness is significantly smaller than the in-plane dimensions, the S4R element, a four-node, quadrilateral shell element with reduced integration, was adopted for the numerical simulations [32–34]. This element type is particularly well suited for thin-plate applications, as it effectively mitigates shear locking while maintaining high computational efficiency. The nonlinear stress–strain relationship was first fitted using the Ramberg–Osgood model and then discretized into a multi-linear form for implementation in ABAQUS.

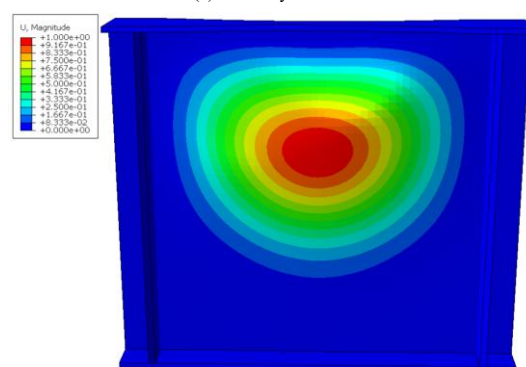
The finite element (FE) models were carefully developed to replicate the experimental loading conditions, allowing for a direct comparison between numerical predictions and experimental results. As illustrated in Fig. 5(a), a vertical displacement was imposed on a designated reference point, which was coupled to the bearing plates through coupling constraints to ensure realistic and consistent load transfer. The bearing plates were modeled as rigid bodies, with

all degrees of freedom constrained to the reference point to enforce a uniform displacement field [35,36]. In addition, Tie constraints were introduced between the girder ends and the stiffened blocks to prevent any relative displacement during loading [37–39]. This modeling approach enabled an accurate representation of the structural response of hybrid-configured girders subjected to localized bearing force.

Using finite element (FE) models, a buckling analysis was performed to assess how initial geometric imperfections affected structural stability [40–42]. The first-order eigenmode, depicted in Fig. 5(b), exhibited a strong agreement with the initial deformations measured during experimental testing, confirming its suitability for imperfection modelling. To integrate these imperfections into the numerical simulations, the *IMPERFECTION keyword was employed, utilizing the first eigenmode derived from the buckling analysis as the basis for the initial geometric imperfection. This mode was scaled to represent realistic deformation amplitudes, ensuring an accurate representation of the structural response under loading conditions [43].



(a) boundary condition and mesh size



(b) First-order buckling mode derived from FE

Fig. 5 FE modelling technique applied in this study [29]

A preliminary assessment of the residual stress indicated that its effect on the resistance of hybrid-configured girders subjected to localized bearing force is negligible. This can be attributed to the fact that the structural response under concentrated forces is predominantly governed by local bearing and web crippling mechanisms, for which the contribution of residual stress is relatively limited. This observation is consistent with the findings reported in previous studies [44,45], which similarly demonstrated that residual stresses exert an insignificant influence on the load-carrying capacity. In light of its marginal effect, the residual stress was therefore neglected in the finite element modeling adopted in this study.

In the present study, the welding configuration, heat input, and geometry of the hybrid and homogeneous girders are essentially identical, only the flange material differs. Consequently, the residual stress distribution in the web, where

patch loading resistance is primarily governed, is expected to be similar in both cases. Since patch loading behaviour is dominated by localized web yielding and crippling rather than global instability, its sensitivity to residual stresses is limited. To further address this concern, a representative critical case was reanalysed with a typical residual stress pattern introduced. The comparison between models with and without residual stresses shows only a marginal difference in the load–displacement response and ultimate capacity.

2.4. Modelling approach validation

To validate the finite element (FE) models, a comprehensive dataset comprising 10 experimental results from the authors' previous study [29] was employed. The validation was conducted by systematically comparing the numerical predictions with the experimental observations in terms of load–

displacement responses, local load-carrying capacity, and failure modes.

As summarized in Table 3, a detailed comparison was performed between the ultimate loads obtained from the experimental tests (F_{test}) and those predicted by the finite element analysis (F_{FE}). The results indicate a mean F_{test}/F_{FE} ratio of 0.988, accompanied by a low coefficient of variation (COV) of 0.015, demonstrating the high level of agreement between the experimental and numerical results. Furthermore, Fig. 6 compares the load–vertical deflection relationships derived from the experimental tests and the FE simulations, revealing a strong correlation over the entire loading range. In addition, Fig. 7 illustrates the comparison between the experimentally observed deformed shapes and those predicted by the FE models. The close agreement in both deformation patterns and failure characteristics further substantiates the capability of the FE models to accurately reproduce the structural behavior of hybrid-configured girders under localized bearing force.

Table 3

Comparison between experimentally and numerically predicted resistances [29]

Specimens	Flange	Specimens	$P_{u, Test}$ (kN)	$P_{u, FEA}$ (kN)	$P_{u, Test} / P_{u, FEA}$
F1803W304-T3.0-ad1.0-S50-1	QN1803	EN 1.4301	99.7	98.8	1.009
F1803W304-T3.0-ad1.0-S50-2	QN1803	EN 1.4301	94.8	98.8	0.96
F1803W304-T3.0-ad1.0-S100-1	QN1803	EN 1.4301	108.6	111.4	0.975
F1803W304-T3.0-ad1.0-S100-2	QN1803	EN 1.4301	110.4	111.4	0.991
F32001W2205-T3.0-ad1.0-S50-1	S32001	EN 1.4462	164.2	164.9	0.996
F32001W2205-T3.0-ad1.0-S50-2	S32001	EN 1.4462	163.6	164.9	0.992
F32001W2205-T3.0-ad1.0-S100-1	S32001	EN 1.4462	173.9	176.8	0.984
F32001W2205-T3.0-ad1.0-S100-2	S32001	EN 1.4462	173.1	176.8	0.979
F32001W2205-T3.0-ad1.5-S100-1	S32001	EN 1.4462	167.9	166.3	1.01
F32001W2205-T3.0-ad1.5-S100-2	S32001	EN 1.4462	164.5	166.3	0.989
Mean					0.988
CoV					0.015

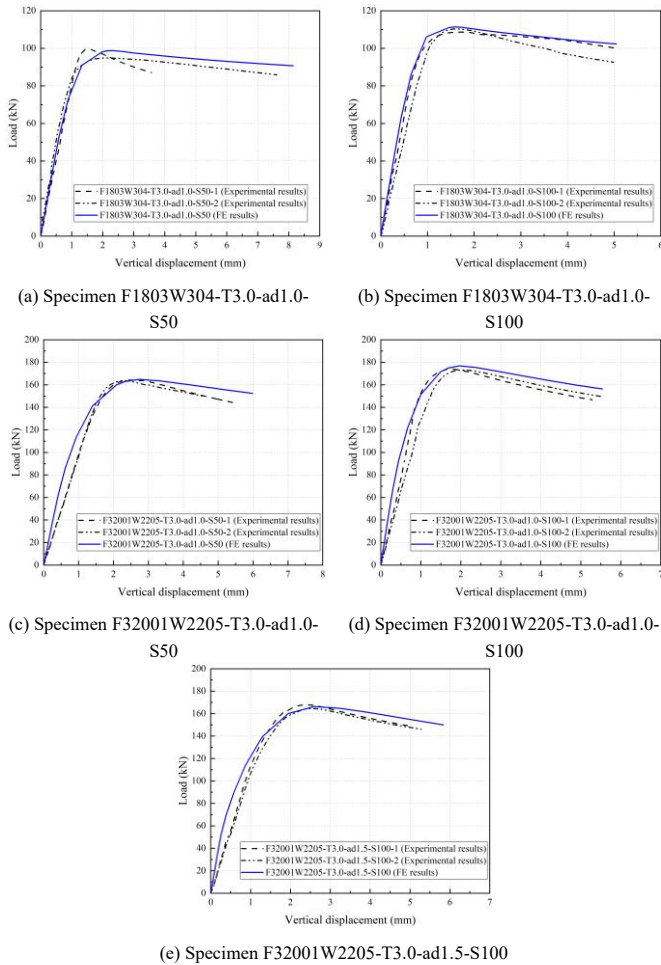


Fig. 6 Load-displacement graphs from the tests and FEA [29]

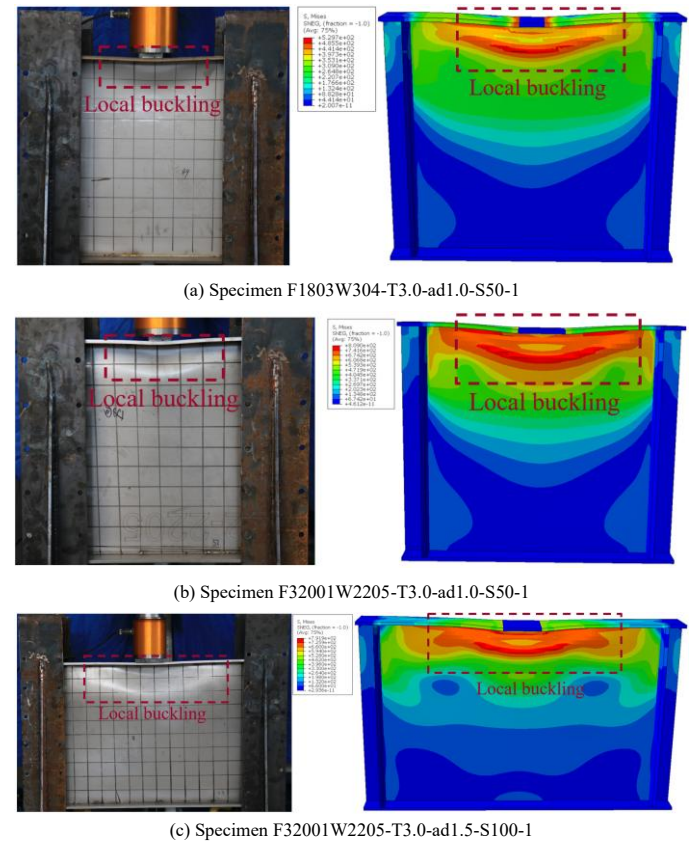


Fig. 7 Comparison of failure modes between the testing and FE models [29]

3. Parametric study

3.1. General

A comprehensive parametric study was carried out using a robust finite

element (FE) model that had been previously developed and extensively validated by the authors, resulting in a total of 1,248 numerical simulations. The material stress–strain relationships adopted in the FE models were derived from the authors' earlier experimental investigations [29]. Key material properties, including yield strength, ultimate tensile strength, and elastic modulus, are systematically summarized in Table 2.

Initially, the localized-loading resistance of hybrid and homogeneous welded girders was evaluated through a thorough comparative analysis. Following that, a parametric analysis was conducted to examine the key variables affecting these hybrid-configured girders' resistance. To assess their individual and combined effects on the behavior of the girder, important parameters such as the bearing width ratio (S_y/h_w), web slenderness ratio (h_w/t_w), local imperfection ratio (w_0/h_w), and web aspect ratio (a/h_w) were changed.

The ratio of bearing width to web height (S_y/h_w) was varied between 0.1 and 0.4 to examine the influence of load distribution characteristics within the bearing region. The web slenderness ratio (h_w/t_w), ranging from 90.9 to 200.0, was considered to assess the effects of web thickness and associated stability behavior on the load-carrying capacity. The web aspect ratio (a/h_w) was investigated within the range of 1.0 to 2.0 to evaluate its influence on the interaction between local bearing behavior and overall web deformation. In addition, the local imperfection ratio (w_0/h_w) was varied from 0.0008 to 0.044 to account for the effects of initial geometric imperfections. The adopted ranges of these governing parameters are summarized in Table 4.

Table 4
Details of the parametric analysis

Key parameters	Range
Web aspect ratio (a/h_w)	1.0, 1.1, 1.2, 1.3, 1.4, 1.5, 1.6, 1.7, 1.8, 1.9, 2.0
Web slenderness ratio (h_w/t_w)	90.9, 94.3, 98.0, 102.0, 106.4, 111.1, 116.3, 122.0, 128.2, 135.1, 142.9, 151.5, 161.3, 172.4, 185.2, 200.0
Local geometric imperfection ratio (w_0/h_w)	0.0008, 0.002, 0.0032, 0.044
Bearing width ratio (S_y/h_w)	0.10, 0.12, 0.14, 0.16, 0.18, 0.20, 0.22, 0.24, 0.26, 0.28, 0.30, 0.32, 0.34, 0.36, 0.38, 0.40

3.2. Comparison between hybrid and homogeneous girders

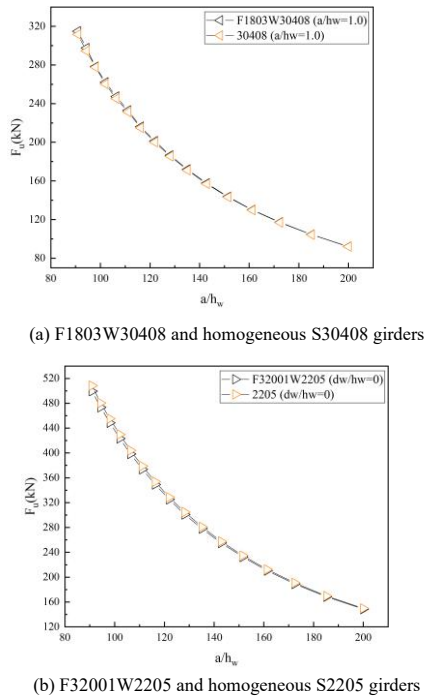


Fig. 8 Comparison between hybrid and homogeneous girders

A comparative evaluation of the local load-carrying capacity is presented in Fig. 8(a) and (b), illustrating the differences in structural performance between hybrid-configured girders and conventional homogeneous girders. Specifically, the analysis examines F1803W304 hybrid-configured girders, which feature flanges made of QN1803 and webs made of EN 1.4301 (S30408), as well as F32001W2205 hybrid-configured girders, constructed with flanges made of S32001 and webs made of EN 1.4301 (S30408). These are compared

to homogeneous girders composed entirely of EN 1.4462 (S2205). The objective is to assess whether hybrid-configured girders can achieve localized-loading resistance comparable to that of homogeneous girders while optimizing material use.

The results demonstrate that the local load-carrying capacity of the F1803W304 and F32001W2205 hybrid-configured girders is comparable to that of homogeneous EN 1.4301 (S30408) and EN 1.4462 (S2205) girders, respectively. This indicates that the use of QN1803 and S32001 as flange materials does not adversely affect the structural performance under localized bearing force conditions. Moreover, given the higher strength of QN1803 compared to EN 1.4301 (S30408), the moment resistance of hybrid-configured girders is expected to exceed that of homogeneous girders. These findings underscore the structural efficiency of hybrid-configured girders, which capitalize on the mechanical advantages of different materials to enhance performance while potentially reducing material costs and weight.

Both hybrid and homogeneous girders exhibit the same dominant failure modes under patch loading, characterized by initial web yielding followed by localized buckling or crushing. This indicates that the patch loading response is primarily governed by localized web behavior, including web crippling, local plastic deformation, and instability. In this loading scenario, the web acts as the principal load-resisting component. The flange mainly serves as a load-introduction element that transfers and distributes the applied patch load into the web. Moreover, the load transfer path from the patch load to the web is essentially identical in both hybrid and homogeneous girders. Although the high-strength flange in the hybrid configuration improves local stiffness, it does not fundamentally alter the deformation pattern, stress distribution, or governing failure mechanism of the web. As a result, both girder types exhibit comparable structural performance under patch loading.

3.3. Effect of local geometrical imperfection ratio (w_0/h_w)

The effect of the local geometric imperfection ratio (w_0/h_w) on the ultimate local load-carrying capacity of the F1803W304 and F32001W2205 hybrid-configured girders is illustrated in Figs. 9(a) and (b). Values of w_0/h_w ranging from 0.0008 to 0.0044 were considered for each web configuration. The results indicate that the variations in local load-carrying capacity induced by different imperfection amplitudes remain consistently below 1% for all investigated specimens. This limited sensitivity suggests that the localized bearing response of stainless steel plate girders is largely governed by material yielding and web crippling mechanisms, with initial local geometric imperfections playing a secondary role. In view of the negligible influence observed and to ensure numerical efficiency and consistency, a local geometric imperfection ratio of $w_0/h_w = 0.0008$ was therefore adopted in the subsequent parametric analyses.

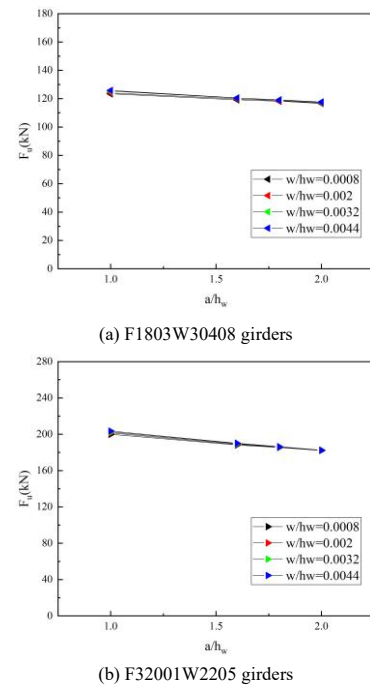


Fig. 9 Effect of initial imperfection on the ultimate strength

3.4. Effect of web aspect ratio (a/h_w)

The influence of the web aspect ratio (a/h_w) on the local load-carrying

capacity of the F1803W304 and F32001W2205 hybrid-configured girders is illustrated in Figs. 10(a) and (b), respectively. The parametric analysis covered a/h_w values ranging from 1.0 to 2.0 with an increment of 0.1. Typical failure modes observed in selected girder specimens are presented in Figs. 11 and 12. The results demonstrate that, for a given web aspect ratio, an increase in the bearing width ratio (S_s/h_w) leads to a pronounced enhancement in local load-carrying capacity. This trend reflects the beneficial effect of load dispersion over a wider bearing region, which delays the onset of localized web crippling. For instance, when $a/h_w = 1.0$, increasing the S_s/h_w ratio from 0.10 to 0.38 results in an increase in the ultimate local load-carrying capacity of 33.7% for the F1803W304 hybrid-configured girders and 31.7% for the F32001W2205 hybrid-configured girders.

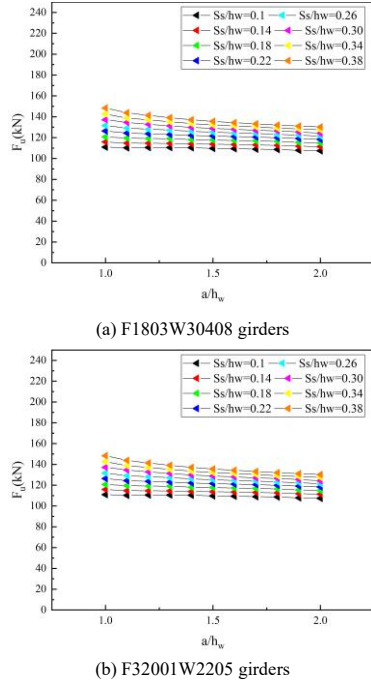


Fig. 10 Effect of a/h_w on the ultimate strength

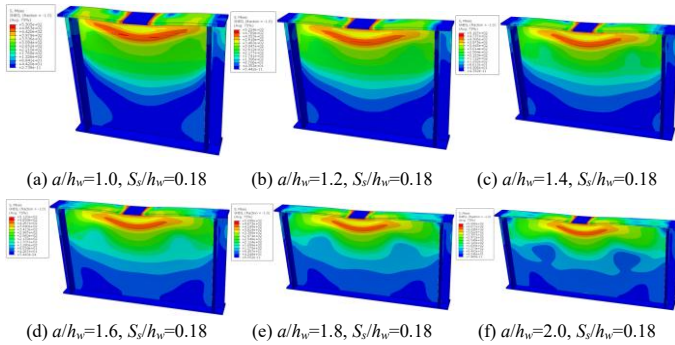


Fig. 11 Typical failure modes for F1803W30408 specimens with varying a/h_w

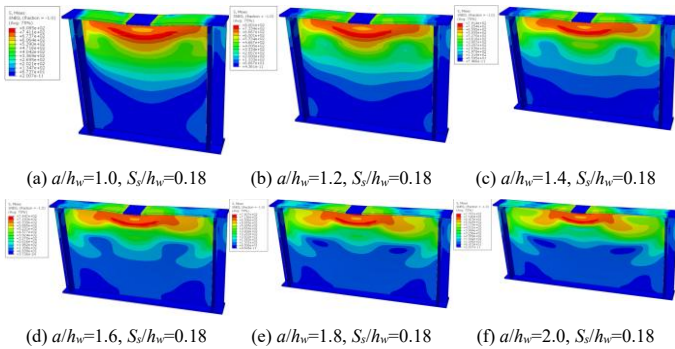


Fig. 12 Typical failure modes for F32001W2205 specimens with varying a/h_w

Furthermore, the local load-carrying capacity exhibits a decreasing trend as the web aspect ratio (a/h_w) increases from 1.0 to 2.0, with this effect being more

pronounced in specimens characterized by lower bearing width ratios (S_s/h_w). Specifically, in F1803W304 hybrid-configured girders, increasing a/h_w from 1.0 to 2.0 results in localized-loading resistance reductions of 3.15%, 4.78%, and 1.22% for S_s/h_w ratios of 0.1, 0.18, and 0.38, respectively. Similarly, in F32001W2205 hybrid-configured girders, the corresponding reductions were 6.56%, 8.34%, and 14.80%. These results underscore the substantial impact of the web aspect ratio on localized-loading resistance, emphasizing the importance of incorporating this parameter in the formulation of new design equations.

3.5. Effect of web slenderness (h_w/t_w)

The influence of the web slenderness ratio (h_w/t_w) on the local load-carrying capacity of the F1803W304 and F32001W2205 hybrid-configured girders is presented in Figs. 13(a) and (b). In this parametric analysis, the web aspect ratio (a/h_w) was systematically varied from 1.0 to 2.0 in increments of 0.2.

The findings reveal that for fixed values of h_w/t_w and increasing a/h_w leads to a pronounced reduction in the member's capacity under localized bearing force. In F1803W304 hybrid-configured girders, increasing the a/h_w ratio from 1.0 to 2.0 leads to a strength reduction of 13.4% for $h_w/t_w = 90.9$ and 5.1% for $h_w/t_w = 200.0$. Similarly, for F32001W2205 hybrid-configured girders, the same increase in a/h_w causes strength reductions of 19.9% and 9.3% for h_w/t_w values of 90.9 and 200.0, respectively.

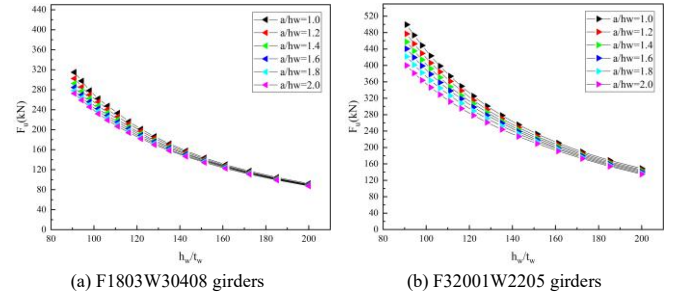


Fig. 13 Effect of h_w/t_w on the ultimate strength

The results also demonstrate that a marked decline in localized-loading capacity was observed as h_w/t_w increased from 90.9 to 200.0. In F1803W304 hybrid-configured girders with fixed a/h_w values of 1.0, 1.6, and 2.0, the reductions are 70.7%, 68.6%, and 67.9%, respectively, while in F32001W2205 hybrid-configured girders, the corresponding reductions are 70.2%, 68.2%, and 66.3%. In addition, as illustrated in Figs. 14 and 15, increasing web slenderness ratios result in more pronounced local buckling behavior, underscoring the critical role of web slenderness in governing the local load-carrying capacity of hybrid configured girders.

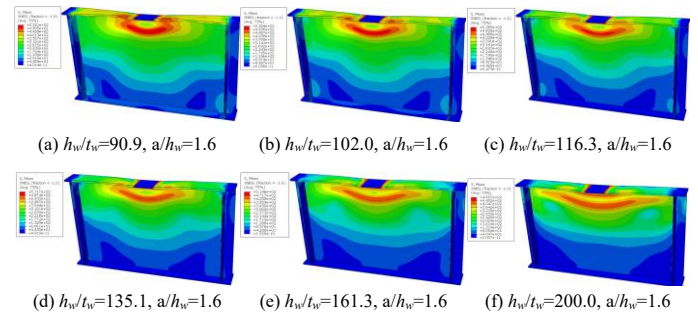


Fig. 14 Typical failure modes for F1803W30408 specimens with varying h_w/t_w

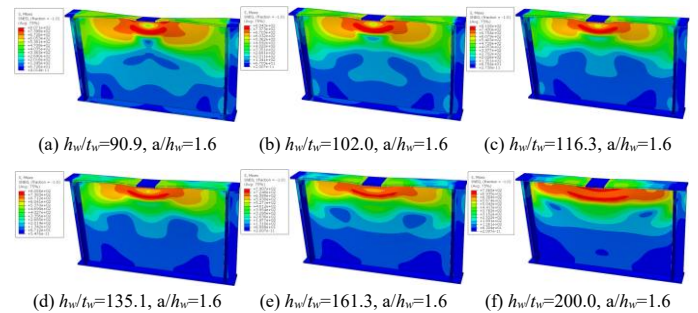


Fig. 15 Typical failure modes for F32001W2205 specimens with varying h_w/t_w

3.6. Effect of bearing width ratio (S_s/h_w)

The role of the bearing width-to-web depth ratio (S_s/h_w) in influencing localized-load resistance of F1803W304 and F32001W2205 hybrid-configured girders is presented in Figs. 16(a) and (b), with representative failure modes shown in Figs. 17 and 18. The results indicate a consistent increase in localized-loading resistance as the bearing width ratio (S_s/h_w) increases from 0.1 to 0.38. These observations are consistent with the trends identified in Section 3.5, which demonstrated that increasing h_w/t_w leads to a substantial drop in the peak localized load-bearing capacity. For F1803W304 hybrid-configured girders with an h_w/t_w ratio of 142.9, increasing S_s/h_w ranging from 0.1 to 0.38 with step sizes of 0.04 leads to corresponding resistance gains of 5.0%, 10.6%, 15.4%, 20.7%, 26.3%, 31.7%, and 37.4%. Similarly, for F32001W2205 hybrid-configured girders under the same conditions, the increases are 5.3%, 10.5%, 15.4%, 20.4%, 25.2%, 30.2%, and 35.3%. These results highlight the critical role of S_s/h_w in local bearing capacity.

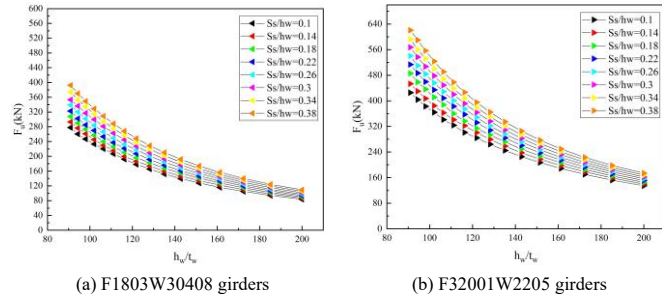


Fig. 16 Effect of S_s/h_w on the ultimate strength

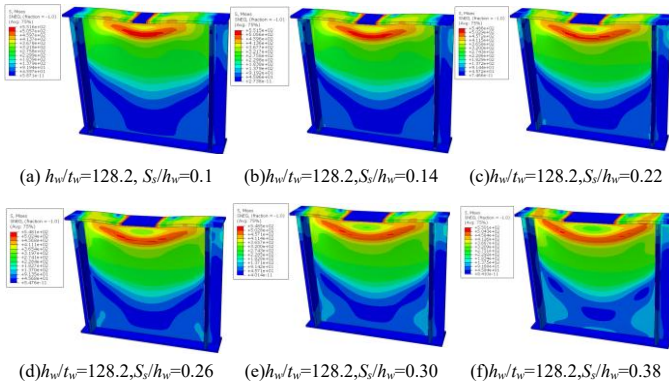


Fig. 17 Typical failure modes for F1803W30408 specimens with varying S_s/h_w

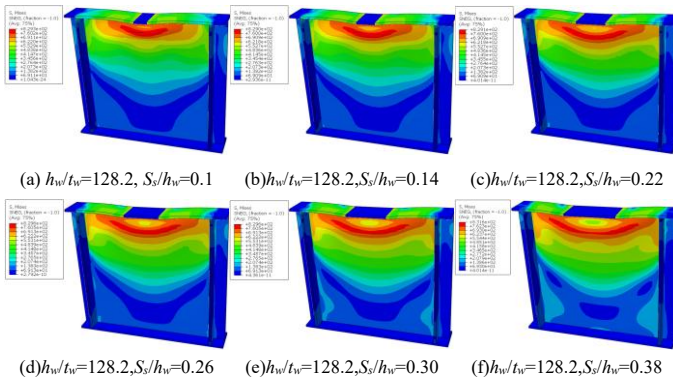


Fig. 18 Typical failure modes for F32001W2205 specimens with varying S_s/h_w

The comparative analyses presented above clearly demonstrate that the bearing width ratio, web aspect ratio, and web slenderness exert a dominant influence on the local load-carrying capacity of hybrid-configured girders. These parameters govern the load transfer mechanisms and stability characteristics of the web, and therefore play a critical role in determining the structural performance of such members. Consequently, they must be explicitly incorporated into the proposed design formulations developed in this study to ensure both predictive accuracy and engineering reliability. By contrast, the effect of initial geometric imperfections was shown to be negligible within the investigated range. This finding indicates that their influence on local load-

carrying capacity is secondary and may be reasonably neglected in the development of the new design recommendations, thereby enabling a simplified yet robust design framework.

4. Existing design rules for homogeneous girders

4.1. General

As previously mentioned, the design methods for hybrid stainless steel girders under localized forces are not currently available. The current design criteria outlined in GB 50017 [26], EN 1993-1-5 [27], and ANSI/AISC 360-22 [28] were generated with the assumption of homogeneous girders. These codes do not account for the unique characteristics and performance variations inherent in hybrid-configured girders, which combine different material grades in their construction. This section will provide a summary and discussion of the current design criteria, as outlined follows:

4.2. EN 1993-1-5 [27]

The current version of EN 1993-1-5 [27] provides design provisions for evaluating the resistance of homogeneous steel girders under three distinct loading conditions, with particular emphasis on the effects of plate behavior. A key distinction between EN 1993-1-5 [27] and GB 50017 [26] lies in the treatment of strength reductions associated with web buckling. EN 1993-1-5 [27] explicitly accounts for the influence of web instability on the load-carrying capacity by incorporating reduction factors that reflect the susceptibility of slender webs to local buckling. In contrast, GB 50017 [26] adopts a different design philosophy, in which the effects of web buckling are treated in a less explicit manner. This fundamental difference may result in discrepancies in the predicted resistance of girders, particularly for members with relatively high web slenderness ratios, where web instability plays a governing role in the overall structural response.

The design strength associated with failure under localized bearing force ($P_{u,EN}$) is determined by considering several critical parameters that influence the structural behavior of hybrid-configured girders. These parameters include the yield strength of the web (f_{yw}), the web thickness (t_w), the effective length (L_{eff}), and the partial safety factor (γ_{M1}). The interaction between these factors governs the localized-loading resistance. The mathematical formulation of this relationship is provided in Eqs. (1)-(7)

$$P_{u,EN} = \frac{f_{yw} L_{eff} t_w}{\gamma_{M1}} \quad (1)$$

$$L_y = s_s + 2t_f(1 + \sqrt{m_1 + m_2}) \leq a \quad (2)$$

$$m_1 = f_{yf} b_f / f_{yw} t_w \quad (3)$$

$$m_2 = 0.02(h_w/t_f)^2, \text{ if } \lambda_F > 0.5; m_2 = 0, \text{ if } \lambda_F \leq 0.5 \quad (4)$$

$$\chi_F = \frac{0.5}{\lambda_F} \leq 1.0 \quad (5)$$

$$\lambda_F = \sqrt{\frac{I_y t_w f_{yw}}{F_{cr}}}; F_{cr} = 0.9 k_F E \frac{t_w^3}{h_w} \quad (6)$$

$$k_F = 6 + 2\left(\frac{h_w}{a}\right)^2 \quad (7)$$

4.3. ANSI/AISC 360-22 [28]

In current practice, the evaluation of localized bearing resistance in homogeneous mild steel members is primarily governed by the design framework established in ANSI/AISC 360-22 [28]. The specification adopts a design philosophy that is largely in line with the principles underlying EN 1993-1-5 [27], thereby ensuring a reasonable level of consistency across major international steel design codes. Despite this general alignment, AISC 360-22 [28] does not explicitly incorporate the effect of web aspect ratio in the

formulation of local resistance. The omission of this geometric parameter limits the ability of the specification to capture the influence of web configuration on localized failure mechanisms

This limitation may result in discrepancies in strength predictions, particularly for members with varying web geometries. According to AISC 360-22 [28], the ultimate resistance of homogeneous mild steel members subjected to localized bearing force is evaluated using Eqs. (8)–(10):

(1) Transverse loading is applied at a distance $\geq d/2$ from the girder's end

$$P_{u,AISC} = 0.8t_w^2 \left[1 + 3 \left(\frac{S_s}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{E f_{yw} t_f}{t_w}} \quad (8)$$

(2) Transverse loading is applied at a distance $< d/2$ from the girder's end:

$$P_{u,AISC} = 0.4t_w^2 \left[1 + 3 \left(\frac{S_s}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{E f_{yw} t_f}{t_w}}; \text{ for } S_s / d \leq 0.2 \quad (9)$$

$$P_{u,AISC} = 0.4t_w^2 \left[1 + \left(\frac{4S_s}{d} - 0.2 \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{E f_{yw} t_f}{t_w}}; \text{ for } S_s / d > 0.2 \quad (10)$$

4.4. GB 50017 [26]

The existing provisions in GB 50017 [26] define design criteria for assessing the resistance of structural members subjected to localized bearing actions. Nevertheless, the scope of this standard is limited to members fabricated from homogeneous mild steel, without explicit consideration of the distinct mechanical properties and load-transfer mechanisms associated with stainless steel or hybrid structural systems. Consequently, the applicability of GB 50017 [26] to girders with hybrid configurations remains questionable. This limitation underscores the necessity for updated or supplementary design provisions that can accurately reflect the unique characteristics of advanced materials, ensuring reliable and efficient structural performance assessments.

To evaluate the resistance of homogeneous mild steel girders subjected to localized forces, the following Eqs. (11)–(12) can be applied:

$$P_{u,GB} = \frac{f_{yw} t_w l_z}{\Psi} \quad (11)$$

$$l_z = S_s + 5t_f + 2h_R \quad (12)$$

4.5. Evaluation of the existing design standards

As summarized in Table 5, a comprehensive comparative assessment was conducted to evaluate the accuracy of existing design provisions for homogeneous girders. The results of the parametric study were systematically compared with the strength predictions obtained using widely adopted design standards, including GB 50017 [26], EN 1993-1-5 [27], and ANSI/AISC 360-22 [28].

Table 5

Comparison of FEA results with predicted values by the existing standards

Material		GB 50017 [26]	EN 1993- 1-5 [27]	ANSI/AISC 360-22 [28]
		$F_{u,GB} / F_{u,FE}$	$F_{u,EN} / F_{u,FE}$	$F_{u,AISC} / F_{u,FE}$
F1803W304	Mean	1.033	0.587	0.850
	COV	0.228	0.109	0.069
F32001W2205	Mean	1.294	0.519	0.755
	COV	0.230	0.116	0.080

For EN 1993-1-5 [27], the average strength ratio ($F_{u,EN}/F_{u,FE}$) is 0.587 and 0.519 for F1803W304 and F32001W2205 hybrid-configured girders, respectively, yielding coefficient of variation (COV) values of 0.109 and 0.116. Figs. 19(a) and (b) further compare these predictions with FEA results, highlighting that EN 1993-1-5 [27] provides notably conservative estimates. This suggests that the current provisions may underestimate the localized-loading resistance of hybrid-configured girders, potentially leading to inefficient material utilization and overly cautious design recommendations.

Future revisions to EN 1993-1-5 [27] may be necessary to enhance its predictive accuracy.

For ANSI/AISC 360-22 [28], the average strength ratio ($F_{u,AISC}/F_{u,FE}$) is 0.850 and 0.755, with COVs of 0.069 and 0.080 for F1803W304 and F32001W2205 hybrid-configured girders, respectively. As illustrated in Figs. 19(c) and (d), ANSI/AISC 360-22 [28] consistently predicts localized-loading resistance with an average conservatism of 19.8%. Compared to both GB 50017 [26] and EN 1993-1-5 [27], ANSI/AISC 360-22 [28] demonstrates improved predictive accuracy, making it a more reliable reference for assessing the localized-loading resistance of hybrid-configured girders. The different conservatism of EN 1993-1-5 [27] and AISC 360-22 [28] mainly stems from their treatment of web instability and parameter sensitivity. EN 1993-1-5 [27] includes web buckling reduction linked to web slenderness and strength, leading to increased conservatism for higher web grades and instability-controlled regimes. In contrast, AISC 360-22 [28] does not explicitly consider geometric parameters such as the load length-to-web depth ratio (a/h_w), resulting in reduced sensitivity to web grade and parameter regime. This explains the observed differences in conservatism and highlights directions for future code improvement.

For GB 50017 [26], the mean strength ratio ($F_{u,GB}/F_{u,FE}$) is 1.033 and 1.294, with corresponding COVs of 0.228 and 0.230 for F1803W304 and F32001W2205 hybrid-configured girders, respectively. Figs. 19(e) and (f) provide a comparative assessment, revealing that GB 50017 [26] overestimates localized-loading resistance, with strength predictions averaging 16.4% higher than those derived from FEA simulations. These findings indicate that the current GB 50017 [26] provisions are unconservative and may require modifications to improve their reliability in predicting the performance of hybrid-configured girders under localized bearing force. The approximately 16% unconservative prediction mainly arises from the simplified assumptions adopted in GB 50017 [26] for patch loading resistance. The code provisions [26] are largely based on empirical formulations calibrated for conventional homogeneous mild steel sections. In contrast, the specimens investigated in this study are made of stainless steel, which exhibits distinct material characteristics, including pronounced strain hardening and different post-yield behavior.

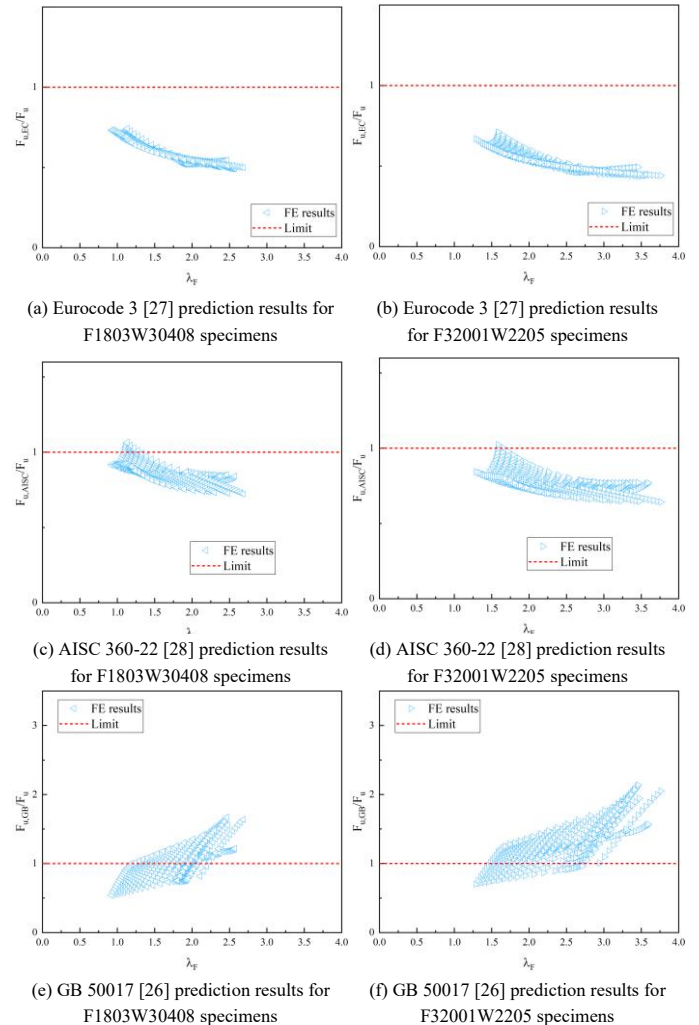


Fig. 19 Comparison between numerical results with design strength derived from current design methods [26-28]

It should be noted that a brief FE uncertainty assessment has been included by examining the sensitivity of the predicted patch loading capacity to (i) mesh refinement, (ii) initial geometric imperfection amplitude, and (iii) material property variability within reasonable bounds. The comparison confirms that the conclusion regarding the unconservatism of GB 50017 [26] remains valid within this FE uncertainty range.

According to the comparative results, the current design requirements are unreliable in their capacity to anticipate the hybrid-configured girders' localized-loading resistance. Substantial discrepancies were identified between the parametric study outcomes and the strength values estimated by existing codes, highlighting the inherent limitations of current design standards.

5. New design proposals for hybrid-configured girders

5.1. General

In this study, a modified coefficient (ξ) was introduced into the existing design formulations to improve the accuracy of local load-carrying capacity predictions for hybrid-configured girders. This coefficient is intended to capture the distinct mechanical response arising from the hybrid material configuration, thereby addressing inherent limitations in current design provisions developed for homogeneous members. Based on this modification, two simplified design approaches were subsequently proposed, specifically tailored for hybrid-configured girders. These approaches are formulated on the basis of the design concepts adopted in EN 1993-1-5 [27] and ANSI/AISC 360-22 [28], and provide refined expressions for evaluating local load-carrying capacity while explicitly accounting for the unique structural characteristics of hybrid-configured girders. In earlier research, the authors [19] used a similar methodology to develop improved design techniques for welded girders.

The novelty of this study lies not merely in introducing a modified coefficient, but in identifying and quantifying the key mechanical differences between hybrid and homogeneous girders under patch loading. Existing patch loading formulations were developed for homogeneous sections and implicitly assume uniform material behavior across the flange and web. In hybrid girders, however, differences in σ - ϵ nonlinearity and strain hardening between the flange and web lead to altered load transfer, stress redistribution, and plastic zone development near the load introduction region. The coefficient ξ was therefore introduced to account for these hybrid-specific mechanisms in a rational manner. Each term in ξ represents a distinct physical contribution: (i) the relative flange-web material and stiffness mismatch, governing load sharing and effective load spread into the web; (ii) the web-dominated local failure mechanism, which controls patch loading resistance; and (iii) the influence of material nonlinearity and post-yield redistribution on the ultimate capacity. The form of ξ was thus derived based on observed mechanical behavior rather than purely empirical fitting.

5.2. Modified ANSI/AISC 360-22 [28]

Similarly, the proposed calculation methods for evaluating the localized-loading resistance ($F_{u, AISC-M}$) of F1803W304 and F32001W2205 hybrid-configured girders are derived from Eqs. (13) and (14), based on ANSI/AISC 360-22 [28]. To improve the accuracy of design predictions for S30408 and S2205 webs, a modified coefficient (ξ) was introduced. This coefficient accounts for key parameters, including S_s/h_w , a/h_w and h_w/t_w , and can be determined using Eqs. (13)–(14):

$$F_{u, AISC-M} = \xi \left(\frac{S_s}{h_w}, \frac{a}{h_w}, \frac{h_w}{t_w} \right) \frac{f_{yw} L_{eff} t_w}{\gamma_{M1}} \quad (13)$$

$$\xi \left(\frac{S_s}{h_w}, \frac{a}{h_w}, \frac{h_w}{t_w} \right) = \begin{cases} 0.6949 \times \left[1 + 0.2951 \left(\frac{S_s}{h_w} \right)^{1.2122} \right] \times \left[1 - 0.1752 \left(\frac{a}{h_w} \right)^{0.6024} \right] \times \left[1 + 0.0731 \left(\frac{h_w}{t_w} \right)^{0.539} \right] & \text{(For F1803W30408)} \\ 0.3348 \times \left[1 + 0.2582 \left(\frac{S_s}{h_w} \right)^{0.5839} \right] \times \left[1 - 0.2865 \left(\frac{a}{h_w} \right)^{0.4255} \right] \times \left[1 + 0.6810 \left(\frac{h_w}{t_w} \right)^{0.372} \right] & \text{(For F32001W2205)} \end{cases} \quad (14)$$

Figs. 20(a) and (b) present a comparison between the localized loading capacities predicted using the modified ANSI/AISC 360-22 design approach and those obtained from the parametric finite element investigation. As summarized in Table 6, the mean ratios of the finite element results (FEM) to the corresponding design predictions for the F1803W304 and F32001W2205 hybrid-configured girders are 1.001 and 1.002, respectively. The associated coefficients of variation (COVs) are 0.023 and 0.025, indicating a high level of consistency and low scatter. These results demonstrate that the modified ANSI/AISC 360-22 design equations provide accurate and reliable predictions

of local load-carrying capacity for hybrid stainless steel girders, thereby confirming their suitability for practical engineering design applications.

Table 6

Comparison of FEA results with predicted values by the proposed methods

Material		Modified EN 1993-1-5	Modified ANSI/AISC 360-22
		$F_{u, EN} / F_{u, FE}$	$F_{u, AISC} / F_{u, FE}$
F1803W304	Mean	1.000	1.001
	COV	0.025	0.023
F32001W2205	Mean	1.000	1.002
	COV	0.029	0.025

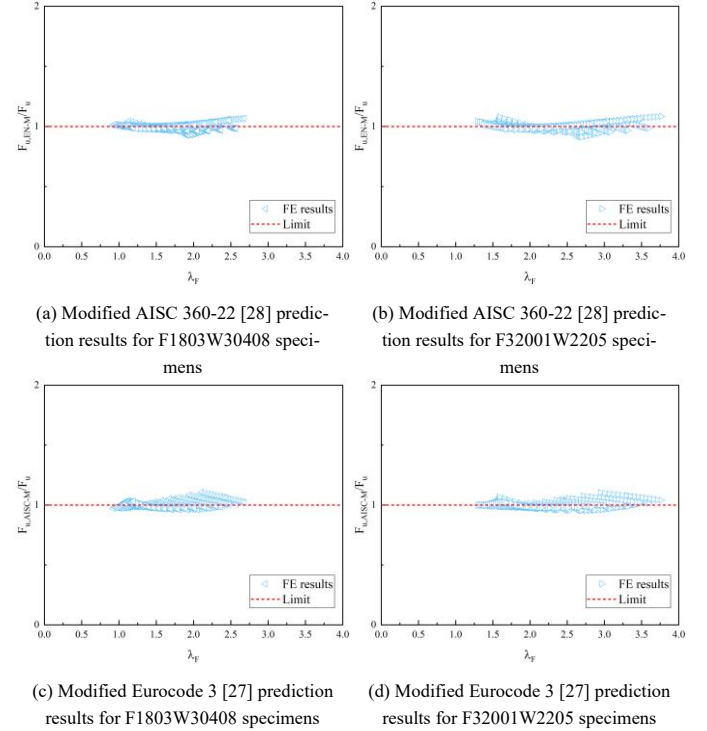


Fig. 20 Comparison between numerical results with design strength derived from new design methods

5.3. Modified EN 1993-1-5 [27]

The proposed calculation methods for evaluating the local load-carrying capacity ($F_{u, EN-M}$) are presented in Eqs. (15)–(16), which are developed through targeted adaptations of the EN 1993-1-5 design framework [24]. To improve the accuracy of resistance predictions for hybrid-configured girders, a modified coefficient (ξ) is introduced in the present study, specifically calibrated for the F1803W304 and F32001W2205 configurations. This coefficient is intended to account for the distinct material properties and mechanical behavior associated with the hybrid stainless steel composition. It is formulated as a function of key governing parameters, including the bearing width ratio (S_s/h_w), web aspect ratio (a/h_w) and web slenderness ratio (h_w/t_w). The modified coefficient ξ is determined using Eqs. (15)–(16):

$$F_{u, EN-M} = \xi \left(\frac{S_s}{h_w}, \frac{a}{h_w}, \frac{h_w}{t_w} \right) \frac{f_{yw} L_{eff} t_w}{\gamma_{M1}} \quad (15)$$

$$\xi \left(\frac{S_s}{h_w}, \frac{a}{h_w}, \frac{h_w}{t_w} \right) = \begin{cases} 0.6268 \times \left[1 + 0.4200 \left(\frac{S_s}{h_w} \right)^{1.4857} \right] \times \left[1 - 0.0012 \left(\frac{a}{h_w} \right)^{2.8587} \right] \times \left[1 + 0.0447 \left(\frac{h_w}{t_w} \right)^{0.7259} \right] & \text{(For F1803W30408)} \\ 0.5914 \times \left[1 + 0.3006 \left(\frac{S_s}{h_w} \right)^{0.9936} \right] \times \left[1 - 0.0139 \left(\frac{a}{h_w} \right)^{2.2613} \right] \times \left[1 + 0.0677 \left(\frac{h_w}{t_w} \right)^{0.7} \right] & \text{(For F32001W2205)} \end{cases} \quad (16)$$

As illustrated in Figs. 20(c) and (d), a comparative assessment was performed between the localized loading capacities predicted using the modified EN 1993-1-5 equations and those obtained from the parametric finite element analyses of the F1803W304 and F32001W2205 hybrid-configured girders. This comparison was conducted to evaluate the accuracy of the revised equations in capturing the local load-carrying capacity of these stainless steel

girders.

As summarized in Table 6, the mean ratios of the finite element results to the corresponding design predictions are equal to 1.000 for both the F1803W304 and F32001W2205 configurations. The associated coefficients of variation (COVs) are 0.025 and 0.029, respectively, indicating low scatter and a high level of agreement. These findings confirm that the revised EN 1993-1-5-based design equations provide reliable and precise estimates of local load-carrying capacity for hybrid stainless steel girders.

6. Concluding remarks and limitations

A novel generation of hybrid stainless steel girders has been used in marine structures recently. This study introduces new simplified design methods for assessing the resistance of hybrid stainless steel girders under localized bearing force, expanding on the authors' earlier experimental research. A comprehensive investigation was carried out, involving 1248 finite element (FE) simulations. The principal findings of this study are summarized as follows:

(1) A finite element (FE) model, previously developed and rigorously validated by the authors, was utilized to perform a comprehensive parametric investigation. The study focused on evaluating the influence of critical parameters, including the bearing width ratio, web aspect ratio, web slenderness, and local geometric deformations, on the structural behavior of the tested girders.

(2) The parametric study results reveal that the localized-loading resistance of hybrid stainless steel girders is primarily influenced by three key factors: the web aspect ratio, web slenderness ratio, and bearing width ratio. The analysis demonstrates a consistent enhancement in resistance as the bearing width ratio (S_f/h_w) increases from 0.1 to 0.4. Conversely, a notable decline in resistance is

observed when the web slenderness (h_w/t_w) rises from 90.9 to 200.0. In comparison, the web aspect ratio (a/h_w), ranging from 1.0 to 2.0) exhibits a relatively limited effect on resistance.

(3) The comparison between hybrid and homogeneous girders demonstrates that the use of QN1803 flanges does not adversely affect performance under localized bearing force. Furthermore, owing to the higher strength of QN1803 relative to EN 1.4301 (S30408), hybrid-configured girders are expected to achieve greater moment resistance compared to their homogeneous counterparts.

(4) The comparison between the predicted strength values and numerical results revealed notable inconsistencies with existing design codes. Specifically, the predictions based on GB 50017 (2017) were unconservative, underestimating the strength by an average of 16.4%. In contrast, EN 1993-1-5 (2006) and ANSI/AISC 360-22 (2022) yielded conservative estimates, overestimating the strength by 44.7% and 19.8%, respectively. These substantial discrepancies between the parametric study outcomes and code-based predictions underscore the limitations of current design standards in accurately assessing the resistance of such hybrid-configured girders.

(5) New simplified design recommendations were formulated for determining the resistance of hybrid-configured girders. A modified coefficient (ζ) was introduced, built upon the framework of existing design equations. The newly proposed methods enhance the accuracy of predicting the localized-loading resistance of such hybrid-configured girders.

(6) The present study focuses on internal one-flange (IOF) midspan patch loading, which represents a fundamental and widely used reference case for isolating web-dominated mechanisms. Other loading scenarios are identified as important extensions beyond the current scope.

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EXPERIMENTAL INVESTIGATION FOR ANTI-SLIPPING PERFORMANCE OF STRUCTURAL CARBON STEEL (Q345B) SLIP-RESISTANT CONNECTIONS WITH PARTICLES EMBEDDED IN CONNECTED PLATES

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ABSTRACT

Friction-type high-strength bolted connections have gradually become popular in steel structure buildings due to their inherent advantages, but their shear resistance capacity still needs to be improved. To better enhance the anti-slip performance of ordinary steel structure connections, this paper proposes an innovative anti-slip connection concept that involves embedding specific particles into the surface of ordinary steel. This method enhances the shear resistance of the connection by increasing the friction coefficient of the contact surfaces, with significant effectiveness. Appropriate particle materials and shapes were selected via theoretical analysis. The concept's effectiveness and feasibility were verified via two static test stages: particle embedding tests on steel plates and anti-slip tests of bolted connections under pre-tension. The study reveals the influence laws of factors such as particle diameter, arrangement pattern, distance from the hole wall, and the magnitude of bolt pre-tension. Results of the particle embedding tests indicate that for 6mm steel plates, when the particle diameter is less than 3.5mm, the embedding load increases approximately proportionally with the number of particles. The anti-slip test indicates that as the diameter and number of embedded particles increase, the connection slip coefficient improves significantly. Embedding 4 particles with a 3mm diameter results in a connection anti-slip coefficient of 2.8. The new particle-embedded connection outperforms friction-type high-strength bolted connections in maintaining stable anti-slip loads, highlighting its potential for engineering applications.

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1. Introduction

Steel structure connections are the key to ensuring structural safety and stability [1-3], and the slip resistance of bolted connections directly determines their bearing capacity [4-7]. For high-rise buildings, heavy industrial structures and other key projects, the requirement for the slip resistance coefficient of connections is usually ≥ 1.0 [8-10], but the existing methods are difficult to meet this demand, so it is urgent to develop a new high-efficiency slip resistance improvement technology [11-13].

Existing methods to improve the slip resistance of bolted connections mainly focus on surface treatment (grinding, sandblasting, painting, etc.) and bolt type optimization [14-16]. Scholars have conducted relevant research on this [17-19]. Common methods in construction are as follows: Grinding with a grinding wheel, sandblasting or shot peening, applying inorganic zinc-rich paint after sandblasting or shot peening, generating red rust after sandblasting or shot peening. (1) Grinding with a grinding wheel [20]. The effect after treatment is difficult to stabilize, and the operation process is cumbersome. (2) Sandblasting or shot peening [21]. Under conventional treatment conditions, shot peening has a better effect than sandblasting, and shot peening can be reused. (3) Applying inorganic zinc-rich paint after sandblasting or shot peening [22]. If the steel is exposed to the air without treatment after sandblasting or shot peening, it is prone to secondary rusting. To avoid this situation, inorganic zinc-rich paint is often applied in the later stage of construction [23]. During spraying, tape is pasted near the screw holes to avoid spraying on unnecessary places. (4) Generating red rust after sandblasting or shot peening [24]. After sandblasting or shot peening the surface of the steel plate, no relevant treatment is done on the plate surface, and a layer of red rust formed on its surface is used to increase the roughness of the steel plate. After red rust is formed on the surface of the steel plate, the loose rust is removed without damaging the red rust surface.

Because the rust surface method is difficult to control the rusting time and rusting degree, it is rarely used in large steel structural parts. However, these methods for increasing the friction coefficient of contact surfaces in components have notable limitations and limited effectiveness. With the increase in the height of modern buildings and the complexity of functions, bolt connections have also kept up with the development: simplified construction, use of new materials, development of new products, etc [25].

This paper proposes a new type of high-efficiency steel structure anti-slip connection method of particle-embedded anti-slip connection to increase the friction coefficient of the contact surfaces of components. Through two stages of static tests, the test of embedding particles into the steel plate and the connection anti-slip test after applying pre-tension to the bolts, the influence laws of factors such as particle diameter, arrangement mode, distance from the hole wall, and the magnitude of bolt pre-tension are revealed.

2. Selection of test materials

2.1. Selection of connecting steel plates and bolts

The specimen material adopts Q345B ordinary steel plates with dimensions of 300mm $\times$ 40mm $\times$ 6mm. The bolts used are 8.8-grade high-strength external hexagon bolts with a nominal diameter of 6mm and a length of 25mm. In order to provide a theoretical reference for the load required for subsequent particle embedding into the steel plate and the load of the steel plate before the particles push to form plows, a material property test of the steel plate should be carried out first to obtain the yield strength f_y and tensile strength f_u of the steel plate. The measured dimensions of the specimens and the test results are listed in Table 1. It can be concluded that the yield strength of the connecting steel plate is 457.06 MPa, and the tensile strength is 539.51 MPa.

Table 1
Mechanical properties of ordinary steel plates

Specimen	Thickness t(mm)	Width b(mm)	Cross-sectional area A (mm ²)	Tensile strength f_u (Mpa)	Average value of f_u (MPa)	Yield strength f_y (MPa)	Average value of f_y (MPa)
C6-1	5.7	15.00	85.50	546.20	539.51	466.48	457.06
C6-2	5.7	15.00	85.50	542.37		462.59	
C6-3	5.7	15.00	85.50	537.55		452.58	
C6-4	5.8	15.00	87.00	531.93		446.57	

Note: For all specimens, dimensions were measured at 3 positions in the test section, and the minimum cross-sectional dimension in the table is the smallest of the measured values.

2.2. Selection of embedded-in particles

A suitable particle shape can effectively reduce the difficulty of embedding particles into the connecting plate, facilitating manual operation. GCr15 bearing steel balls have higher hardness, so they are selected as the particles to be embedded in. The particle diameters are 1.5mm, 2mm, 2.5mm, 3mm, and 3.5mm respectively.

3. Particle embedding test

3.1. Design of particle embedding test

This test is a room-temperature compression test, conducted on a CSS-44100 electronic universal testing machine. Particles were fixed at specific positions on the steel plate surface using double-sided tape to ensure centering, as illustrated in Fig.1. The loading system is controlled by displacement, with a loading speed of 2mm/min, until the upper and lower steel plates are in complete contact and compressed without any gaps. The criterion for judging the tight contact of the plates is that no light can pass through them, as shown in Fig.2.

3.2. Phenomena and data of embedding tests

3.2.1. Embedding particles with a diameter of 2mm or less into connecting steel plates

In the test, GCr15 bearing steel particles with the same diameter but different quantities were embedded into the connecting steel plates as required, and the arrangement of the particles is shown in Fig.3. After the test, the failure morphology of the particles and the surface of the connecting plate was observed, as shown in Fig.4. The particles can be easily taken out after the test, and they are smooth without deformation. The dents on the surface of the connecting plate are clear, and there are no cracks or bulges around them, indicating that the test effect is very good. For the groups with two or more

particles, the distance between particles, which is 5 times the particle diameter, was taken as the variable control factor.



Fig. 1 Fix the particle position



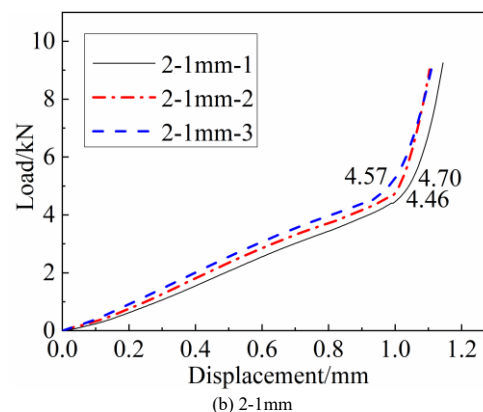
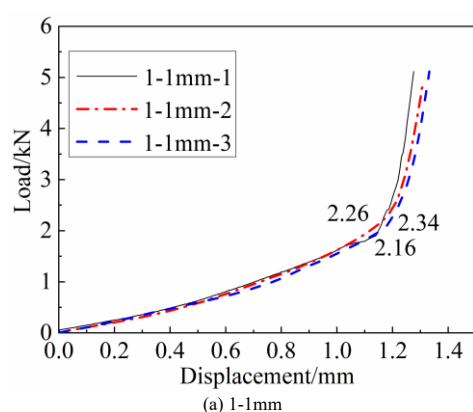
Fig. 2 Fully embedded into the steel plate



Fig. 3 Particle layout



Fig. 4 Test effect diagram of embedding into 1.5mm particles



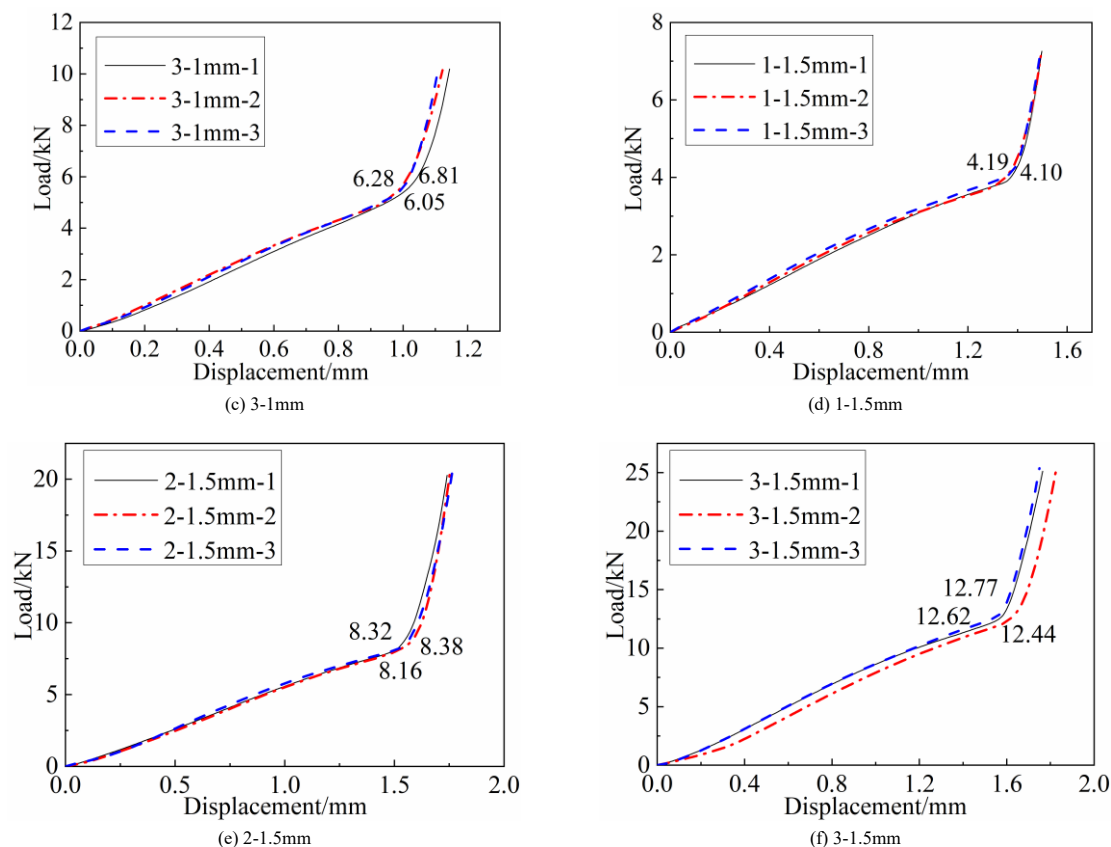


Fig. 5 Load-displacement curves of embedding different numbers of particles

Table 2

Average minimum embedded load

Particle diameter	Load (kN)	1 particle	2 particles	3 particles
1 mm	1	2.16	4.46	6.80
	2	2.26	4.57	6.28
	3	2.34	4.70	6.04
	Average	2.25	4.58	6.37
1.5mm	1	4.10	8.16	12.44
	2	4.10	8.32	12.62
	3	4.19	8.38	12.77
	Average	4.13	8.29	12.61

(1) Test data of embedding 1mm and 1.5mm particles into connecting steel plates

It can be seen from Fig.5 that the process of embedding particles into the connecting plate is smooth and stable, and the curves have a high degree of coincidence. This reflects that the particle embedding process and its minimum embedding load can be stably controlled in actual operation, ensuring that the particles are completely embedded in. The specific data are shown in Table 2. The minimum load required to embed particles into the steel plate basically increases in multiples as the number of particles increases, which also proves the reliability of the experimental data to a certain extent.

(2) Embedding 2mm particles into the connecting steel plate

In accordance with the test requirements, 2mm-diameter particles were embedded into the connecting plate in different quantities, and the arrangement of the particles is similar to that shown in Fig.6. After the test, the failure morphology of the particles and the surface of the connecting plate was observed. The particles could be easily taken out after the test, with a smooth surface and no deformation. The dents on the surface of the connecting plate were clear, and there were no cracks or bulges around them, indicating that the test effect was good. The load-displacement curves and specific data for different numbers of particles are shown in Fig.7.



Fig. 6 Test effect diagram of embedding into 2mm particles

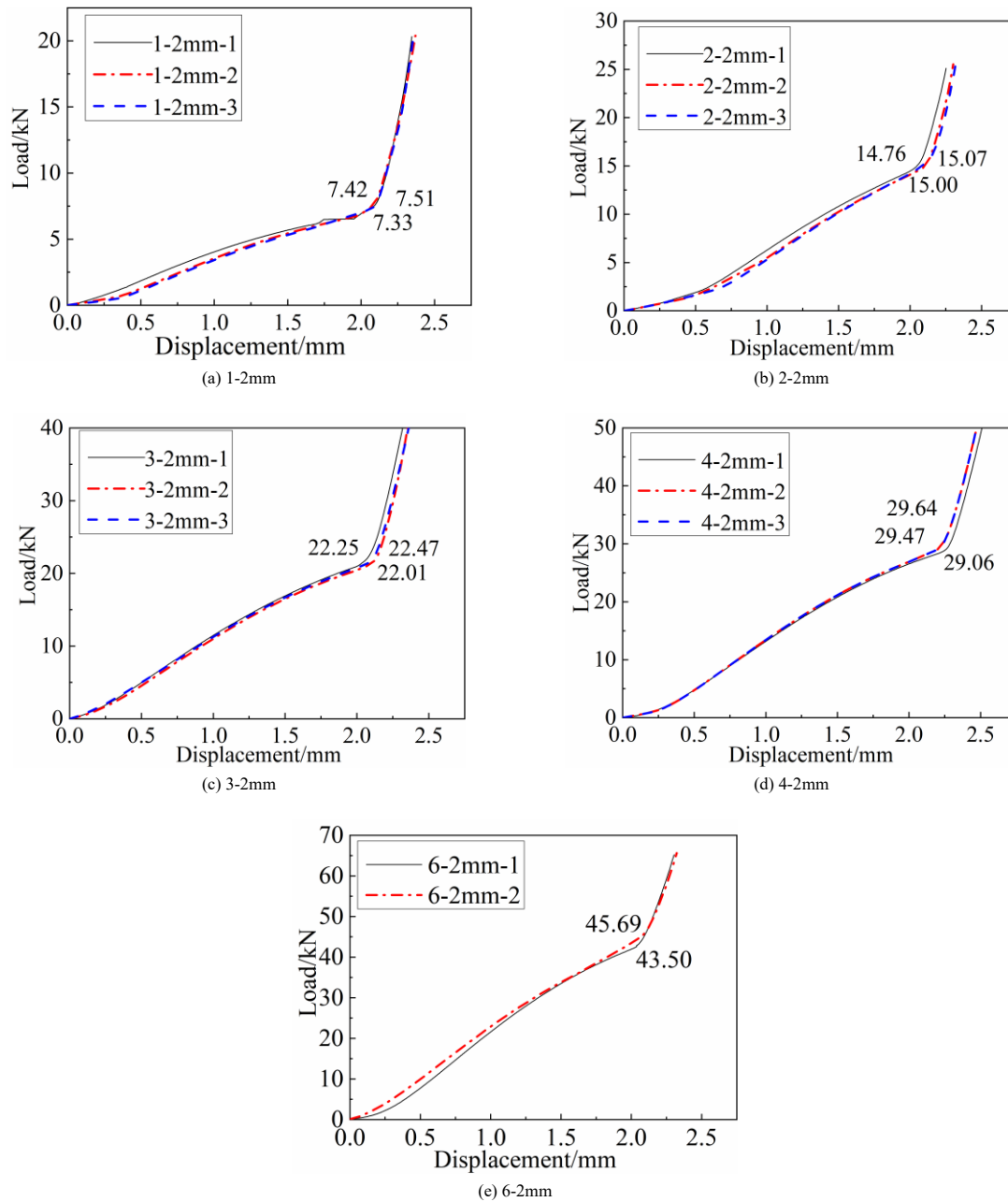


Fig. 7 Load-displacement curves of embedding different numbers of particles

The process of embedding particles into the connecting plate is smooth and stable, with a high degree of curve coincidence. This indicates that the particle embedding process and its minimum embedding load can be stably controlled, ensuring that the particles are completely embedded in. The specific data are shown in Table 3.

Table 3
Average minimum embedded loads

Particle diameter	Load (kN)	1 particle	2 particles	3 particles	4 particles	6 particles
2mm	1	7.41	14.76	22.47	29.06	43.50
	2	7.33	15.00	22.01	29.47	45.69
	3	7.50	15.07	22.25	32.32	45.69
	Average	7.41	14.94	22.24	30.28	44.96

The minimum load required to embed particles into the steel plate basically increases in multiples as the number of particles increases, and 2mm diameter particles are suitable for this connection.

3.2.2. Embedding large-diameter particles into connecting steel plates

Due to the relatively large particle size of 3.5mm particles, in order to detect whether the particles are deformed or crushed during the embedding process, the same particle was repeatedly embedded in the test. It was found that the particles were almost undamaged, and no deformation was observed when measured with an electronic digital caliper. There was no damage to the particles caused by the embedding of the stainless steel connecting plate, which indicates that particles of this size can maintain their shape well during the connection anti-slip test and fully exert their shear resistance in the connection. The embedded holes were observed under magnification, showing smooth edges without roughness, and a slight sense of protrusion was felt when touching around the furrows by hand, as shown in Fig.8. It can be seen that the 3.5mm diameter particles match the steel plate well. The load-displacement curves and specific data for different numbers of particles with diameters of 2.5mm, 3mm, and 3.5mm are shown in Fig.9 and Table 4.

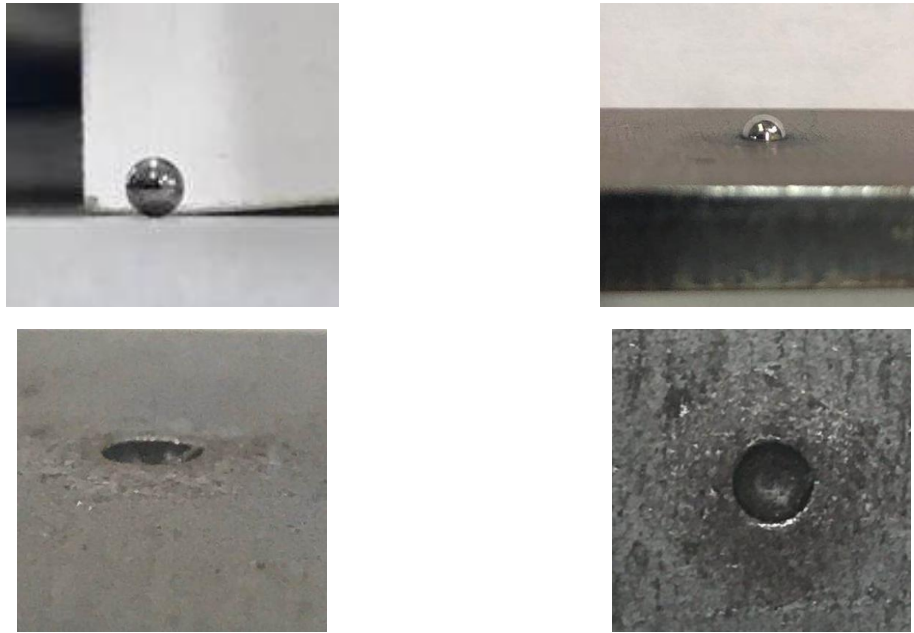
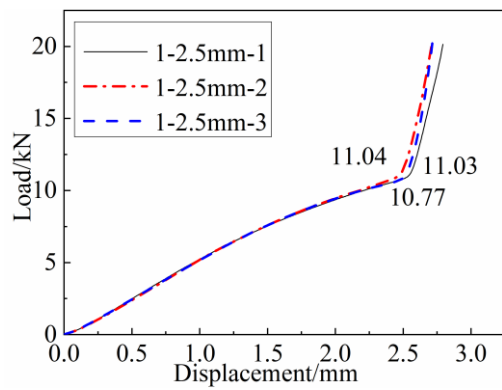
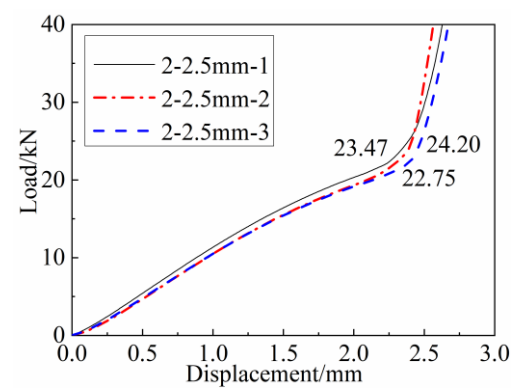


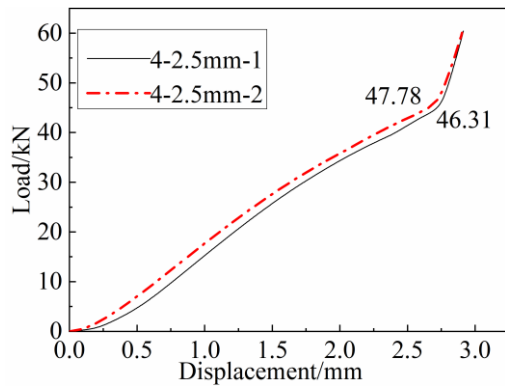
Fig. 8 Test effect diagram of embedding into 3.5mm particles



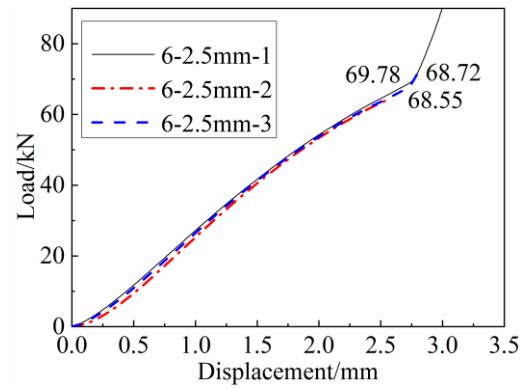
(a) 1-2.5mm



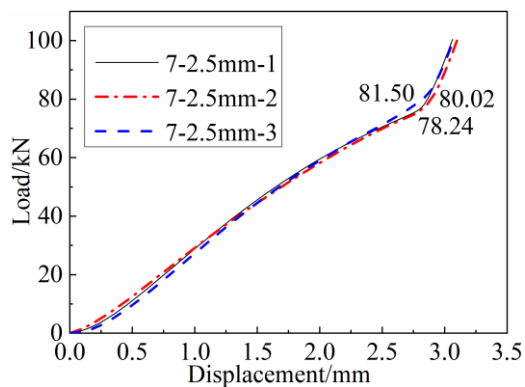
(b) 2-2.5mm



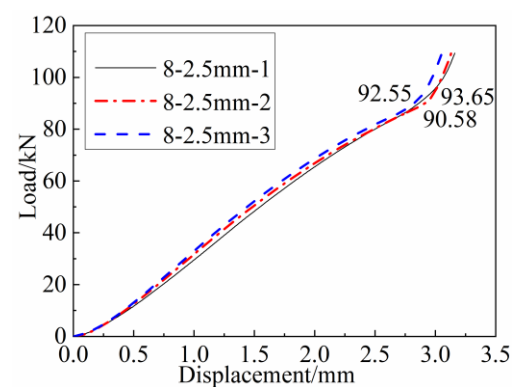
(c) 4-2.5mm



(d) 6-2.5mm



(e) 7-2.5mm



(f) 8-2.5mm

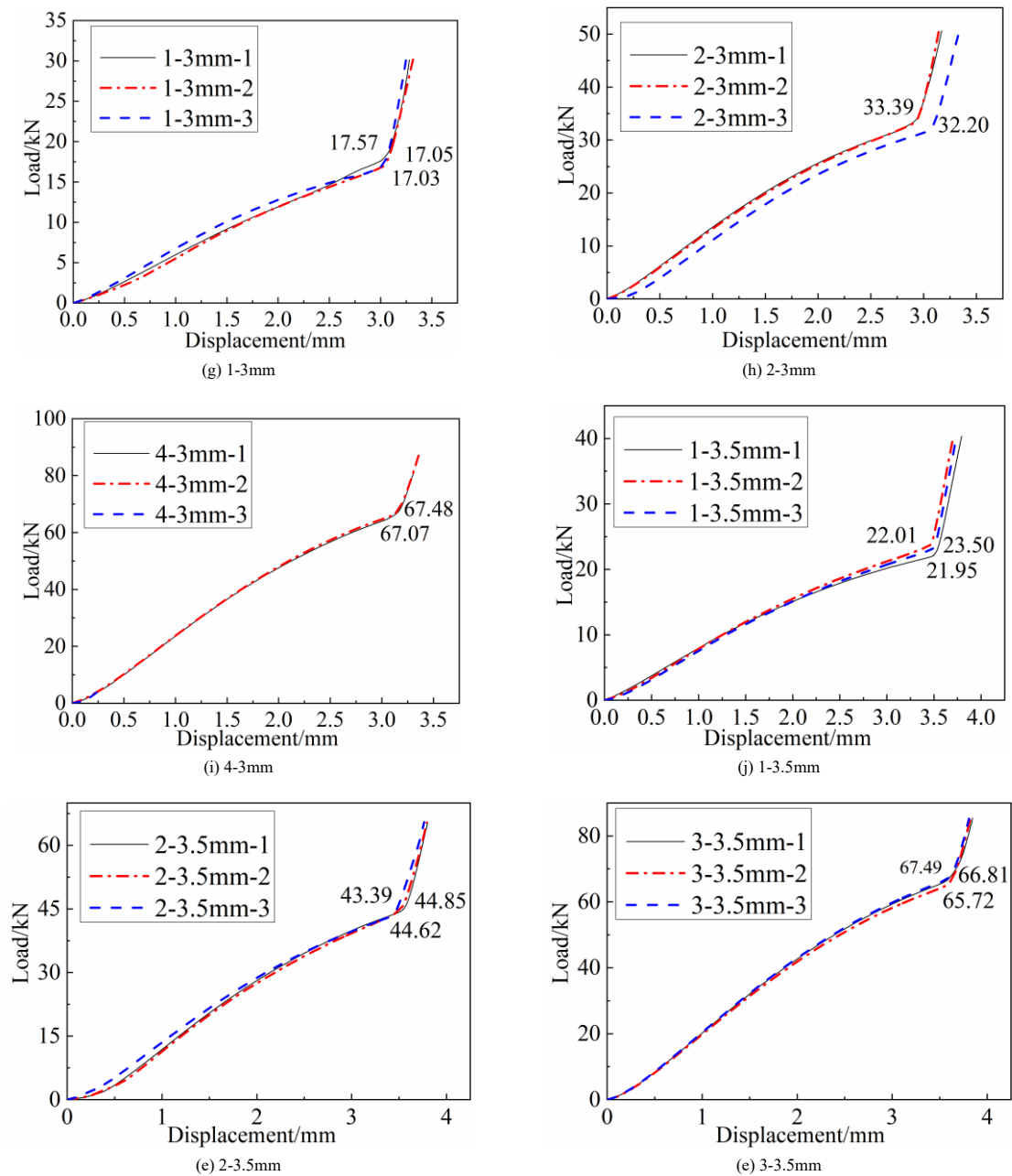


Fig. 9 Load-displacement curves of embedding different numbers of particles

Table 4

Average minimum embedded loads

Particle diameter	Load (kN)	1 particle	2 particles	3 particles	4 particles	6 particles	7 particles	8 particles
2.5mm	1.00	10.77	22.75		46.31	68.55	78.24	90.58
	2.00	11.03	23.47		47.78	68.72	80.02	92.55
	3.00	11.04	24.20		47.78	69.78	81.50	93.65
	Average	10.95	23.47		47.29	69.02	79.92	92.26
3mm	1	17.03	33.39		66.92			
	2	17.05	33.39		66.92			
	3	17.57	32.20		66.92			
	Average	17.22	32.99		66.92			
3.5mm	1	21.95	43.39	65.72				
	2	22.01	44.62	66.81				
	3	23.50	44.85	67.49				
	Average	22.49	44.29	66.67				

Like small-diameter particles, the minimum load required to embed particles into the steel plate basically increases in multiples as the number of particles increases, which indicates that within a certain range of particle

diameters, the number of particles does not affect the embedding load value of a single particle.

3.3. Theoretical analysis of particle embedding test

Furthermore, to clarify the relationship between particle size and embedded load, the data obtained from the experiment were sorted out, and it was found that with the increase of particle diameter, the increase of the minimum indentation load value was not proportional to the increase of particle volume, as shown in Table 5.

Table 5

The relationship between the embedded loads and the volume of the particles

Particle diameter/mm	Particle volume/mm ³	Volume increase multiple	Embedded load/kN	Load increase multiple
1	0.52		2.25	
1.5	1.77	3.38	4.20	1.87
2	4.19	8.00	7.41	3.29
2.5	8.18	15.63	10.95	4.87
3	14.14	27.00	17.22	7.65
3.5	22.45	42.88	22.49	10.00

Considering the need to gain an in-depth understanding of the extent to which particle size affects the indentation load and to provide reference specifications for the future engineering application of this new type of connection, this paper has fitted relevant calculation equations based on experimental data, the square of the fitting correlation coefficient is 0.9976, which indicates that the fitting degree of the curve is very good. Thus, Eq.(1) can be obtained.

$$F_e = \beta(2.06d^2 - 1.0529d + 1.1914) \quad (1)$$

Where: β -Correction coefficient (greater than 1); d -Particle diameter; F_e - Minimum indentation load of the particle. The correction coefficient β ($\beta > 1$) reflects the hardness matching relationship between particles and steel plates, and is associated with the Brinell hardness formula. Combined with experimental data, the value range of β is 1.2~1.5, which is positively correlated with the Brinell hardness of Q345B steel (HB=190~235) and GCr15 steel (HB=600~700). β can be pre-estimated through the material hardness value.

4. Anti-slip performance test

4.1. Test design

Q345B steel plates are used as connecting plates, with dimensions of 300mm × 40mm and a thickness of 6mm; the particles embedded into the steel plates are GCr15 bearing steel particles, with particle diameters of 1.5mm, 2mm, 2.5mm, and 3mm respectively; the bolts used for connection are 8.8-grade high-strength outer hexagon bolts with a diameter of 6mm. In order to effectively determine the anti-slip performance of the embedded particles, this test adopts a typical single-friction-surface and single-bolt pre-tightened bolt connection form, as shown in Fig.10. However, it should be emphasized that the single-shear test has limitations: when multiple particles are arranged, the warpage of the plate end leads to poor fit between particles and indentations.

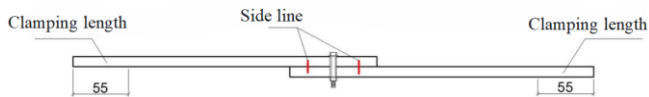


Fig. 10 High-strength bolt connection methods

The anti-slip effect of embedded particles was tested under the conditions of different numbers of particles and different particle diameters. Since the number of particles arranged is not large, double-sided tape marked with position points was not customized. In the experiment, specific diameter circles were drawn on white paper in advance and specific points were marked, then the circles were cut out and placed at the designated positions on the connecting plate, so that specific points could be marked on the connecting plate, as shown in Fig.11. This process is simple and easy to operate, with high accuracy, and the marking paper can be reused.



Fig. 11 Actual operation diagram of particle layout

After keeping the test piece embedded tightly, connect it with high-strength bolts and tighten them. Since there is only one bolt, a torque wrench is used to directly tighten it to the final torque value, and it is ensured that the steel plate does not shift during the tightening process. This test is an indoor normal-temperature tensile test, which is carried out on the WAW-600-G microcomputer-controlled electro-hydraulic servo universal testing machine. Both ends of the test piece are fixed and clamped, as shown in Fig.12. The test loading system was displacement-controlled in accordance with specifications, with a loading rate of 2 mm/min. The tensile process continues until the bolt shank is sheared off, which is observed on the load-displacement curve as a sharp drop in bearing capacity.

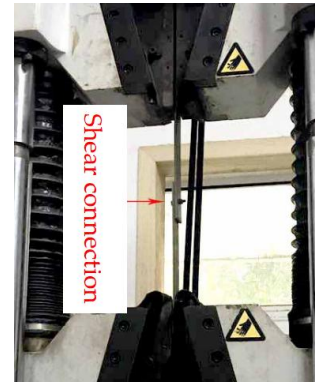


Fig. 12 Equipment for the anti-slip tests

Through 10 sets of experiments tested with a torque wrench, it is found that the breaking torque is all around 27 N·m, so T_i is set as 27 N·m. The average ultimate bearing capacity when the bolt is pulled off is 23.45 kN. Calculated by the equation $k = T_i / dP_i$, we can get $k = 0.19$. To ensure that the steel plate firmly clamps the particles so that their shear resistance can be fully exerted: Take $T = 9.78 \text{ N} \cdot \text{m}$, $T = 90\%T_i = 24.3 \text{ N} \cdot \text{m}$ for comparison; When $T = 24.3 \text{ N} \cdot \text{m}$, $P = T / kd = 21.32 \text{ N} \cdot \text{m}$; When $T = 9.78 \text{ N} \cdot \text{m}$, $P = T / kd = 8.58 \text{ N} \cdot \text{m}$.

To clarify the performance of the connection under repeated loads, cyclic load tests and vibration tests were carried out. The 10,000 cyclic load tests (load amplitude is 50% of the ultimate load) show that the particles have no looseness, the indentation deformation is $\leq 0.02 \text{ mm}$, and the pre-tension loss is $\leq 5\%$, which is much lower than the pre-tension loss of friction-type high-strength bolts (about 15%). Vibration tests (frequency 10~100Hz, acceleration 0.5g) show that the attenuation of the slip resistance coefficient of the connection is $\leq 8\%$, which meets the long-term use requirements of industrial structures.

4.2. Phenomena and data of anti-slip tests

4.2.1. Anti-slip test of connections under a bolt pre-tension of 8.58kN

The test was stopped when the computer showed a sharp drop in the anti-slip bearing capacity of the connection. At this point, the connection had suffered shear failure. It was observed that the steel plates had obvious misalignment, and the plate ends were warped, though not significantly, as shown in Fig.13. When the connection was disassembled, plow furrows caused by the misalignment of the plates and the extrusion of the particles could be seen on the surfaces of the two connecting steel plates. The surfaces of the plow furrows were smooth, with no cracks or visible deformation around them, and there were protrusions at the ends of the plow furrows caused by the pushing of the particles. All the particles were located on one of the connecting steel plates, and no fragmentation or damage was observed after they were taken out, as shown in Fig.14.



Fig. 13 Failure of bolt connection after testing

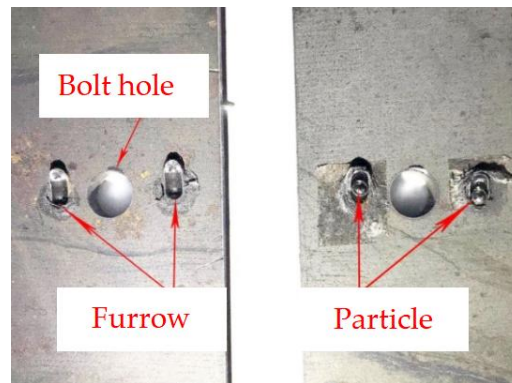


Fig. 14 Furrow on the plate and embedded particle

The load-displacement curves of the anti-slip tests for connections with particles of different quantities embedded into the steel plate surface to a depth of 2mm are shown in Fig.15. The specific test data are shown in Table 6. Embedding particles into the surface of Q345B steel structures can significantly

improve the anti-slip coefficient of the connections. After embedding 1, 2, and 4 particles of 2mm into the connecting plate, the anti-slip coefficients increased by 97.5%, 155%, and 270% respectively. With the increase in the number of particles, the ultimate slip load also increased.

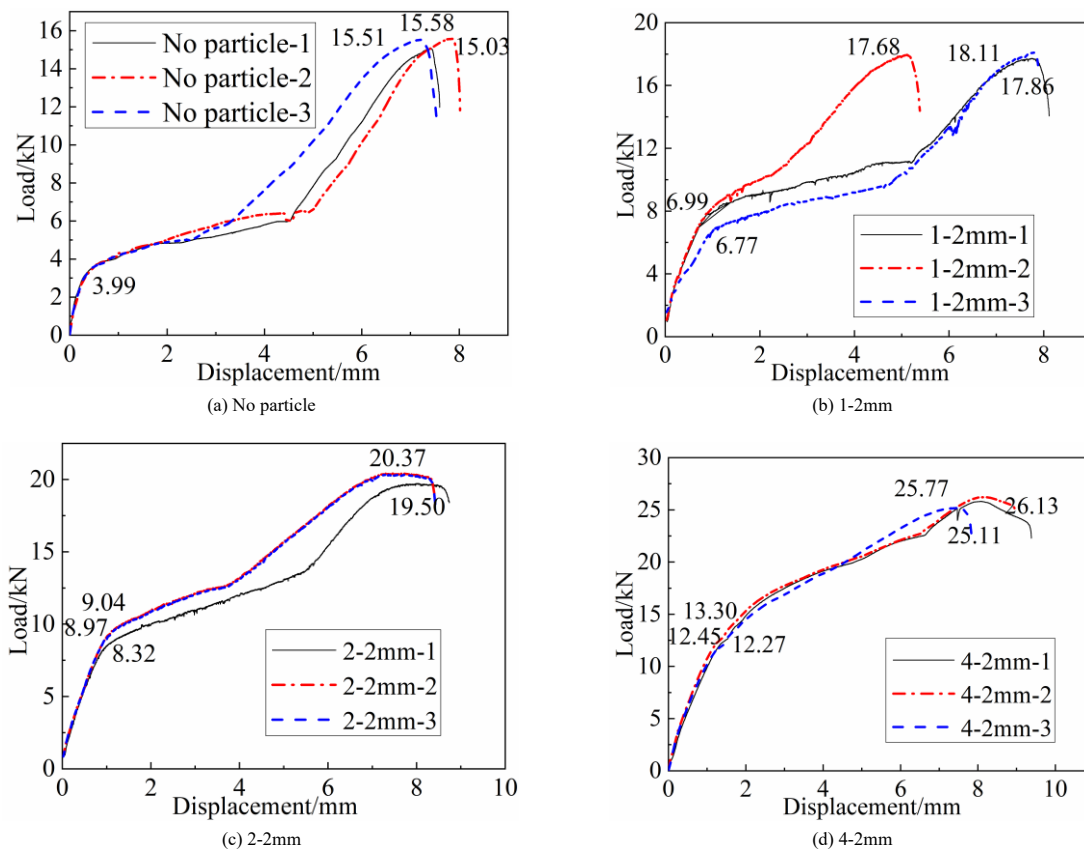


Fig. 15 Load-displacement curves of anti-slip connections with different numbers of particles

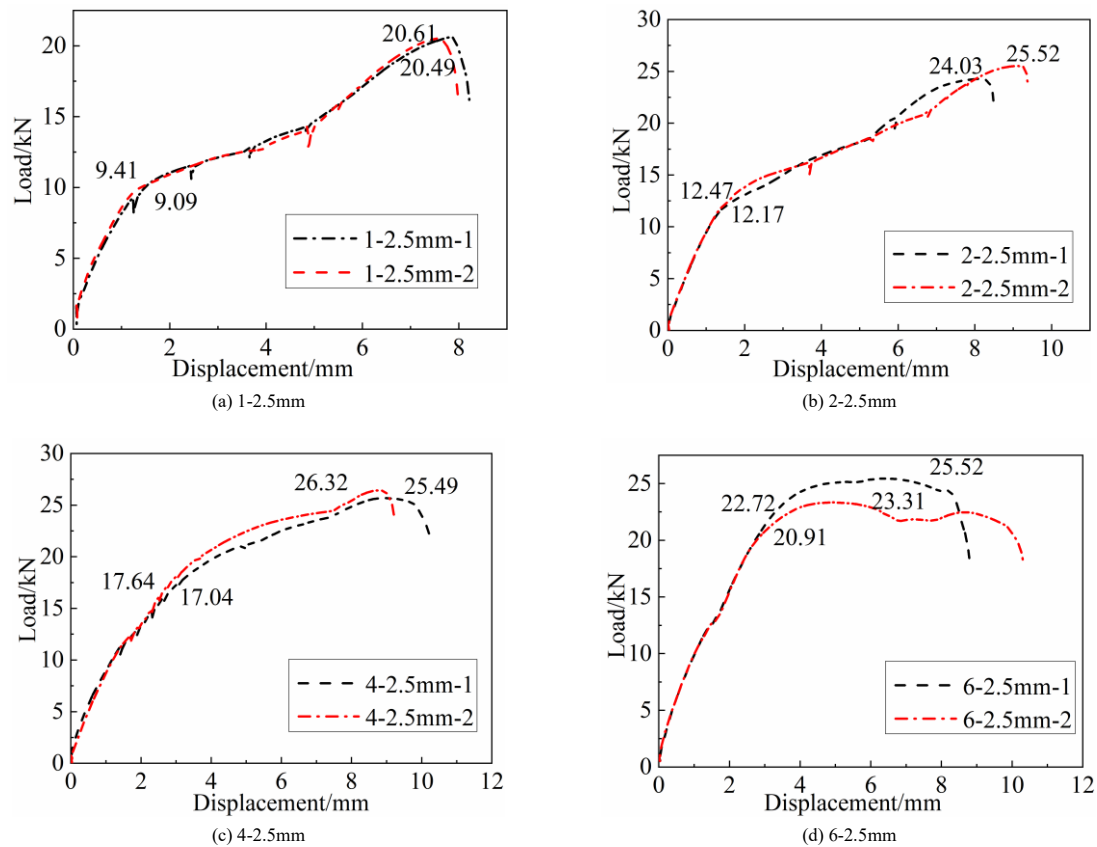
Table 6

Test data on anti-slip connections with different numbers of embedded-in particles

2mm particle		No particle	1 particle	2 particles	4 particles
Critical load F_{sc} /kN	1	3.39	6.99	8.32	13.30
	2	3.39	6.99	9.04	12.45
	3	3.39	6.32	8.97	12.27
	Average	3.39	6.77	8.78	12.67
Anti-slip coefficient μ		0.40	0.79	1.02	1.48
Ultimate load F_{su} /kN	1	15.03	17.68	19.50	25.77
	2	15.51	17.86	20.37	26.13
	3	15.58	18.11	20.37	25.11
	Average	15.37	17.88	20.08	25.67

The load-displacement curves of the anti-slip tests for connections with particles of different quantities embedded into the steel plate surface to a depth of 2.5mm are shown in Fig.16. The specific data are shown in Table 7. After embedding 1, 2, 4, and 6 particles with a diameter of 2mm, the anti-slip coefficients increased by 170%, 260%, 405%, and 535% respectively. A comparison of the data shows that the "anti-slip efficiency" of the connection

after embedding particles with a diameter of 2.5mm into the steel plate is equivalent to twice the effect achieved by embedding particles with a diameter of 2mm. Moreover, the connection has the advantages of simple operation, stable force bearing, and remarkable effect during the processing and the process of bearing slip load. To explore the limit of particle size under this scenario, the test continued to try increasing the particle size.

**Fig. 16** Load-displacement curves of anti-slip connections with different numbers of particles**Table 7**

Test data on anti-slip connections with different numbers of embedded-in particles

2.5mm particle		1 particle	2 particles	4 particles	6 particles
Critical load F_{sc} /kN	1	9.09	12.17	17.04	20.91
	2	9.41	12.47	17.64	22.72
	Average	9.25	12.32	17.34	21.82
Anti-slip coefficient μ		1.08	1.44	2.02	2.54
Ultimate load F_{su} /kN	1	20.61	24.03	25.49	23.31
	2	20.49	25.52	26.32	25.41
	Average	20.55	24.78	25.91	24.36

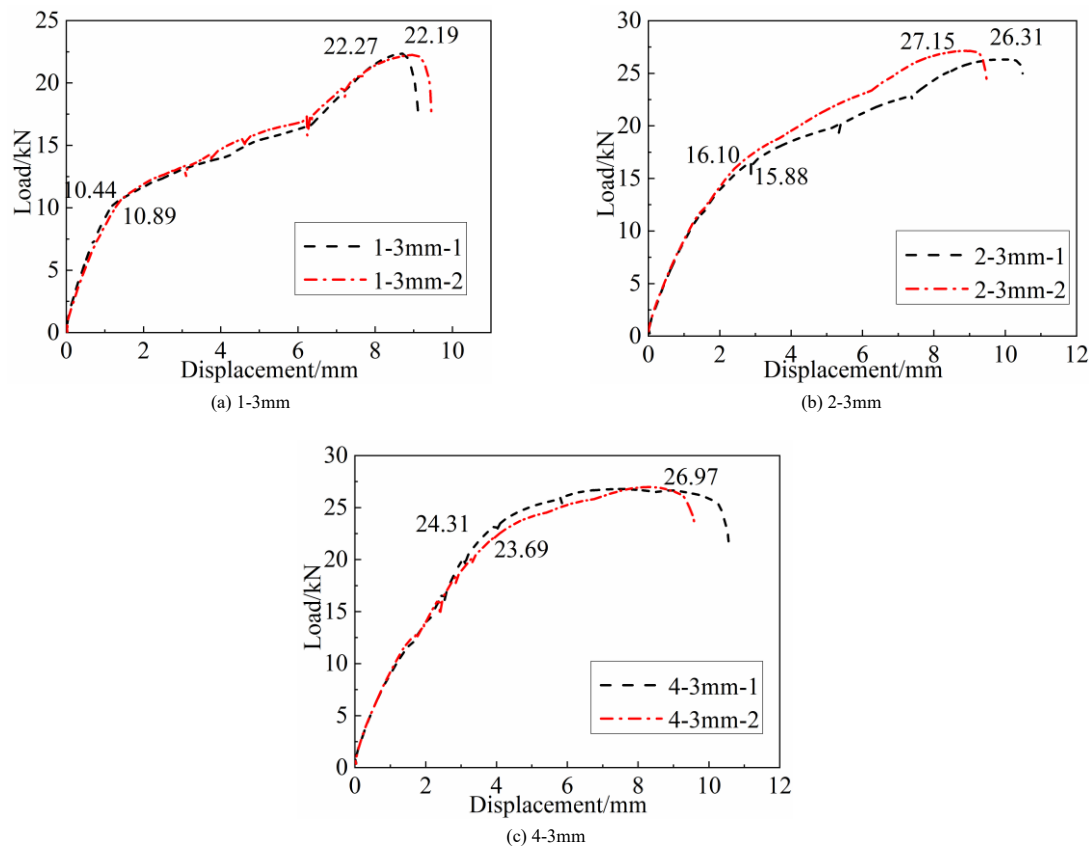


Fig. 17 Load-displacement curves of anti-slip connections with different numbers of particles

Table 8

Test data on anti-slip connections with different numbers of embedded-in particles

3mm particle		1 particle	2 particles	4 particles
Critical load F_{sc} /kN	1	10.44	15.88	23.69
	2	10.89	16.10	24.31
	Average	10.67	15.99	24.00
Anti-slip coefficient μ		1.24	1.86	2.80
Ultimate load F_{su} /kN	1	22.27	26.31	26.79
	2	22.19	27.15	26.79
	Average	22.23	26.73	26.79

The load-displacement curves of the anti-slip tests for connections with particles of different quantities embedded into the steel plate surface to a depth of 3mm are shown in Fig.17. The specific data are shown in Table 8. After

embedding 1, 2, and 4 particles with a diameter of 3mm, the anti-slip coefficients increased by 210%, 365%, and 600% respectively. The anti-slip effect of particles with a diameter of 3mm has been significantly improved compared with that of particles with a diameter of 2.5mm. After embedding 4 particles into the steel plate, the anti-slip coefficient of the connection reached 2.80, which is much higher than the effect obtained by conventional treatment methods on the surface of the connecting plate.

Compared with small-diameter particles, the plow furrows left on the surface of the steel plate after the failure of the connection with 3mm-diameter steel balls are more obvious, and the accumulations formed by pushing the surface of the steel plate are also higher. Due to the limitation of the selected thickness of the connecting plate, it is not advisable to continue increasing the size of the particles to avoid invisible internal damage to the connecting plate, as shown in Fig.18. Under repeated loads, the connection has good stability: the particles do not loosen, the indentation deformation is small, and the pre-tension loss is low, which can meet the long-term service requirements of industrial structures. The minimum ratio of plate thickness to particle diameter (t/d) is recommended to be ≥ 1.7 . At this ratio, the connecting plate has no internal damage and the anti-slip performance is stable.

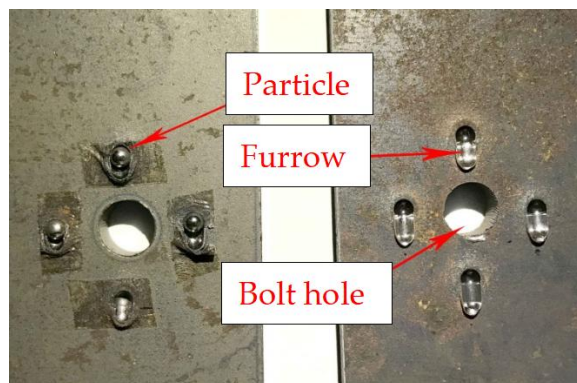


Fig. 18 Furrow on the plate and embedded particle

4.2.2. Anti-slip test of connections under a bolt pre-tension of 21.32kN

Considering the influence of different pretension forces on the new type of connection, the test added a pretension force of $P = 21.32$ kN as the control

group. The load-displacement curves of the anti-slip tests for connections with particles of different quantities embedded into the surface to a depth of 2mm are shown in Fig.19. The specific data are shown in Table 9.

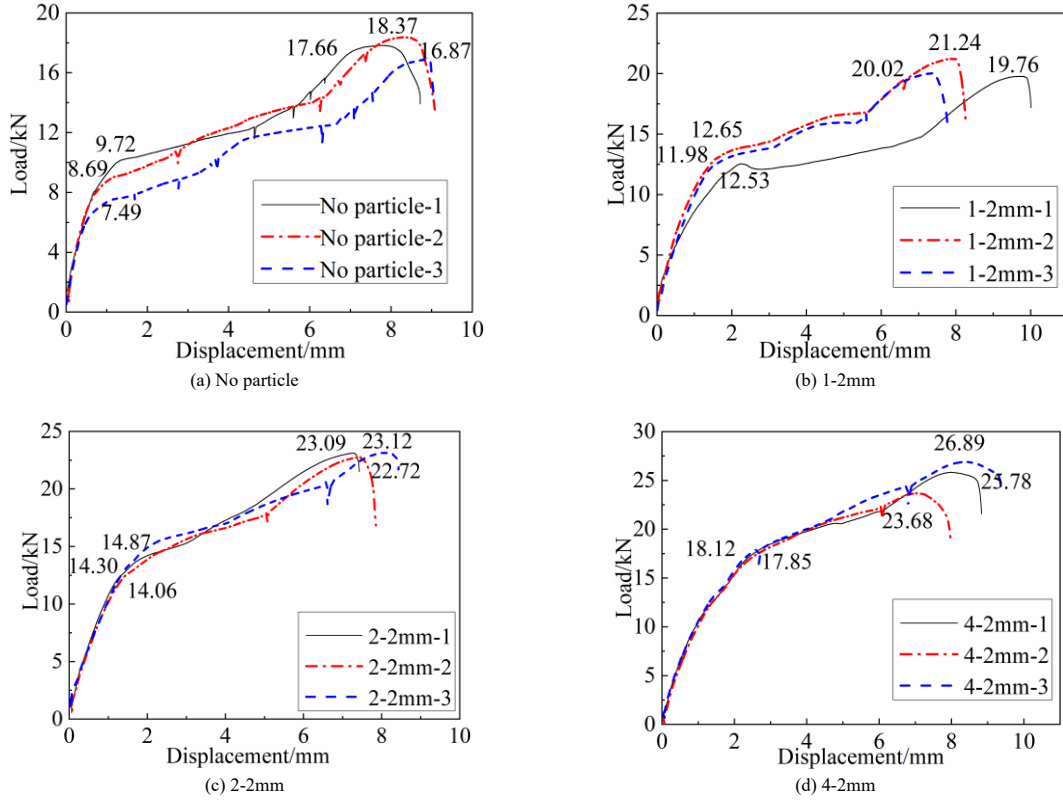


Fig. 19 Load-displacement curves of anti-slip connections with different numbers of particles

Table 9

Test data on anti-slip performance of connections with different numbers of embedded-in particles

2mm particle		No particle	1 particle	2 particles	4 particles
Critical load F_{sc} /kN	1	9.72	12.53	14.30	17.85
	2	8.69	12.65	14.06	17.85
	3	7.49	11.98	14.73	18.12
	Average	8.63	12.39	14.36	17.79
Anti-slip coefficient μ		0.40	0.58	0.67	0.84
Ultimate load F_{su} /kN	1	17.66	19.76	23.09	25.78
	2	18.37	21.24	22.72	23.68
	3	16.87	20.02	23.12	26.89
	Average	17.63	20.34	22.98	25.45

A comparison of Tables 6 and 9 shows that with the increase of pretightening force, the anti-slip coefficient of the connection without embedded particles remains unchanged, which is in line with the law of friction, and both the critical load and ultimate load increase. After embedding 1, 2, and 4 particles of 2mm into the connecting plate, the anti-slip coefficients increased by 45%, 67.5%, and 110% respectively.

4.3. Analysis of connection anti-slip test

4.3.1. Influence of particle diameter on anti-slip load of the new connection

A vertical comparison of the data in Table 10 reveals that when the pretension force of the connection is fixed, the anti-slip load of the connection increases significantly as the particle diameter increases.

Table 10

The anti-slip load of the new type of connection

Pretension /kN	Particle diameter d/mm	1 particle	2 particles	4 particles	6 particles
8.58	2	6.77	8.78	12.67	
	2.5	9.25	12.32	17.34	21.82
	3	10.67	15.99	24	

To process the critical load data more clearly, this research divides the load into the sum of the friction force N_f between the connecting plate surfaces and the anti-slip load N_{pv} provided by the particles, which is expressed by Eq.(2):

$$F_{sc} = N_f + N_{pv} \quad (2)$$

Where, F_{sc} -Critical anti-slip load; N_f -Friction force between the connecting plate surfaces (the anti-slip load of the connection when no particles are embedded in); N_{pv} -Anti-slip load provided by the particles;

Since this paper mainly studies the anti-slip load N_{pv} provided by particles, the critical anti-slip load F_{sc} data in Table 10 is subtracted by the friction force N_f between the connecting plate surfaces to obtain Table 11.

Table 11

The anti-slip load of the new type of connection

Pretension/kN	Particle diameter d/mm	1 particle	2 particles	4 particles	6 particles
8.58	2	3.38	5.39	9.28	
	2.5	5.86	8.93	13.95	18.43
	3	7.28	12.60	20.61	

The slip resistance of the particle-embedded connection is similar to that of the woodruff key, where the particles play a role of "locking" to prevent relative slip between the connecting plates. However, the woodruff key analogy has limitations, so the "three-dimensional constrained particle shear model" is proposed to clarify the slip resistance mechanism: the slip resistance of particles comes from the shear deformation and the plastic flow resistance of the material around the indentation, Eq.(3) is derived:

$$f_{pv} = \frac{8N_{pv1}}{\pi d^2} \quad (3)$$

Where, N_{pv1} -Anti-slip load provided by a single particle (N); d -Particle diameter (mm); f_{pv} -Strength of the contact area of the indentation on the surface of the connecting plate (MPa).

The strength of the contact area of the indentation on the connecting plate surface was calculated for single particles with different diameters, based on Eq.(3), as shown in Table 12.

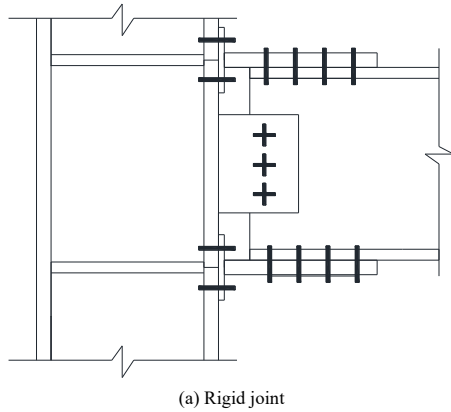
Table 12

Contact area strength of indentations on the surface of connecting plates

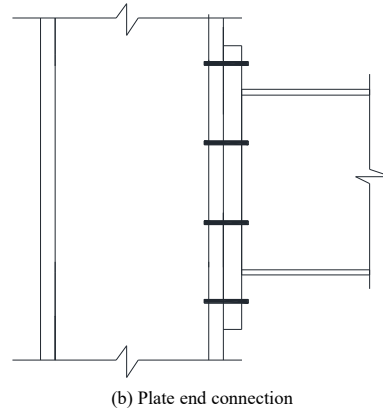
Particle diameter d/mm	2	2.5	3
Strength f_{pv}/MPa	2149.65	2387.58	2058.41

To further understand the strength of the contact area of the indentation on the surface of the connecting plate, reference is made to the concept of the bearing strength of hole walls involved in the bearing connection of high-strength bolts. In Chapter 2, a tensile test on the mechanical properties of the connecting steel plate was conducted, the lower yield strength of the connecting steel plate is 457.06 MPa, and the minimum tensile strength is 539.51 MPa. The bearing strength of the steel plate is only related to the tensile strength of the steel plate, and has no direct connection with the yield strength of the steel plate. According to the provisions in GB50017-2017 [26], the hole wall bearing strength of several steels from Q235 to Q460 can be calculated by Eq.(4).

$$f_c = 1.26 f_u \quad (4)$$



(a) Rigid joint



(b) Plate end connection

Fig. 20 Bolted connections subjected to tension

Therefore, the research on the influence of changes in pretension force on the anti-slip performance of the new type of anti-slip connection with particles embedded into ordinary steel is particularly important. The data on the anti-slip load of the connection under different pretension forces are analyzed and shown in Table 13.

Table 13

Anti-slip loads of connections under different pretension conditions

Pretension/kN	Particle diameter d/mm	No particle	1 particle	2 particles	4 particles
8.58	2	3.39	6.77	8.78	12.67
21.32	2	8.63	12.39	14.36	17.79

To more clearly analyze the shear resistance effect of particles in the connection, Eq.(2) is used to derive $N_{pv} = F_{sc} - N_f$. After excluding the factor of frictional resistance between the connecting plates, Table 14 is obtained.

Table 14

The anti-slipping load provided by the particles

Pretension/kN	Particle diameter d/mm	1 particle	2 particles	4 particles
8.58	2	3.38	5.39	9.28
21.32	2	3.75	5.73	9.31

A comparison of the data in Table 14 reveals that the magnitude of the bolt pretension has little impact on the anti-slip effect exerted by the particles. When a connection is subjected to tension under external forces, the pretension of high-strength bolts will be partially released, leading to a decrease in the frictional force between the connecting plates. However, for the new type of connection, the particles can still exert a strong shear resistance effect at this time, ensuring the effectiveness of the connection. This reflects that the new

The obtained hole wall bearing strength is 679.78 MPa, which is much lower than the data in Table 12. The strength of the connecting plate in the contact area of the particle indentation, under the environment of triaxial compression, is much higher than its bearing strength, which fully ensures the anti-slip performance of the particles.

4.3.2. Influence of bolt pretension force on anti-slip load of the new connection

Stress relaxation is a common phenomenon in metal material connections [27]. After the final tightening of high-strength bolts, their pretension will lose to a certain extent due to the influence of plate thickness, bolt diameter, temperature, time, etc., resulting in a reduction in the clamping force of the components. Especially for friction-type high-strength bolts, this will cause a great loss of their shear resistance. On the other hand, T-type connections are often used in steel structure connections, such as rigid joint connections of frame structures, end plate connections, etc. These connections bear a certain bending moment while bearing slip loads [28], which increases the internal force of some bolts [29-30], as shown in Fig.20. Since the tension in these connections usually does not act exactly on the bolt axis, but is transmitted through plates, angle steels or T-section steels that are in vertical contact with the bolt shank, the clamping force between them and the connecting plate is reduced, the friction force is decreased, and thus the shear resistance of the entire connection is lowered.

anti-slip connection structure with embedded particles can better ensure the stability of the connection's anti-slip load compared with high-strength bolt friction-type and high-strength bolt bearing-type connections.

4.3.3. Influence of the number of particles on the anti-slip load of the new connection

A horizontal comparison of data in Table 11 shows that, unlike the particle embedding test results, the anti-slip load provided by particles in the connection tests does not increase proportionally with the number of particles. After an in-depth analysis of the data, the structure of the new connection, and the test method, the following two main influencing factors are summarized: (1) The impact of the increase in the number of particles on the increase in the resilience of the connecting plate. After the bolt is tightened, both the steel balls and the steel plate have a certain degree of elasticity, which leads to a decrease in the connection pretension and the clamping force of the steel plate on the particles, which may result in a reduction in the shear resistance effect of the particles. Regarding the influence of pretension, the anti-slip loads provided by particles under different pretensions are compared, as shown in Table 14. When the pretension increases by 2.48 times, the anti-slip load provided by a single particle only increases slightly (10% when 1 particle is embedded in, 3% when 2 particles are embedded in, and 0.08% when 4 particles are embedded in), and the increase tends to decrease as the number of particles increases. Moreover, in horizontal comparison, after the pretension is increased, the anti-slip load provided by a single particle still decreases significantly as the number of particles increases. This indicates that pretension is not the main reason for the decrease in the anti-slip effect of particles. (2) The impact of the connection method. To effectively determine the anti-slip performance of embedded particles and reduce other influencing factors, this test adopts a typical single-shear pre-tightened bolt connection form. However, when this connection method is used in the connection anti-slip test, since the direction of the shear force is not collinear with the length direction of the connecting plate, a bending moment will be generated between the two connecting plates, which increases with the increase of the test shear force, and the warping of the plate end gradually increases. When the plate is long, the impact of this phenomenon on

conventional connections is not obvious, but for the new connection in this research, when the bending moment is large enough to cause a gap between the two connecting plates, the particles no longer fit with the embedded holes of the connecting plate, and the area of the particles bearing the slip load is greatly reduced, as shown in Fig.21. With the significant reduction in the stressed area, the anti-slip effect of a single particle is obviously reduced.

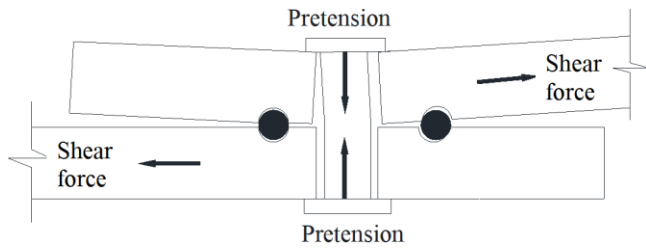


Fig. 21 The force-bearing part of the particle changes from a semicircular surface to a line

Analysis of the test phenomena reveals that when the number of particles embedded into the connecting plate is small, the depth of the plow furrows on the plate observed after the completion of the connection anti-slip test is approximately equal to the depth of the original embedded area of the particles, as shown in Fig.22. This indicates that when the particles bear the slip load, their acting surface remains relatively intact, and the gap between the connecting plates has no significant impact on the particles, allowing them to fully exert their anti-slip effect.

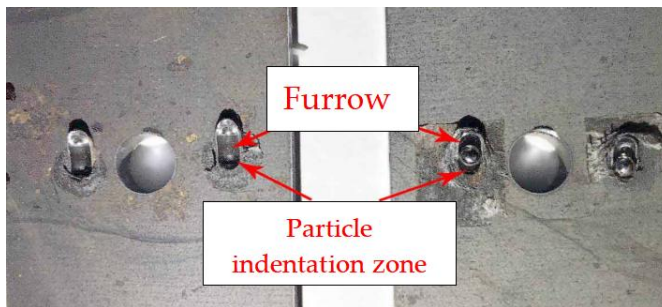


Fig. 22 Depth of furrow on the plate with little number of embedded particles

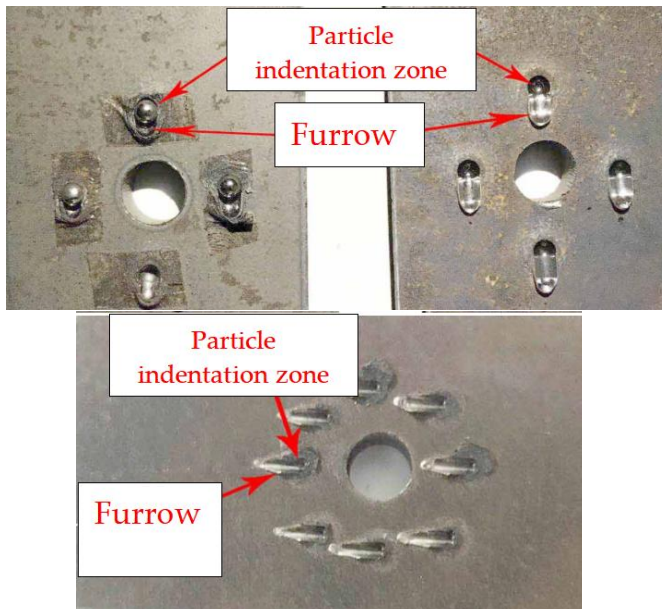


Fig. 23 Depth of furrow on the plate with large number of embedded particles

A comparison of Fig.23 shows that as the number of particles increases, after the connection is subjected to tension, the warpage of the plate end gradually intensifies. At this time, the number of exposed particles and their exposed parts increase, the depth of the plow furrows gradually decreases, and the particles as well as the local plate material around the embedded holes are no longer in a state of triaxial compression. This leads to a reduction in the anti-slip effect formed by a single particle. It is suggested that in subsequent studies, a better operation method for embedding particles should be developed to facilitate the use of a double-shear connection form, thereby reducing the impact

of bending moments during the process of the connection bearing slip loads. Increasing the thickness of the connecting plate and the diameter of the bolts can eliminate the influence of the warping of the connecting plate end, making the test more in line with engineering practice and preparing for the standardization of the new connection in the future. After comprehensive analysis, the adoption of the single-shear pre-tightened bolt connection form will have a certain impact on the test results, causing the effect of particles to not be fully exerted during the test. However, this does not affect the property that embedding particles into the connecting plate can improve the overall anti-slip performance of the connection. Even under the condition that the effect of the new connection is reduced, embedding 4 particles with a diameter of 3mm into the connecting plate can make its anti-slip coefficient reach 2.8, which is 5 times the anti-slip coefficient achieved by other current steel plate surface treatment methods, as shown in Table 15 [23,26]. This indicates that the new connection has extremely high value.

Table 15

Comparison of slip resistance coefficients of different surface treatment methods

Surface treatment method	Anti-slip coefficient (μ)
Grinding with a grinding wheel	0.35–0.45
Sandblasting/shot peening	0.40–0.50
Applying inorganic zinc-rich paint after sandblasting	0.45–0.55
Generating red rust after sandblasting	0.40–0.50
Proposed particle embedding (4×3mm particles)	2.80

4.3.4. Fatigue life consideration

Indentations formed by particle embedding may act as stress concentration sources, which may affect the fatigue life of the connection under cyclic loads. It was observed in the experiment (Section 3.2) that there were no cracks at the edge of the indentation, and the surface of the connecting plate was in a triaxial compressive stress state after particle embedding (Section 4.3.1), which could partially inhibit stress concentration. Referring to the GB50017-2017 specification and relevant literature, the influence of the indentation size (maximum depth $\leq 3.5\text{mm}$) on the fatigue life of Q345B steel can be controlled within 10%. In engineering applications, it is recommended that particles be arranged away from welds and high stress concentration areas, and the edges of indentations be slightly ground with a grinding wheel (roughness $R_a \leq 3.2\mu\text{m}$) to further reduce fatigue risks.

The particle-embedded connection shows high slip resistance coefficient and stable load-bearing capacity. The slip resistance mechanism is mainly related to the shear deformation of particles and the plastic flow resistance of the surrounding material of the indentation. Through reasonable design and construction measures, the influence of indentations on fatigue life can be effectively controlled.

4.3.5. Durability and corrosion resistance

The coupling of dissimilar metals (GCr15 particles and Q345B steel) may cause galvanic corrosion, so it is necessary to discuss the performance in humid environments and corresponding protective measures. The potential difference between GCr15 and Q345B is small ($\leq 0.1\text{V}$), so the risk of galvanic corrosion is low. The tiny gaps formed after particle embedding can be sealed by applying inorganic zinc-rich paint to avoid water vapor intrusion. In marine or high-humidity environments, it is recommended to use galvanized bolts with sealants to further improve corrosion resistance.

4.3.6. Double-shear connection performance

The single-shear test adopted in this study has the problem of warpage (prying force) of the connecting plate under load. It is necessary to discuss the performance of this problem in the actual double-shear connection. In the actual double-shear connection, the connecting plates on both sides of the bolt are symmetrically stressed, so the warpage (prying force) is significantly inhibited, the fit between particles and indentations is higher. In terms of force flow characteristics, in the double-shear connection, the particles uniformly bear the shear force, and there is no local exposure phenomenon. The anti-slip load increases approximately linearly with the number of particles, which is in contrast to the nonlinear growth in the single-shear test. This linear growth characteristic is more conducive to the design and calculation of engineering applications.

5. Suggestions for anti-slip connection design

The particle-embedded slip-resistant connection proposed in this study exhibits excellent anti-slipping performance, which provides a new idea for improving the slip resistance of steel structure connections. To promote industrial application, two feasible industrial-grade application schemes are proposed: Scheme 1 (factory pre-embedding): During the production or processing of steel plates, special molds are used to position the particles, and GCr15 steel balls are pressed into the preset positions by automatic pressure equipment (the pressing parameters are controlled according to the fitting formula in Section 3.3), ensuring uniform particle embedding depth and avoiding on-site operation errors. Scheme 2 (particle-integrated gasket): A high-strength steel gasket with reserved particle grooves is designed, and GCr15 particles are pre-fixed in the gasket grooves. During on-site installation, only the conventional bolt connection process is required, and the particles are naturally embedded into the surface of the connecting plate with the bolt preload, reducing construction complexity.

Future research will focus on optimizing the design parameters of the connection, verifying the long-term service performance of the two industrial application schemes proposed above, and carrying out full-scale model tests to provide more reliable data support for engineering applications.

6. Conclusions

This paper mainly conducts experimental verification and research on the innovative concept of the new connection formed by embedding particles into ordinary steel. Particle embedding tests and anti-slip tests were carried out, and the calculation formulas were fitted. The main conclusions are as follows:

(1) With the increase in the number of particles, the embedding load basically increases proportionally. When single GCr15 bearing steel balls with diameters of 1.5mm, 2mm, 2.5mm, 3mm, and 3.5mm are embedded into the surface of the connecting steel plate respectively, the minimum embedding loads required are 4.13kN, 7.41kN, 10.95kN, 17.22kN and 22.49kN.

(2) As the diameter and quantity of embedded particles increase, the anti-slip coefficient of the connection increases significantly. After 4 particles with a diameter of 3mm are embedded into the connecting plate, the anti-slip coefficient of the connection reaches 2.8, which is 5 times that of other current treatment methods.

(3) With the increase in the number of particles, the main factor leading to the decrease in the shear resistance effect of a single particle in the particle group embedded into the steel plate is the limitation of the single-shear connection mode. That is, when the number of particles increases and the overall shear resistance of the connection increases, the warping of the connecting plate becomes more obvious, and some particles are exposed, resulting in a decrease in the shear resistance effect of a single particle.

(4) After the bolt pretension exceeds the critical value, it has little impact on the anti-slip effect of the particles. When the friction between connecting plates decreases due to the tension on the connection or the relaxation of the connection pretension, the particles can still exert their shear resistance effect. Compared to friction-type high-strength bolted connections, the new particle-embedded anti-slip connection better ensures the stability of the anti-slip load.

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ECCENTRIC COMPRESSION MECHANICAL PROPERTIES OF CONCRETE-FILLED SQUARE STEEL TUBE UNDER CORROSIVE PITTING

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ABSTRACT

This paper studies the eccentric compression of short concrete-filled square steel tube column under corrosive pitting. The random pitting is implemented in Python and the short concrete-filled square steel tube column is simulated by the finite element software ABAQUS. The results show that pitting will affect the ultimate bearing capacity of the specimen. The ultimate load of specimens with regular pitting is higher than that of specimens with random pitting. The pitting will reduce the strength of the steel tube, leading to the decrease of the longitudinal stress of the steel tube. Pitting will cause the early contact between steel tube and concrete, reducing the interaction between steel tube and concrete, and causing the reduction of the bearing capacity of the specimen. For every 0.25 increase in eccentricity, the bearing capacity will decrease by approximately 20%. As the strength of the concrete increases, the bearing capacity of the regular pitting specimens is much higher than that of the random pitting specimens, approximately by about 5%. For every 1mm increase in the thickness of the steel tube, the bearing capacity can be enhanced by approximately 8%. Increasing the strength of concrete, the strength of steel tube, and the thickness of steel tube, the bearing capacity of the regular pitting specimens is generally higher than that of the random pitting specimens.

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1. Introduction

Corrosive pitting is a common metal corrosion, as shown in Fig.1. The metal loss caused by the pitting is very small, but it can cause a serious decline in the performance and structural strength of the steel, or even the sudden

destruction of the structure^[1-3]. The occurrence of pitting corrosion in concrete-filled steel tubular (CFST) structures results in the deterioration of structural stiffness and bearing capacity, as well as a reduction in the service life of the structures.^[4-6].



Fig. 1 Metal pitting corrosion

Many experts have studied the mechanical behavior of CFST structures after corrosion^[7-10]. Xie^[11] *et al.* performed to examine how the acid rain corrosive environment influences mechanical properties of concrete-filled steel tubular (CFST) members. The results indicated that corrosion significantly decreased the yield strength, elastic modulus, and tensile strain capacity of steel plates, along with a notable decline in the ultimate strength of CFST members. Hua^[12] *et al.* conducted finite element analysis on concrete-filled steel tubular (CFST) columns subjected to the combined influence of sustained loading and corrosion. The study revealed that sustained loading causes concrete stress hysteresis and alters the strength. Zhang^[13] *et al.* investigated the effects of acid precipitation corrosion and freeze-thaw cycles on the degradation of the axial properties of concrete-filled square and thin-walled steel tube. The results showed that acid rain corrosion had more serious effects on stiffness, ductility, confinement effects and ultimate bearing capacity than freeze-thaw. Ali^[14] *et al.* tested 11 long concrete-filled square steel tube columns under uniaxial eccentric loading and examined the impacts of concrete strength, slenderness ratio, and eccentricity on parameters like buckling mode and ductility. The findings showed that the type of failure behavior of the eccentric members with high slenderness ratio was overall buckling, starting with yielding of the outer steel tube followed by cracking of the concrete on that side; the ultimate strength of the eccentric members decreased with increasing slenderness ratio and eccentricity. Hui^[15] *et al.*

performed uniaxial axial compression tests on 15 concrete-filled steel tube columns with varying concrete strengths and corrosion levels to evaluate corrosion's impact on their axial compressive properties. The study revealed that corrosion diminished the characteristics of concrete-filled steel tubular (CFST) members. However, increasing the steel tube's wall thickness can sustain high ductility and stiffness, thereby counteracting corrosion's adverse effects on the structure. Wang^[16] *et al.* established a numerical model for concrete-filled locally corroded steel tube to study the effects of corrosion position, corrosion volume rate, and corrosion area rate on the bearing capacity under eccentric loading. When the corrosion volume rate was identical, the bearing capacity of the specimen with mid-span corrosion was found to be lower than that of the specimen with end corrosion. Luo^[17] *et al.* introduced an artificially processed defect to simulate the localized corrosion on the surface of steel tube, and tested 45 CFST long columns, medium-length columns, and other columns with different slenderness ratios by different corrosion locations, corrosion volume, corrosion area, and corrosion wall thickness. The results indicated that all specimens' failure modes manifested as overall compressive-bending failure accompanied by localized outward buckling. As the corrosion volume loss increased, the load-bearing capacity, stiffness, and ductility of CFST specimens with localized corrosion decreased to different extents.

In conclusion, the current research on corroded concrete-filled steel tube mainly focuses on concrete filled circular steel tube. The research on concrete-filled square steel tube is relatively scarce. However, in real life, the usage rate of concrete-filled square steel tube is no less than that of circular ones. Moreover, the research on concrete-filled steel tube affected by pitting corrosion is even more scarce. In this paper, the regular and random pitting is implemented in Python and the short square CFST column is simulated by the finite element software ABAQUS. The load-deflection, stress distribution and interaction between different specimens under regular and random pitting are analyzed. The impacts of various parameters on mechanical properties under regular and random pitting are taken into account.

2. Constitutive model of materials

2.1. Steel tube

Based on the data in Fig.2, The elastoplastic mechanical behavior of steel tube has the same properties under tension and compression and the stress-strain relationship is:

$$\sigma = \begin{cases} E_t \varepsilon_t & (\varepsilon_t \leq \varepsilon_{te}) \\ -A\varepsilon_t^2 + B\varepsilon_t + C & (\varepsilon_{te} < \varepsilon_t \leq \varepsilon_{te1}) \\ f_{ty} & (\varepsilon_{te1} < \varepsilon_t \leq \varepsilon_{te2}) \\ f_{ty} \left[1 + 0.6 \frac{\varepsilon_t - \varepsilon_{e2}}{\varepsilon_{e3} - \varepsilon_{e2}} \right] & (\varepsilon_{te2} < \varepsilon_t \leq \varepsilon_{te3}) \\ 1.6f_{ty} & (\varepsilon_t > \varepsilon_{te3}) \end{cases} \quad (1)$$

The variables in the equation are as follows: $A = \frac{0.2f_{ty}}{(\varepsilon_{te1} - \varepsilon_{te})^2}$, $B = 2A\varepsilon_{te1}$, $C = 0.8f_{ty} + A(\varepsilon_{te})^2 - B\varepsilon_{te}$, $\varepsilon_{te} = 0.8f_{ty} / E_t$, $\varepsilon_{te1} = 1.5\varepsilon_{te}$, $\varepsilon_{te2} = 10\varepsilon_{te}$, $\varepsilon_{te3} = 100\varepsilon_{te}$, E_t is the steel's elastic modulus, f_{ty} is the steel's yield strength, f_{tu} is the steel's ultimate strength.

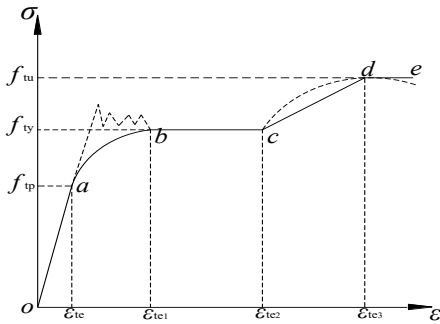


Fig. 2 Steel's stress-strain relationship

2.2. Concrete

The damage model is applied to the concrete, as illustrated in Fig.3, and the stress-strain relationship is:

$$\left. \begin{aligned} x \leq 1 & \quad y = \alpha_a x + (3 - 2\alpha_a)x^2 + (\alpha_a - 2)x^3 \\ x > 1 & \quad y = \frac{x}{\alpha_d (x-1)^2 + x} \end{aligned} \right\} \quad (2)$$

Where $y = \sigma/f_c$, $x = \varepsilon/\varepsilon_c$, f_c 、 ε_c are uniaxial compressive strength and peak compressive strain of concrete, $\varepsilon_c = (700 + 172\sqrt{f_c}) \times 10^{-6}$, $\alpha_a = 2.4 - 0.0125f_c$, $\alpha_d = 0.157f_c^{0.785} - 0.905$, $\varepsilon_{cu} = \varepsilon_c (1 + 2\alpha_d + \sqrt{1 + 4\alpha_d}) / 2\alpha_d$.

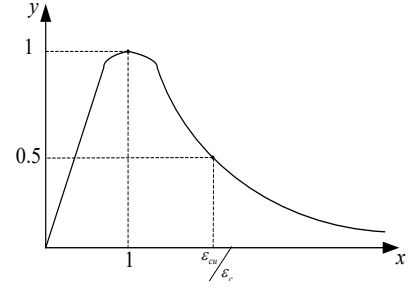


Fig. 3 stress-strain curve of concrete

3. Trial-based validation of the finite element model

The computational simulation results are verified with the test data according to the reference[18].

3.1. Failure pattern of specimen

The failure pattern of the specimen as depicted in the finite element analysis is illustrated in Figure 4. A0-a denotes the uncorroded specimen, whose failure mode is characterized by local bulging in the mid-section of the steel tube and its crushing behavior of the concrete in the compression zone. The failure pattern of specimen A3-O20-A is marked by outward bulging of the steel tube's compression area within the corrosion region, accompanied by lateral bending deformation in the same area. The maximum deflection occurs in junction of the corroded area and uncorroded area, and near the corrosion side. And the steel tube at the junction shows different degrees of buckling. The failure mode of A3-M20-a is the overall lateral bending in the middle section, and the maximum deflection is similar to that of the uncorroded specimen. It occurs in the middle area of the specimen, and the steel tube at the junction of the corroded area on both sides shows different degrees of bulging. The failure mode of A3-B20-a is lateral bending deformation at both ends of the corrosion area, and these deformation are distributed at both ends of the specimen in a symmetrical way. In the junction of the corroded area, the steel tube shows different degrees of buckling, and the concrete within the corrosion affected zone at the junction is crushed.



(a) A0-a



(b) A3-O20-a



Fig. 4 Comparison of failure modes between test and finite element model

3.2. Load-deflection curve

Fig.5 presents the load-deflection curves obtained from tests and finite element analysis. The difference of the ultimate bearing capacity of the specimen is 8.4kN, 1.0kN and 10.6kN, and the deformation difference of the middle section is 0.28mm, 1.09mm and 0mm. The finite element simulation agrees well with the test.

3.3. Load-longitudinal strain curve test

Fig.6 illustrates the load-longitudinal strain curves from both the test and finite element analysis. A high degree of consistency is observed between the test and finite element results for each specimen in the elastic and elastoplastic stages, with an overall consistent trend, confirming the feasibility of the modeling approach.

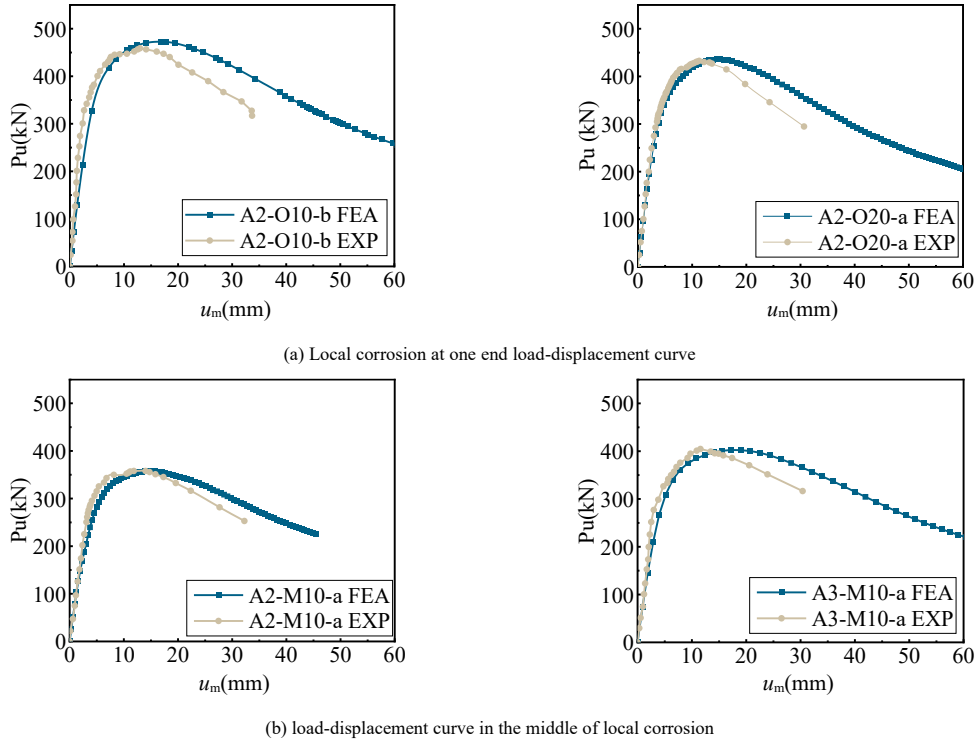
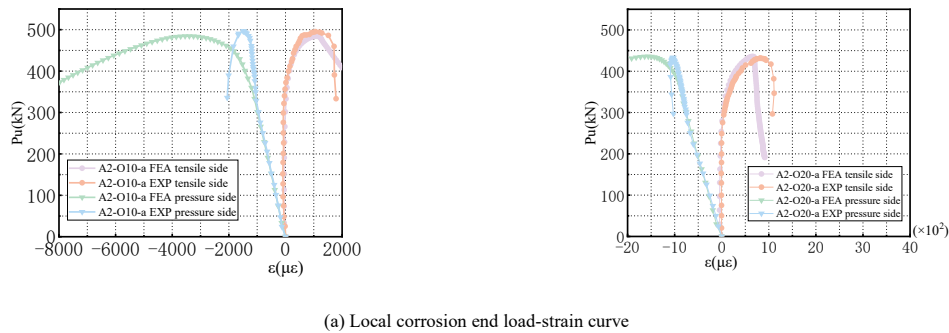
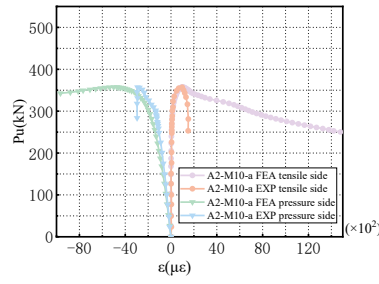


Fig. 5 Comparison of load-displacement curves

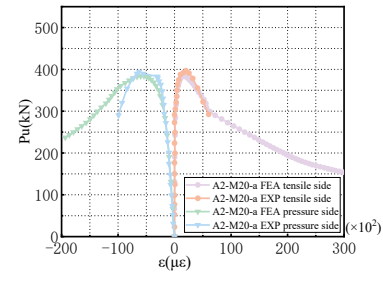


(a) Local corrosion end load-strain curve



(b) Local corrosion central load-strain curve

Fig. 6 Load-strain curve comparison



4. Realization of the random distribution of pitting

4.1. Shape and distribution of pitting

Because ABAQUS can't directly realize the random distribution of steel tube pitting, the 'RAND function' with the help of 'Python' is used to realize the secondary development of ABAQUS finite elements and complete the random distribution of pitting, as shown in Fig.7. The pitting shape is defined as the cylinder, as shown in Fig.8.

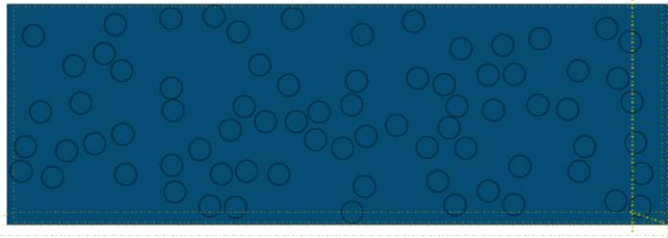


Fig. 7 Random distribution of pitting

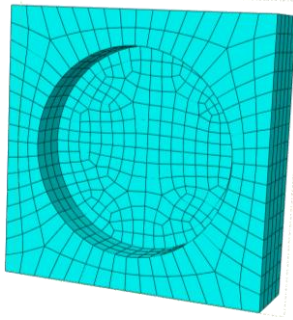


Fig. 8 Pitting shape

4.2. Test model parameter setting

To facilitate the finite element calculation, the variation in corrosion volume is achieved by adjusting the pitting depth. The corrosion volume DOV and pitting number n are used as the parameters to establish a finite element model of different pitting numbers. Corrosion volume DOV is defined as follows:

$$DOV = \frac{1}{V} \sum_{i=1}^n A_i \times h_i \quad (3)$$

Where V is the volume of the intact steel tube; N is the total number of pitting; A_i is the area of the i th pitting; h_i is the depth of the i th pitting.

In the square CFST column, the steel tube is Q345, concrete is C60, section size D is 200mm×200mm, column length L is 600mm, and steel tube thickness t is 4mm. The specimen parameters are shown in Table 1.

In Table 1, I is the intact specimen, Re is the pitting is regular arrangement, and Ra is the pitting is randomly generated. n is the total number of pitting, R is the radius of the pitting, h is the depth, V is the corrosion volume and e is

the eccentric distance. Table 2 presents the ultimate bearing capacities of each specimen. P_{Re} represents the ultimate bearing capacity of the specimen experiencing regular pitting corrosion, while P_{Ra} denotes that of the specimen affected by random pitting corrosion.

Table 1
Test specimen parameters

	number	n	R (mm)	h (mm)	DOV (%)	e (mm)
I	0	/	/	/	/	20
Re/Ra	205	100	17.3	1/2/3/4	5/10/15/20	20
	405	200	12.2			
	605	300	10			
	805	400	8.65			

Table 2
Bearing capacity of various specimens under the influence of pitting

Number	Etching pit number	Volume loss rate	P_{Re} /kN	P_{Ra} /kN
$I-0$	/	/	2267.3	2267.3
$Re-205-1$	100	5%	2187.0	2179.0
$Re-405-1$	200		2211.5	2181.1
$Re-605-1$	300		2215.4	2183.0
$Re-505-1$	400		2216.6	2186.5
$Re-205-2$	100	10%	2137.8	2121.3
$Re-405-2$	200		2178.3	2144.5
$Re-605-2$	300		2186.7	2160.5
$Re-505-2$	400		2192.0	2163.6
$Re-205-3$	100	15%	2057.8	2038.7
$Re-405-3$	200		2143.4	2083.5
$Re-605-3$	300		2162.1	2122.9
$Re-505-3$	400		2167.4	2123.8
$Re-205-4$	100	20%	1625.1	1721.3
$Re-405-4$	200		1838.5	1726.3
$Re-605-4$	300		1908.5	1760.8
$Re-505-4$	400		1933.5	1804.7

5. Analysis of regular and random pitting

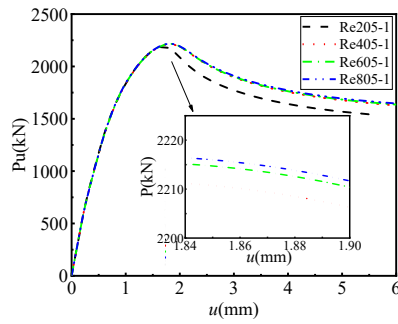
5.1. Load-deflection curve

Fig.9 presents the load-deflection curves for specimens with different regular and random pitting. For the specimens with regular pitting, corresponding to volume losses of 5%, 10%, 15%, and 20%, respectively. As corrosion progresses, the bearing capacity of the specimen decreases. Ultimate load of the specimens named $Re-205$ decreases obviously. When the corrosion is 5%, 10%, 15% and 20%, the bearing capacity decreases 60.8kN, 110.0kN, 190.0kN and 622.7kN, respectively. The results indicate that specimen's ultimate bearing capacity gradually deteriorates with increasing corrosion.

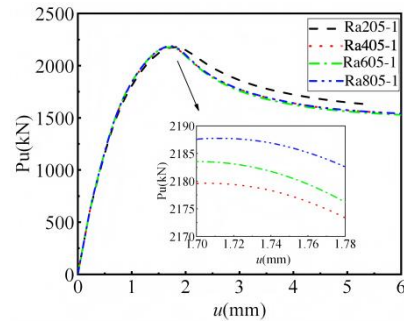
Notably, when the corrosion level reaches 20%, the bearing capacity degradation is 165% relative to that at a 15% corrosion level. This is because as the steel tube's volume loss rises, its outer cross-sectional area shrinks, reducing confinement on concrete and thus lowering the ultimate bearing capacity. Compared with regular pitting, the degradation of bearing capacity caused by random pitting will be more obvious than regular pitting with the same pitting number and corrosion rate. With the escalation of corrosion degree, the specimen's bearing capacity decline becomes more pronounced, especially when the corrosion level attains 20%. When the corrosion increased from 5% to 15%, the maximum ultimate load increment can be increased from 7.5kN to 85.1kN. However, once the quantity of corrosion pits surpasses 300, the bearing capacity of the specimen remains largely unaffected. For example, when the pitting increases from 300 to 400, the maximum ultimate load change of the specimen is about 3.5kN. When the volume loss reaches 20%, no matter the specimens are under regular pitting or random pitting, the corrosion pit penetrates the steel tube, resulting in the weakest confinement of the steel tube on the concrete and a sudden drop in the bearing capacity.

However, under the same corrosion condition, specimen's ultimate bearing capacity under regular pitting varies with the number of pitting sites. Take the Re205 for example, when the corrosion is 5%, The maximum load carrying capacity of Re405, Re605 and Re805 increases by 24.5kN, 28.4kN and 29.6kN, respectively; When the corrosion is 10%, The ultimate bearing capacity of Re405, Re605 and Re805 increases by 40.5kN, 48.9kN and 54.2kN, respectively; When the corrosion is 15%, The ultimate bearing

capacity of Re405, Re605 and Re805 increases by 80.6kN, 104.3kN and 109.6kN, respectively; When the corrosion is 20%, the ultimate bearing capacity of Re405, Re605 and Re805 increases by 213.4kN, 283.8kN and 308.4kN, respectively. This is due to the same volume corrosion, the area of single pit such as Re405, Re605 and Re805 decreases compared with Re205 in the compression area of middle section, and the constraint effect on concrete gradually increases. Consequently, it will result in an elevation of the bearing capacity. Under a constant corrosion condition, the impact of an increasing number of pitting sites on the bearing capacity will gradually diminish. When the number of pitting sites reaches 300, the variation in pitting begins to affect the stability of the specimen's bearing capacity. Take Re605 and Re805 for example, the ultimate load at each corrosion is extremely close, and the maximum difference is about 25.4kN. This is because when the number of corrosion pits gradually increases to 300, the corrosion's impact on the bearing capacity is closer to uniform corrosion. Specimen's load-carrying capacity with random pitting is significantly lower than that of specimen with regular pitting. The bearing capacity of Ra405, Re605 and Re805 is 112kN, 148kN and 120kN larger than that of Ra205, Ra405 and Ra605. However, when the pitting is 100, the bearing capacity of Re205 will be lower than that of Ra205. This may be related to the distribution of pitting. Compared with Ra205, the pitting of Re205 is near the middle part, and the reduction of cross-section area of the steel tube is more obvious, resulting in the weaker constraint of tube on the concrete.

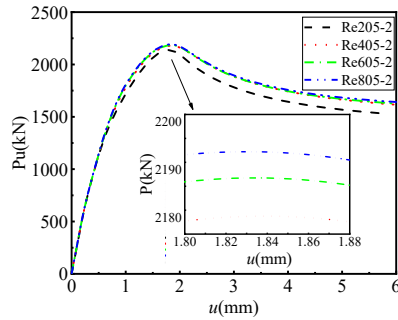


Regular pitting

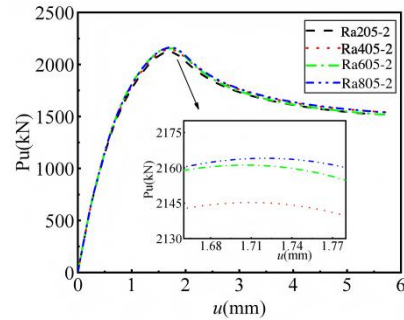


Random pitting

(a) DOV = 5%

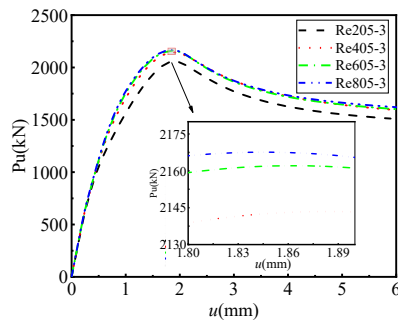


Regular pitting

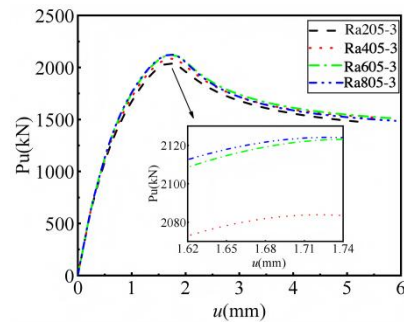


Random pitting

(b) DOV = 10%



Regular pitting



Random pitting

(c) DOV = 15%

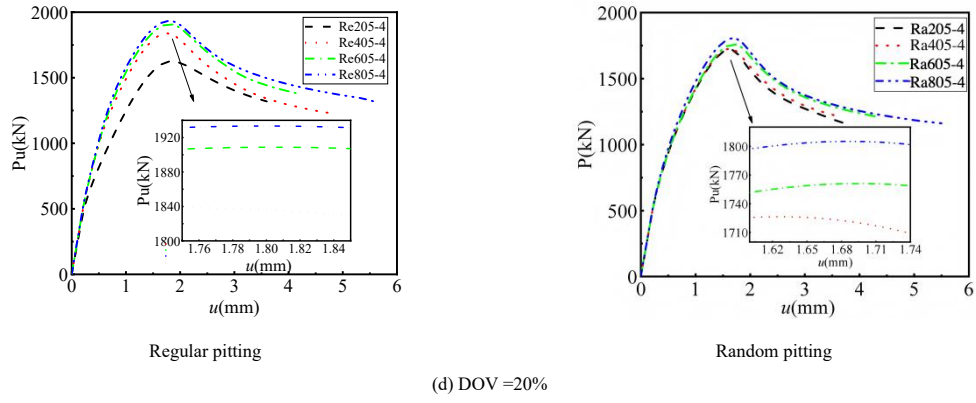
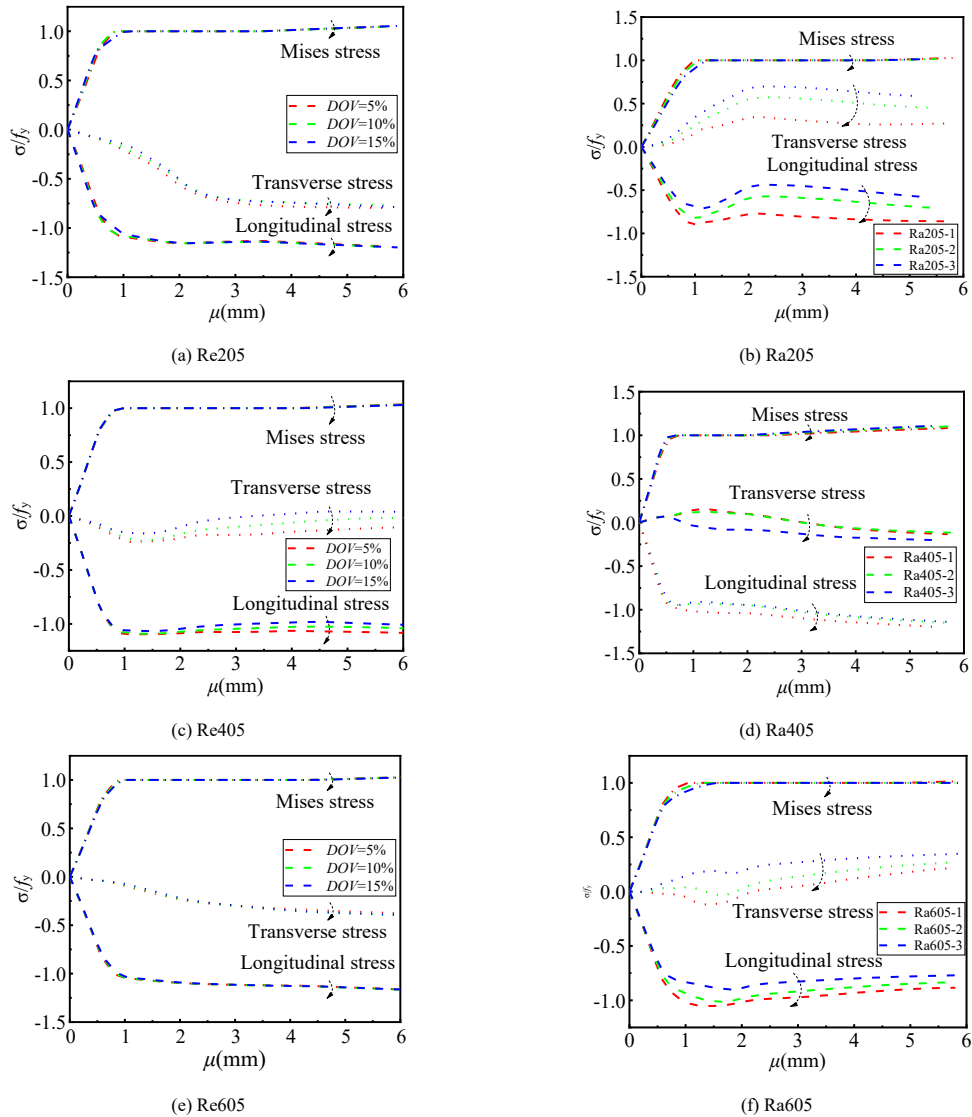


Fig. 9 Load-displacement of different DOV

5.2. Stress distribution

Fig.10 illustrates the stress-vertical displacement correlation for the steel tube. It shows that the pitting volume loss exerts a negligible impact on the Mises stress of the steel tube. For regular pitting, the development of transverse stress and longitudinal stress under the same pitting will be slightly different with the increase of corrosion. As the tube attains its yield strength, the longitudinal stress in the middle section progressively diminishes with escalating corrosion. This can be ascribed to the fact that the increasing

corrosion gradually degrades the steel tube's strength, consequently leading to a reduction in its longitudinal stress. The transverse stress of the steel tube progressively decreases with the increase in corrosion, which is due to the weakening of concrete constraint by steel tube caused by pitting. At the same time, the increase of pitting will also cause the early yield of steel tube, which the most obvious is Re205. When the corrosion is 5%, the displacement is 0.97mm when the steel tube yielded, while the corrosion is 10% and 15%, the displacement is 0.94mm and 0.85mm, decreasing by 3% and 12.4%, respectively.



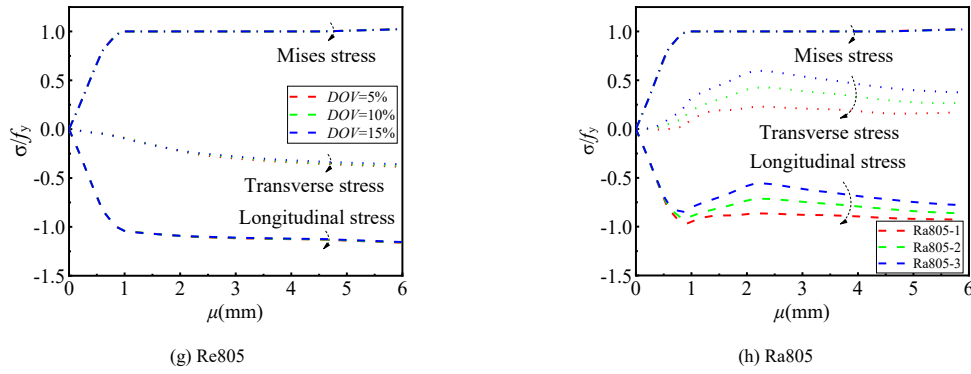


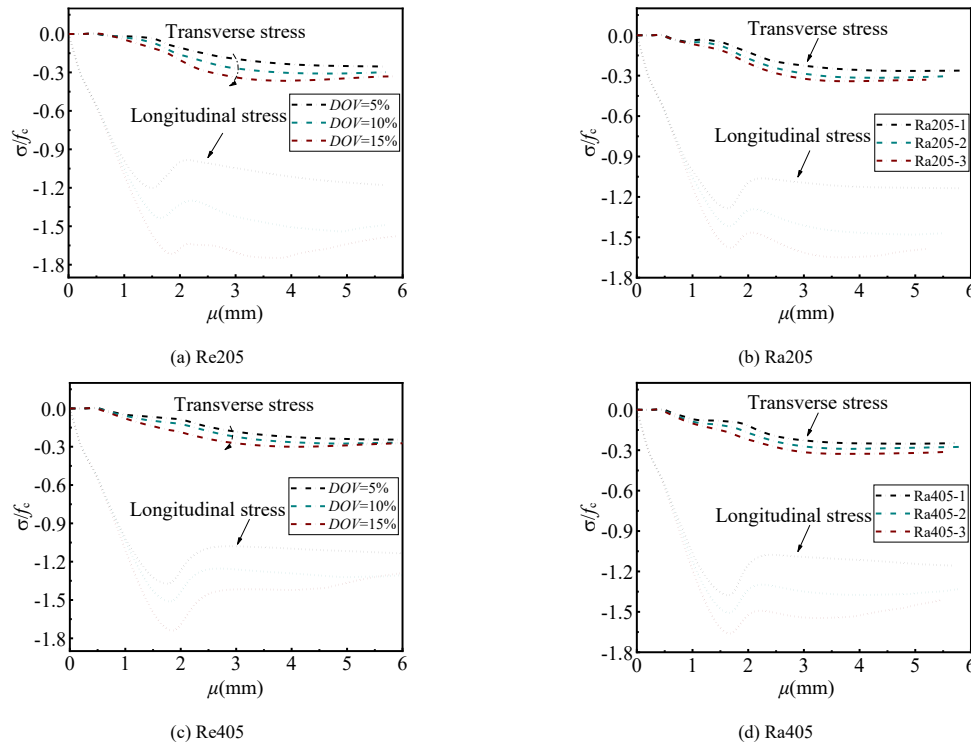
Fig. 10 Longitudinal stress-vertical displacement curve of steel tube

For random pitting, pitting volume loss significantly affects steel tube's longitudinal and transverse stresses. When the pitting ratio rises from 5% to 15%, the longitudinal stress decreases as the pitting ratio increases. The longitudinal stress of Ra205, Ra405, Ra605, and Ra805 ranged from $0.88f_y$, $0.97f_y$, $1.04f_y$, $0.94f_y$ to $0.67f_y$, $0.93f_y$, $0.90f_y$, $0.78f_y$, by about 24%, 4%, 13.5% and 17%, respectively. The transverse stress in the steel tube progressively rises as the corrosion degree increases, which is due to the increase of pitting, which strengthens the contact between the corrosion pit and concrete, leading in the increase of transverse stress. The increase of pitting will also cause the advance yield of steel tube. Compared with regular pitting, random pitting will be more obvious than regular pitting. Take Ra205 for example, the displacement of the specimen with the corrosion of 5% is 0.63mm when the steel cylindrical conduit is yielded, and the displacement of the specimen with the corrosion of 10% and 15% is 0.58mm and 0.55mm, reducing 8% and 12.7%, respectively.

Fig.11 illustrates the stress-vertical displacement relationship of concrete in the middle section. It's evident that concrete's transverse stress rises with increasing corrosion. This is attributable to the enhanced contact between the steel tube and concrete induced by the increase in pitting. Additionally, the longitudinal stress of concrete also increases as corrosion intensifies. For regular pitting, take the Re205 with 100 corrosion pits for example, concrete's longitudinal stress in the limit state with 5% of corrosion is $1.13f_c'$. Whereas the concrete's longitudinal stress with 10% and 15% corrosion is $1.41f_c'$, $1.71f_c'$, increasing by 24.8% and 51.3%, respectively. Owing to the proliferation of

pitting, the load-bearing capacity of the steel tube progressively diminishes, while the load borne by the concrete gradually rises, consequently leading to an increase in the longitudinal stress of the concrete. Simultaneously, the presence of pitting induces an uneven longitudinal stress distribution in the middle section concrete, inducing stress concentration and thereby elevating the longitudinal stress of the concrete. Additionally, the evolution of corrosion pits also influences the stress development of the concrete. At the same corrosion, the longitudinal stress of concrete will change with the change of the pitting. This is due to the increase of the pitting, which increases the effective constraint of the concrete at the middle section. At the same time, With the increase of the pitting, the pitting will be uniform at middle section, and the stress concentration will be significantly weakened.

For random pitting, take Ra205 for example, the axial-direction stress within concrete in the limit state with corrosion of 5% is $1.28f_c'$, and the longitudinal stress of concrete with corrosion of 10% and 15% of the specimens is $1.42f_c'$, $1.58f_c'$, increasing by 11.2% and 23.4%, respectively. The increase of longitudinal stress of concrete under random pitting is similar to that of regular pitting, which is caused by the increase loading of concrete caused by the change of pitting, and the constraint of surrounding pitting on concrete is uneven, resulting in the stress concentration. However, different from the regular pitting, the longitudinal stress of concrete in the middle section is not much different under the same corrosion, which indicates that the change of pitting will not lead to the obvious change of the stress along the axial direction in concrete in the middle section under the random pitting.



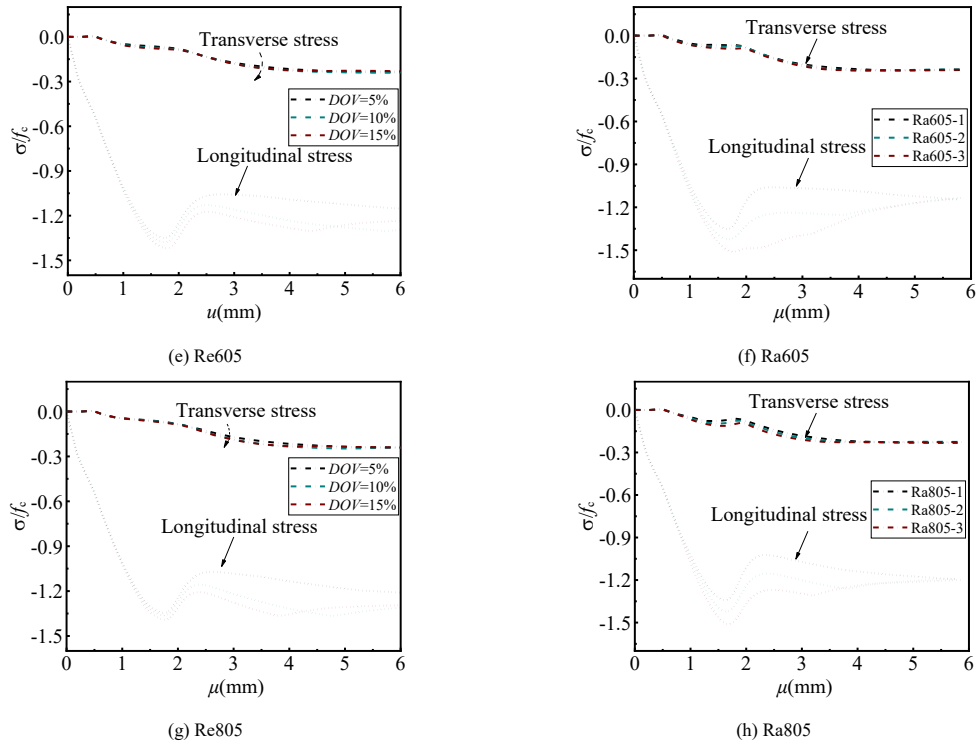
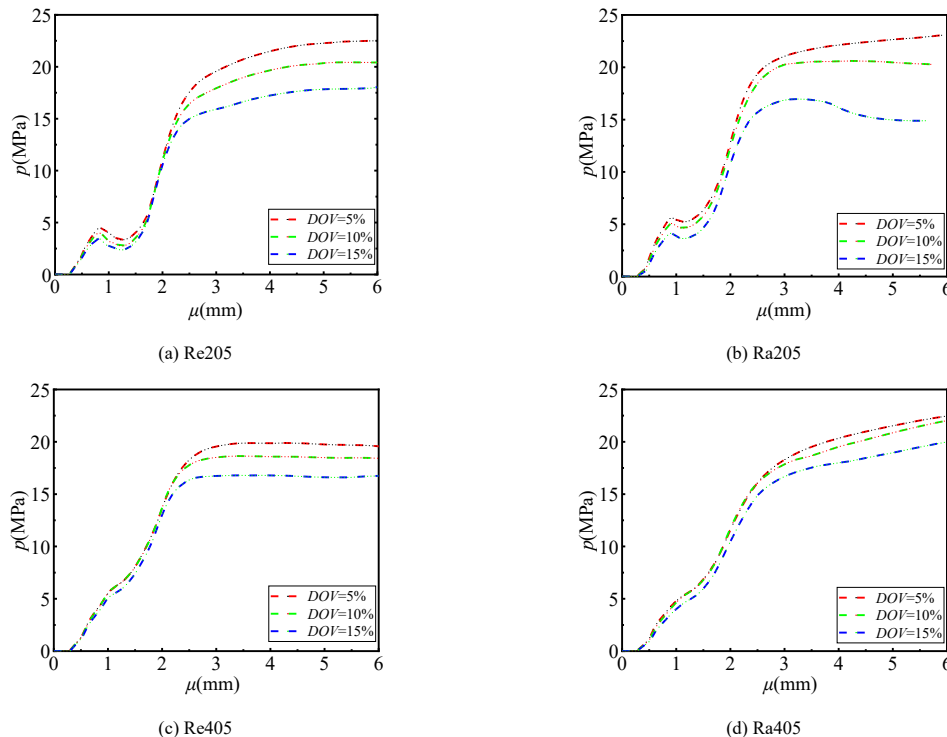


Fig. 11 Longitudinal stress-verticle displacement curve of concrete in middle section

5.3. Interaction

Fig12 illustrates the relationship between vertical displacement and contact stress for the steel tube-concrete interface. It shows that the contact stress is closely related to the corrosion degree and has the same change rule. During the initial loading phase, the contact stress at the steel-concrete interface is zero, implying no contact between the two. This stems from steel's higher Poisson's ratio than concrete's at the initial loading stage, leading to greater transverse deformation in steel relative to concrete, which results in the absence of contact between them. As the loading escalates, concrete's transverse deformation exceeds steel's, facilitating their contact. Regular pitting and random pitting trigger local steel buckling, prompting early steel-concrete contact, and will change with the change of corrosion. When the corrosion is 5%, the tube will contact with the concrete when the vertical displacement of the specimen with the pitting number of 100 reaches 0.31mm and 0.29, respectively, 6.5% compared with regular pitting. And the corrosion is 10% and 15%, the vertical displacement decreases by 6% and 10%, 5.1%

and 8.4%, respectively. Moreover, the progression of pitting corrosion will diminish the contact at the steel-concrete interface. When the corrosion is 5%, the contact stress in the limit state is about 8.9MPa and 9.75MPa, respectively, 0.85MPa increment than regular pitting. When the corrosion increases to 10% and 15%, the contact stress decreases to 6.92MPa and 5.43MPa, 8.37MPa and 6.66MPa, respectively. For regular pitting, the change of pits also has an impact on the contact stress. When the number of pits increases from 100 to 400, the contact stress increased from 8.9MPa to 10.49MPa, increasing by about 17.8%, that is the reason why the bearing capacity changes with the increase of the number of pitting. For random pitting, when the number of pitting is 300, the contact stress at the middle section is 11.3MPa, which is significantly greater than the contact stress of other number of pitting. This may be related to the distribution of pits under the random distribution. When the number of pitting is 300, there is no distribution of pitting near the value point, so the contact stress at this point is higher than that under the other number of pits.



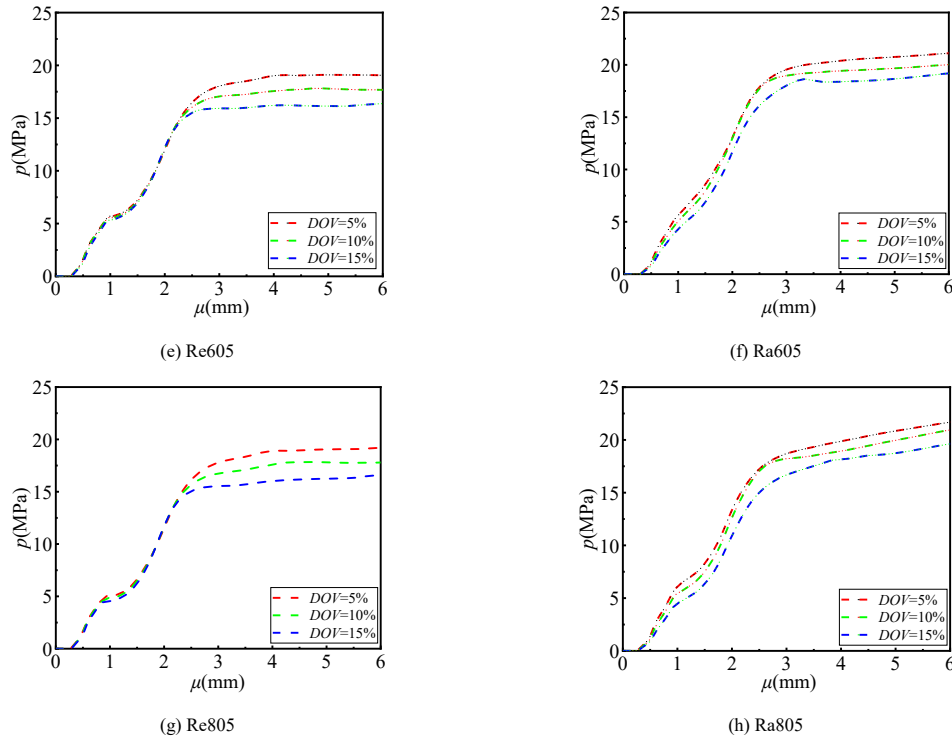


Fig. 12 Mid-section corner contact stress

6. The influence on different parameters

6.1. Eccentricity

Fig.13 presents the load-deflection curve for various eccentricities. It shows that maximum load will decrease with the increment of eccentricity. When the eccentricity increases from 0.25 to 1, maximum load of Re205-1 with regular pitting will decrease from 2187kN to 996kN, decreasing by 54.5%. When the eccentricity increases from 0.25 to 0.5, maximum load will decrease by 23.3%; when the eccentricity increases from 0.5 to 0.75, maximum load will decrease by 21.6%; when the eccentricity increases from 0.75 to 1, maximum load will decrease by 19.4%. However, maximum load of Ra205-1 under random pitting is consistent with the regular pitting. When the eccentricity increases from 0.25 to 1, the decrease ratio of maximum load is about 20.9%, 21.4% and 19%, respectively.

6.2. Concrete strength

Fig.14 illustrates the load-deflection curves for varying concrete strengths. It indicates that both the ultimate load and stiffness increase as the concrete strength rises. When the concrete's strength increases from C30 to C60, the ultimate load of Re205-1 with regular pitting will increase from 1556kN to 2186kN, an increase of 40.5%. When the strength of concrete rises from C30 to C40, the maximum load will increase by 13.8%; when concrete strength rises from C40 to C50, the ultimate load increases by 11.8%; when it rises from C50 to C60, the increase is 10.3%. However, the ultimate load of

Ra205-1 under random pitting is consistent with the regular pitting. When the concrete strength rises from C30 to C60, the increase of the maximum load is about 14.0%, 14.1% and 13.9%, respectively. The ultimate load affected by random pitting is slightly lower than that of affected by regular pitting. For each grade of concrete, the increase of the bearing capacity under the two corrosion conditions is basically consistent.

6.3. Steel strength

Fig.15 presents the load-deflection curves for varying steel strengths. It indicates that the ultimate load will rise as the steel strength increases, while the initial stiffness remains largely unchanged. When the yield strength of steel increases from 235MPa to 460MPa, the maximum load of Re205-1 with regular pitting will increase from 1920kN to 2489kN, increasing by 29.3%. When steel's strength increases from 235MPa to 345MPa, the ultimate load will increase by 13.9%; when steel's strength increases from 345MPa to 420MPa, the ultimate load of the will increase by 9.3%; when steel's strength increases from 420MPa to 460MPa, the ultimate load will increase by 4.1%. However, the ultimate load of Ra205-1 under random pitting is consistent with the regular pitting. When the steel strength increases from 235MPa to 460MPa, the increase ratio of the ultimate load is about 14.3%, 8.3% and 4.0%, respectively. The ultimate load of affected by random pitting is slightly lower than that of affected by regular pitting. For each increase of the steel, the increase of the bearing capacity under the two corrosion conditions is basically consistent.

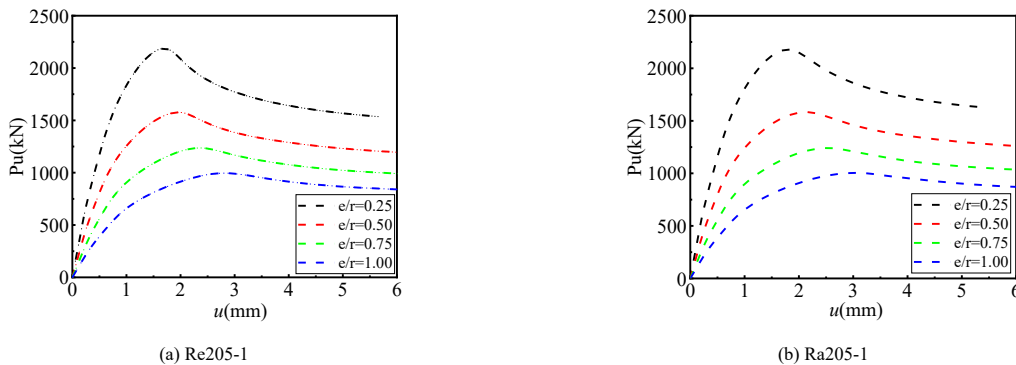


Fig. 13 Load-deflection of different eccentricities

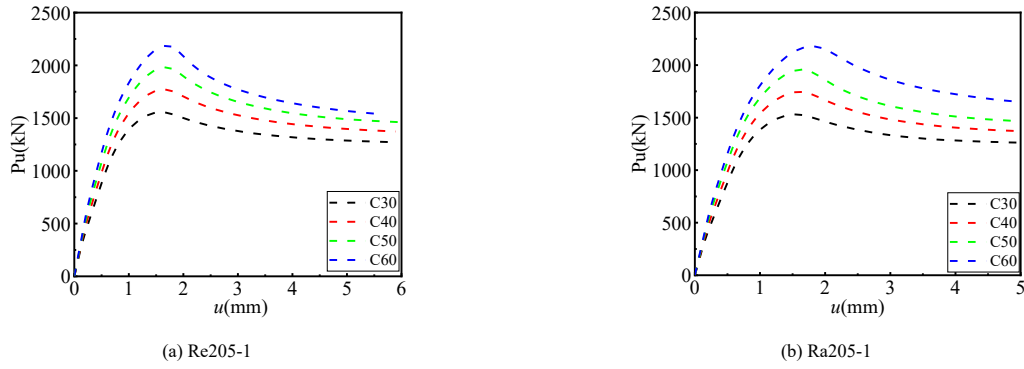


Fig. 14 Load-deflection of different concrete strengths

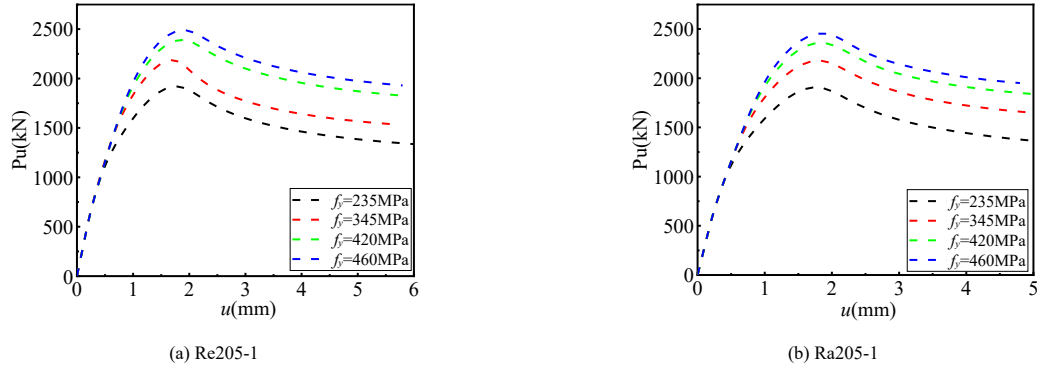


Fig. 15 Load-deflection of different steel strengths

6.4. Steel tube thicknesses

Fig.16 illustrates the load-deflection curves for varying steel tube thicknesses. It indicates that both the ultimate load and stiffness will rise as the steel tube thicknesses increase. When steel tube's thickness increases from 4mm to 7mm, the ultimate load of Re205-1 with of regular pitting will increase from 2187kN to 2750kN, increasing by 25.7%. When the steel tube's thickness increases from 4 mm to 5 mm, the ultimate load exhibits a 7.8% increase; when it increases from 5 mm to 6 mm, the ultimate load rises by

7.9%; and when the thickness goes up from 6 mm to 7 mm, the ultimate load shows a 7.3% increment. However, the ultimate load of Ra205-1 with random pitting is consistent with the regular pitting. When the thickness of the steel tube increases from 4mm to 7mm, the increase ratio of the ultimate load is about 7.5%, 7.6% and 7.0%, respectively. The maximum load affected by random pitting is slightly lower than that of affected by regular pitting. For each thickness of steel, the increase of the bearing capacity under the two corrosion conditions is basically consistent.

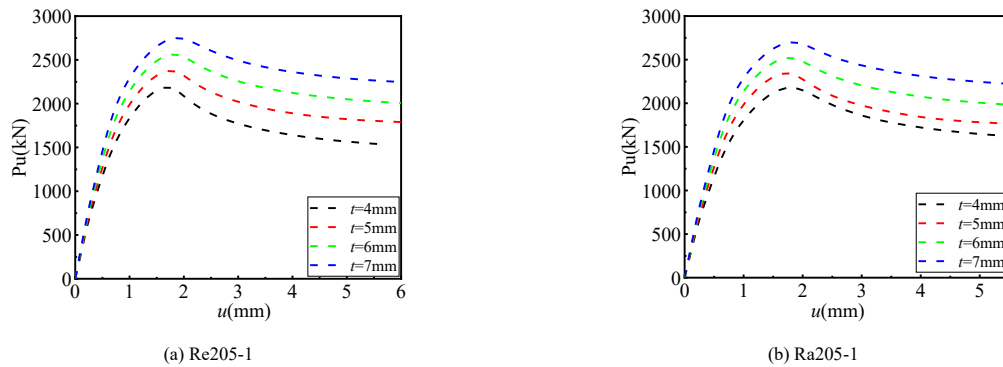


Fig. 16 Load-deflection of different steel tube thicknesses

7. Conclusions

This paper analyses the eccentric compression mechanical property of square CFST column with of regular and random pitting. And parametric analysis like eccentricity, concrete's compressive strength, steel strength and steel tube thicknesses is analyzed. The conclusions are as follows:

(1) Pitting reduces the specimen's maximum load bearing capacity. Specimens exhibiting regular pitting demonstrate a higher ultimate bearing capacity compared to those with random pitting. And under the same pitting, increasing pitting number will increase the bearing capacity, but when the pitting reach 300, the maximum bearing capacity of the specimen will remain stable.

(2) Pitting diminishes steel tube's strength, consequently resulting in a reduction of its longitudinal stress. Increased pitting enhances the

steel-concrete interface interaction, raising transverse stress. It reduces the steel tube's load, increasing the concrete's load and thus elevating its longitudinal stress. At the same time, pitting will cause the steel tube to the yield state in advance, and with the increase of pitting, this situation will become more obvious.

(3) Pitting will cause the early contact between steel tube and concrete, Weakening the steel-concrete interaction, and cause the reduction of the bearing capacity of the specimen. The steel-concrete interaction gradually diminishes as pitting increases. However, under normal pitting conditions, a rise in pitting can enhance this interaction. But this influence rule is not applicable to random pitting.

(4) Under diverse parameters, specimens with regular pitting consistently exhibit higher load bearing capacity than those with random pitting. When the eccentricity increases from 0.25 to 0.5, 0.75, and 1.0, specimens' bearing

capacity with regular pitting will decrease by 23.3%, 44.9% and 64.5% respectively. However, specimens' bearing capacity with random pitting will decrease by 20.9%, 42.3% and 61.3%, respectively. When the strength of concrete increases from C30 to C40, C50 and C60, specimens' bearing capacity with regular pitting will increase by 13.8%, 25.6% and 35.9%, respectively, while the specimens with random pitting will increase by 14.3%, 22.6% and 26.6%, respectively. When the strength of the steel increases from 235MPa to 345MPa, 420MPa and 460MPa, specimens' bearing capacity with regular pitting will increase by 13.9%, 23% and 27.3%, respectively. Specimens' bearing capacity with random pitting will increase by 14.3%, 22.6% and 26.9%, respectively. When the thickness of the steel tube increases from 4mm to 5mm, 6mm and 7mm, specimens' bearing capacity with regular pitting will increase by about 7.8%, 15.7% and 23%, respectively. Specimens' bearing capacity with random pitting will increase by 7.5%, 15.1% and 22.1%, respectively.

Conflict of interest statement

The authors state no conflict of interest exists.

Data availability

The corresponding author can provide access to the data, models, and code that substantiate the outcomes of this work upon a fair request.

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MOMENT CAPACITY PREDICTION OF HIGH STRENGTH STEEL (HSS)-ECC COMPOSITE BEAMS USING MACHINE LEARNING MODELS

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ABSTRACT

The demand for high-performance structures has led to the development of composite beams made of High Strength Steel (HSS) I-section and Engineered Cementitious Composites (ECC) slab connected through headed shear studs, which could enhance both ductility and flexural capacity compared to conventional counterpart. Accurate prediction of flexural strength in HSS-ECC composite beams is crucial for optimal design and safety. This study presents a machine learning approach, including five machine learning (ML) models (i.e., Extreme Gradient Boosting (XGBoost), Artificial Neural Network (ANN), Support Vector Regression (SVR), Categorical Boosting (CatBoost), and Random Forest (RF)) to predict the flexural capacity of HSS-ECC composite beams. A total of 135 composite beam data were compiled and utilized for training and validating the machine learning models. By using performance metrics, it is indicated that all of the employed ML models showed excellent predicted performance, among which the SVR model exhibits superior predictive accuracy by achieving the lowest prediction errors (MAE, MAPE, and RMSE) together with the highest coefficient of determination ($R^2 = 0.99$), indicating superior generalization ability. This offers a viable alternative to traditional analytical approaches for predicting structural engineering.

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Support Vector Regression (SVR)

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1. Introduction

The construction industry continuously pursues structural materials that can provide higher strength and ductility while simultaneously reducing self-weight and overall cost. High-strength steel (HSS), owing to its superior mechanical performance, has been increasingly adopted in modern steel construction. In conventional composite beam systems, a normal-strength concrete (NSC) slab is linked with normal strength steel (NSS) I-section by means of headed shear studs. However, when HSS replaces normal-strength steel in these systems, significant strain incompatibility can arise between the employed materials (i.e., HSS and NSC). This mismatch often leads to unsafe structural behavior, characterized by brittle concrete crushing at ultimate load levels [1–7]. This challenge has motivated researchers worldwide to explore alternative materials capable of working compatibly with HSS in composite beams subjected to sagging moments. Engineered Cementitious Composites (ECC) have recently appeared as a potential substitute for the NSC. Originally developed by Victor Li several decades ago [8–11], ECC has soon demonstrated an advanced cement-based material due to its superior mechanical properties. Unlike ordinary concrete, ECC exhibits high strain capacity in both tension and compression, accompanied by multiple fine cracking and effective crack-bridging behavior [12–17]. Experimental studies by the author [18] demonstrated that composite beams consisting of a HSS I-girder combined with an ECC slab (hereafter referred to as HSS–ECC composite beams) can achieve comparative flexural capacity and enhanced ductility compared to traditional composite beams constructed with NSS and NSC. To facilitate the practical adoption of this composite system in commercial steel structures, various analytical and numerical investigations, including parametric studies, have been carried out [19–20]. Nevertheless, traditional analytical and numerical methods frequently face difficulties in precisely representing the complex and strongly nonlinear response of HSS–ECC composite beams.

Recent advancements in machine learning (ML) have attracted great attention due to its ability to identify complex data patterns and construct accurate, generalizable predictive models for targeted applications [21]. One of the primary advantages of computational intelligence techniques is their strong pattern-recognition capability, which is particularly valuable in engineering problems. Although many ML models function as black-box systems, they are highly effective at identifying intricate relationships within data. In structural engineering, ML approaches have been increasingly adopted to assist decision-making and to extract useful insights from large datasets. In contrast to conventional numerical simulations, ML methods learn directly from extensive experimental and numerical data, enabling reliable predictions with minimal manual intervention. Moreover, by efficiently exploiting existing data, these approaches can reduce computational cost, experimental effort, and the need for repetitive testing [22]. Among the diverse machine learning approaches, Artificial Neural Networks (ANNs) have been extensively utilized to estimate the mechanical response and performance of concrete and steel structural

systems. Their strong capability to represent highly nonlinear relationships and to process multiple input parameters simultaneously makes them particularly well suited for complex civil engineering applications. As a result, ANNs have become one of the most commonly adopted techniques in structural engineering research. Numerous studies have demonstrated the effectiveness of ANN models in predicting different structural behaviors. Typical examples include the estimation of axial compressive strength of various column configurations [23–24], shear strength and deflection of FRP-reinforced concrete beams [25–26], bond performance between GFRP bars and concrete [27], load-carrying capacity of corroded steel beams [28], and shear resistance of perfbond strip connectors [29]. Overall, these investigations highlight the strong predictive capability of ANN-based models for evaluating the structural performance of reinforced concrete systems. In addition to ANN, other advanced machine learning techniques have also shown considerable potential. Extreme Gradient Boosting (XGBoost), for instance, has been effectively applied to predict the strength capacity of ECC-reinforced beams [30] and FRCM-strengthened RC beams [31], yielding highly accurate load capacity estimations. Support Vector Regression (SVR) has likewise received significant attention in beam capacity prediction studies. Ahmad et al. [32] found that SVR can accurately predict splice strength of unconfined beams among the examined models. Similar conclusions were reported by Yuvaraj et al. [33] for fracture behavior of ultra-high-strength concrete beams and by Pal et al. [34] for strength estimation of deep beams. More recently, ensemble learning methods have demonstrated promising performance in structural engineering applications. Ge et al. [19] employed the Random Forest (RF) algorithm to estimate the flexural capacity of hybrid reinforced ECC–concrete composite beams, obtaining satisfactory predictive results. Taffese et al. [35] compared Categorical Boosting (CatBoost) with several ensemble models for predicting the flexural capacity of reinforced ultra-high-performance concrete (UHPC) beams and reported that CatBoost outperformed the other approaches across multiple evaluation metrics. Consistent findings were presented by Ye et al. [36], who confirmed the superior predictive accuracy of CatBoost in estimating the shear capacity of beams constructing using UHPC.

Despite the proven structural advantages of HSS–ECC composite beams, ML techniques' application for predicting their flexural capacity remains largely unexplored. Existing ML-based studies on composite beams are limited and focus on different performance aspects. For example, Oliveira et al. [37] predicted strength of steel–concrete composite beams subjected to lateral distortional buckling by employing ANN, Support Vector Machines (SVM), XGBoost, and RF models. The authors stated that all employed models appeared high accuracy ($R^2 > 0.9$) with XGBoost showing the best performance. Zhang et al. [38] applied ML models to field measurement data to predict temperature distributions in steel–concrete composite beams during hot asphalt paving. However, no prior research has investigated ML-driven prediction of flexural capacity for HSS–ECC composite beams. To address this gap, the present study evaluates five ML models, including XGBoost, ANN, SVR,

CatBoost, and RF, for predicting the flexural capacity of HSS-ECC composite beams. Based on the data of 135 samples with eight input features selected, the above-mentioned ML models are assessed and evaluated. The findings of this study could provide a systematic comparison of selected ML models for predicting the flexural capacity of composite beams employing HSS and ECC, as well as evaluate their robustness for structural engineering applications.

2. Proposed methodology

2.1. Dataset

Due to the limited experimental data on moment capacity of HSS-ECC composite beams, this study collected data obtained from numerical analysis and a parametric study of 132 HSS-ECC composite beam models that have been reported in Refs. [19-20], in accordance with data of 3 HSS-ECC composite beams that have been experimentally conducted and presented in Ref. [18]. Those numerical results were obtained from a numerical model that was successfully verified against experimental data and deemed a reliable tool for simulating the structural performance and flexural capacity of composite beams constructing with HSS girder and ECC slab [18]. Although the numerical model was validated against experimental data [18], it should be emphasized that the present database is dominated by numerically generated samples (132 out of 135). Accordingly, the trained ML models mainly learn the response surface of the validated numerical model, and the reported high accuracy should be interpreted primarily as interpolation capability within the numerical parameter space rather than comprehensive experimental verification. Important real-world uncertainties (e.g., material variability, fabrication tolerances, residual stresses, boundary-condition imperfections, and environmental effects) are not fully represented in numerically generated data. Therefore, expanding the database with independent experiments/field data is recommended in future work. There have been eight input parameters that can influence the flexural capacity of HSS-ECC composite beams, including the beam's geometric dimensions (i.e., ECC slab width (b_{eff}), ECC slab thickness (t_c), HSS section height (h_a), HSS web thickness (t_w), HSS flange thickness (t_f)), material properties of components employed in the composite beams (i.e., ECC compressive strength (f'_c), HSS yielding strength (f_y)), and spacing of shear studs (s_d) which represents the shear connection degree in the beam. The explanation of the beam's geometric dimensions is shown in Fig. 1 and Table 1. It is worth noting that the headed shear stud used in studies of the authors [18-20] had a 19 mm shank diameter and a 100 mm height. A slab with profile steel sheeting with a trough height of 55 mm was employed in these studies [18-20]. The distributed values information of the input features is shown in Table 2 and Fig. 2. In Table 2, the minimum, maximum, and average values of the input parameters are presented as V_{min} , V_{max} , and Ave., respectively. SD and COV stand for the standard deviation and coefficient of variation of the input values, respectively. The dataset of input values is tabulated in detail in the Appendix.

From the Kernel curves shown in Fig. 2, it is seen that the input values for ECC compressive strength f'_c vary between 40 and 70 MPa, and HSS yielding strength f_y is distributed between 690 and 960 MPa. The distribution of ECC slab width (b_{eff}) is between 600 and 1400 mm, while ECC slab thickness (t_c) values vary between 140 and 200 mm. HSS section height (h_a) ranges from 180 to 280 mm, centered around 230 mm. HSS web thickness (t_w) is distributed between 6 and 10 mm, while HSS flange thickness (t_f) varies from 8 to 12 mm, and shear studs spacing is distributed from 75 mm to 200 mm. The output values of moment capacity (M_u) were obtained by analysing a validated numerical model of HSS-ECC composite beams subjected to four-point bending configuration. The values of M_u range from 334.65 to 1070.08 kNm. Consequently, the applicability of the developed ML models is expected to be the highest for beams consistent with the data-generation setup (i.e., four-point bending) and for input variables within the ranges reported in Table 2. Data used outside these ranges and configurations constitutes extrapolation and may lead to increased prediction uncertainty.

Fig. 3 presents the Spearman correlation matrix of the input variables, which depicts the level of association among the selected features. The correlation coefficient varies between -1 and $+1$, where its absolute value reflects the strength of the relationship, and its sign indicates whether the association is positive or negative. A positive coefficient reflects a direct relationship between two variables, whereas a value close to zero implies little to no linear correlation. Conversely, a negative correlation coefficient signifies an opposite trend between variables, implying that higher values of one parameter are linked to lower values of the dependent variable. As observed in Fig. 3, all input features exhibit weak correlations, with correlation coefficients of relatively small magnitude. This suggests that no strong linear dependency exists among the input variables, indicating a low risk of multicollinearity and implying that each feature contributes independent information to the model.

Table 1
Explanation of input parameters

Component	Parameter	Explanation	Unit
ECC slab	f'_c	Compressive strength of ECC	MPa
	b_{eff}	ECC slab width	mm
	t_c	ECC slab thickness	mm
HSS I-section	f_y	Yield strength of HSS	MPa
	h_a	HSS I-section height	mm
	t_f	HSS I-section flange thickness	mm
	t_w	HSS I-section web thickness	mm
Shear studs	s_d	Shear studs spacing	mm

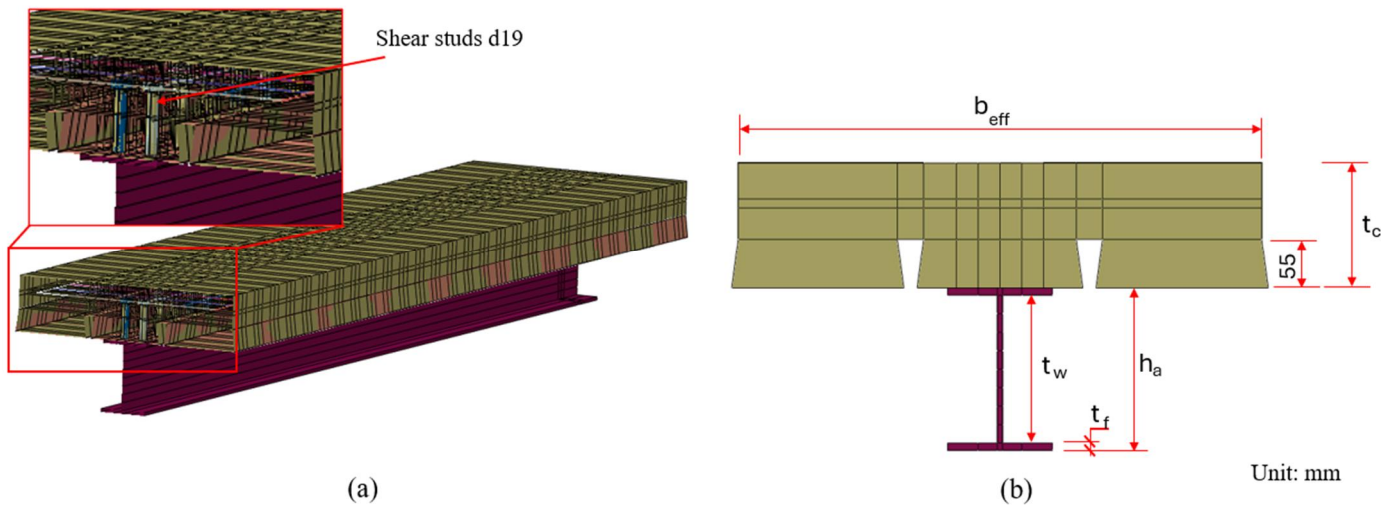


Fig. 1 Composite beam geometry: (a) Beam configuration; (b) Beam cross-section

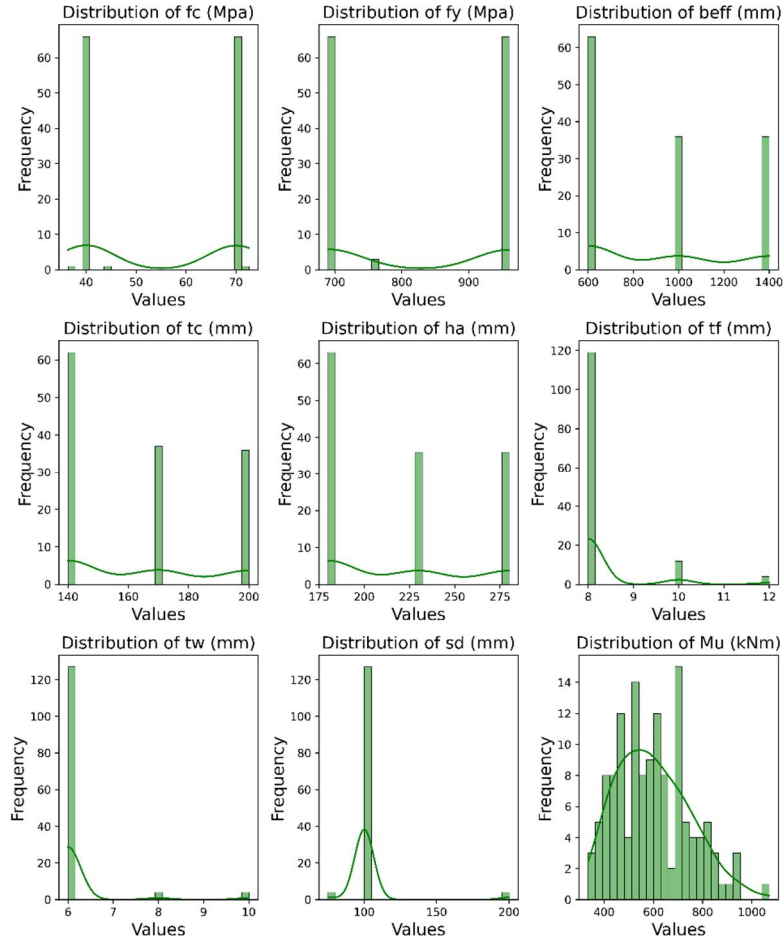


Fig. 2 Statistical distribution of input parameters

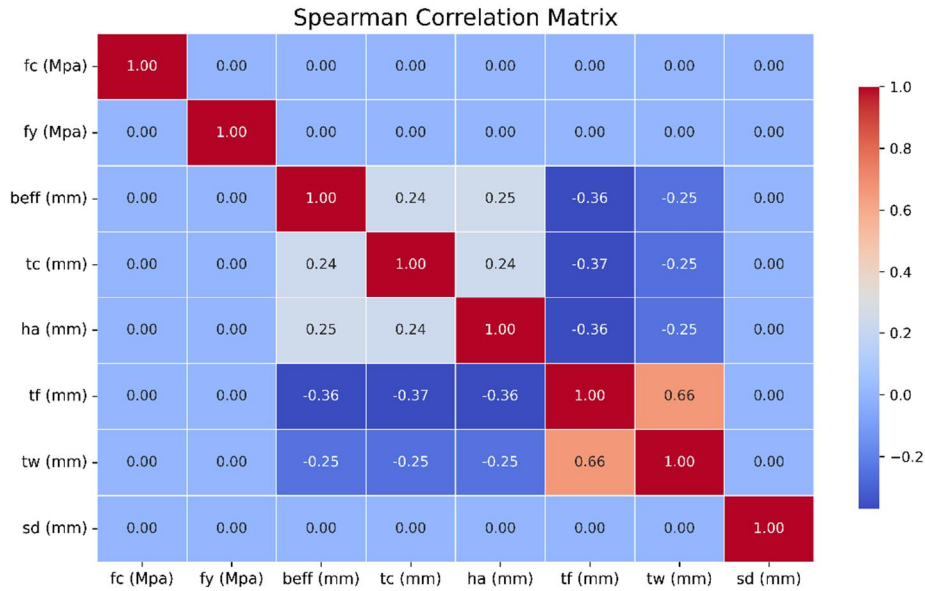


Fig. 3. Input feature Spearman correlation matrix

2.2. Preparation of the dataset

Preparing the dataset is a vital step in machine learning algorithms. It requires transforming all input and output parameters into a uniform dataset format. Converting raw data into a standardized range, such as from zero to 1, is referred to as normalization/standardization. By removing units through normalization/standardization, machine learning models can interpret the data more effectively. Normalization/standardization not only enhances data interpretability but also improves the model's accuracy, generalization, and learning performance. In this study, the dataset was normalized within the range of 0 to 1 using the following equation:

$$V' = \frac{V - V_{\min}}{V_{\max} - V_{\min}} \quad (1)$$

where V is the original value, V' is the normalised value (scaled between 0 and 1), $V_{\min}$ and $V_{\max}$ are the minimum and maximum values of the dataset. After standardizing, the data was separated into training and testing phases, with 80% utilized for training and the rest for testing the model.

Table 2
Statistical information of the input parameters

Parameter	Data type	V _{min}	V _{max}	Ave.	SD	CoV
f _c (MPa)	Input	70	40	55.0	15.06	0.27
f _y (MPa)	Input	690	960	825.0	135.51	0.16
b _{eff} (mm)	Input	600	1400	927.3	334.55	0.36
t _c (mm)	Input	140	200	164.6	25.09	0.15
h _a (mm)	Input	180	280	220.9	41.82	0.19
t _r (mm)	Input	8	12	8.3	0.87	0.11
t _w (mm)	Input	6	10	6.2	0.76	0.12
s _d (mm)	Input	75	200	102.2	17.66	0.17
M _u (kNm)	Output	334.7	1070.1	596.7	149.3	0.25

2.3. Evaluation metrics

The performance of the developed ML models was assessed using several statistical metrics, namely the Coefficient of Determination (R²), Root Mean Square Error (RMSE), Mean Absolute Error (MAE), and Mean Absolute Percentage Error (MAPE). These indicators collectively evaluate the accuracy and reliability of the models in predicting the flexural strength of HSS–ECC composite beams. Lower RMSE, MAE, and MAPE values indicate better predictive precision, while an R² value approaching 1 signifies strong consistency between predicted and actual results. While MAE, RMSE, MAPE, and R² are standard measures of predictive accuracy, their significance in structural engineering depends on how prediction errors affect safety, serviceability, and reliability. MAE reflects the average deviation between predicted and actual structural capacity; a low MAE reduces the risk of systematic bias, where underestimation leads to conservative and costly designs and overestimation may compromise safety. RMSE, by penalizing larger errors, is particularly relevant for structural safety, as occasional large deviations can control failure in critical members; a low RMSE indicates that such extreme errors are unlikely. MAPE expresses errors in relative terms, enabling practical interpretation; values on the order of 5–10% are generally acceptable for preliminary design and parametric studies, as they are typically smaller than the safety margins inherent in design codes, whereas higher values limit direct design applicability. R² indicates how well the model captures overall trends and parameter influences, which is valuable for optimization and sensitivity analysis, but it cannot alone ensure safe predictions and must be considered alongside absolute error metrics. Overall, these metrics suggest that the proposed models are best suited as decision-support or rapid prediction tools rather than direct substitutes for code-based design equations; with conservative bias or additional safety factors. They can aid preliminary design and comparative studies, while final design applications would require further experimental calibration and reliability assessment.

These metrics are expressed as in Eqs. 2–5, where y , $\hat{y}$, $\bar{y}$ denote the actual, predicted, and mean values of the dataset, respectively, whereas n represents the number of observations.

$$R^2 = 1 - \frac{\sum_{i=1}^n (\hat{y}_i - y_i)^2}{\sum_{i=1}^n (y_i - \bar{y})^2} \quad (2)$$

$$RMSE = \sqrt{\frac{1}{n} \sum_{i=1}^n (y_i - \hat{y}_i)^2} \quad (3)$$

$$MAE = \frac{1}{n} \sum_{i=1}^n |\hat{y}_i - y_i| \quad (4)$$

$$MAPE = \frac{1}{n} \sum_{i=1}^n \left| \frac{y_i - \hat{y}_i}{y_i} \right| \quad (5)$$

3. Prediction approaches

This section describes the development of interpretable machine learning models, including SVR, XGBoost, ANN, RF, and CatBoost, and evaluates their predictive performance. The general framework of the proposed models is depicted in Fig. 4. The model development procedure comprises database

construction, data preprocessing, model training, and optimization of hyperparameters.

3.1. ML prediction models

The structure of ML models is illustrated in Fig. 5, and the details are presented in the following sections.

3.1.1. Artificial Neural Network (ANN)

An Artificial Neural Network (ANN), also referred to as a Multilayer Perceptron (MLP), is a data-driven model inspired by the structure of the human brain. It comprises interconnected neurons arranged in an input layer, one or more hidden layers, and an output layer. Each connection carries a weight that is iteratively adjusted during training to reduce prediction error. The input layer represents the selected variables, the hidden layers capture complex nonlinear relationships, and the output layer (typically a single neuron) provides the predicted flexural capacity. In this study, the ANN architecture was optimized through hyperparameter tuning, including the number of hidden layers, neurons per layer, learning rate, and activation functions. The model was trained using backpropagation with Mean Squared Error (MSE) as the loss function, while early stopping and dropout were applied to improve generalization and mitigate overfitting.

3.1.2. Support Regression Vector (SVR)

Support Vector Regression (SVR), an extension of Support Vector Machines (SVM), is specifically formulated to model and predict continuous response variables. Rather than directly minimizing the prediction error, SVR aims to determine a regression function that lies within a specified tolerance margin (ϵ) around the training data. Errors within this margin are disregarded, while deviations exceeding it are penalized. The regression function can be expressed as in Eq. 6:

$$f(x) = w^T x + b \quad (6)$$

where w and b are parameters learned from the data. The goal is to keep the function as simple as possible while allowing deviations up to ϵ . A regularization parameter C controls how much the model tolerates errors outside this margin. Through the application of Kernel functions, including linear, polynomial, and radial basis function (RBF) kernels, SVR is capable of effectively capturing both linear and nonlinear relationships within the data. This makes it a flexible and reliable method for handling complex regression problems.

3.1.3. Extreme gradient boosting (XGBoost)

Extreme Gradient Boosting (XGBoost) is a powerful machine learning technique developed within the gradient boosting paradigm and is applicable to both regression and classification problems. The algorithm constructs an ensemble of weak predictive models (e.g., decision trees) through a sequential learning process. In this approach, each newly generated tree aims to minimize the residual errors produced by the preceding trees. The final prediction of the model is obtained by aggregating the outputs of all individual trees:

$$\hat{y}_i = \sum_{k=1}^K f_k(x_i) \quad f_k \in F \quad (7)$$

where $\hat{y}_i$ denotes the predicted output for the i -th observation, K represents the total number of decision trees in the ensemble, and F defines the functional space of regression trees. The objective function in XGBoost integrates two main components: a loss term L , which quantifies the discrepancy between predicted and actual values, and a regularization term that penalizes model complexity to enhance generalization and reduce the risk of overfitting:

$$Obj = \sum_{i=1}^n L(y_i, \hat{y}_i) + \sum_{k=1}^K \Omega(f_k) \quad (8)$$

$$\Omega(f) = \gamma T + \frac{1}{2} \lambda \|w\|^2 \quad (9)$$

where T is the number of leaves in a tree, w is the leaf weights, γ and λ are the regularization parameters. By combining gradient descent with tree boosting and adding strong regularization, XGBoost achieves high accuracy, robustness,

and efficiency, making it one of the most widely used algorithms in modern machine learning.

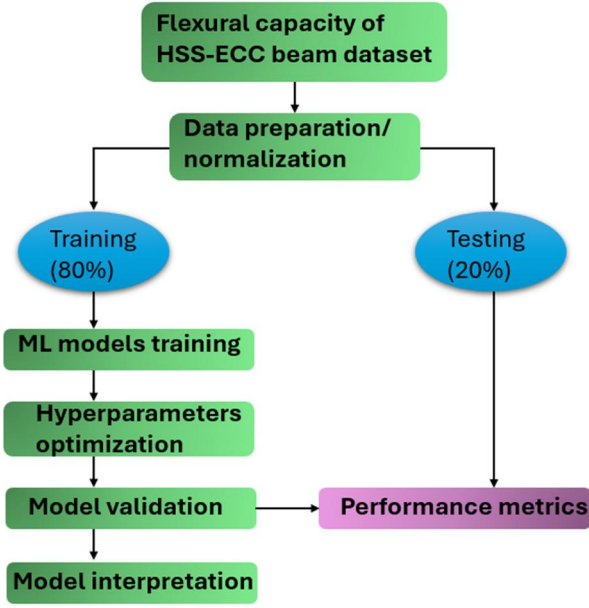


Fig. 4 Model development schemes

3.1.4. CatBoost

CatBoost is a gradient boosting-based machine learning algorithm introduced by Yandex, specifically designed to process categorical variables effectively without requiring complex preprocessing procedures. Similar to other boosting techniques, CatBoost constructs a sequence of decision trees in a stepwise fashion, where each successive tree is trained to reduce the residual errors generated by the preceding ensemble. The model output at iteration t can be formulated as:

$$F_t(x) = F_{t-1}(x) + \eta \cdot h_t(x) \quad (10)$$

where $F_{t-1}(x)$ is the model from the previous iteration, $f_t(x)$ is the newly added weak learner (decision tree), and η is the learning rate. One of the primary strengths of CatBoost is its implementation of ordered boosting, which helps mitigate prediction shift during training. In addition, the algorithm employs an efficient target-based encoding scheme to process categorical features. Together, these mechanisms improve model robustness and generalization capability, making CatBoost especially effective for datasets with numerous categorical variables.

3.1.5. Random Forest (RF)

The Random Forest (RF) model is an ensemble learning method that constructs numerous decision trees and combines their outputs to improve accuracy and reduce overfitting. Each tree is trained on a bootstrap sample of the dataset, and at each split, a random subset of features is selected to increase model diversity. For regression problems, predictions are obtained by averaging the outputs of all trees, while for classification, the final decision is made by majority voting. The RF prediction can be expressed mathematically as:

$$\hat{y} = \frac{1}{T} \sum_{t=1}^T h_t(x) \quad (11)$$

where T is the trees' total number, $h_t(x)$ is the t -th decision tree prediction, and $\hat{y}$ is the final prediction. The randomness in feature selection and data sampling gives the model robustness and strong generalization ability.

3.2. Optimization of the models' hyperparameters

To enhance model accuracy, hyperparameters were optimized using a grid search approach with 6-fold cross-validation. Each parameter combination in the predefined grid was evaluated, and the set yielding the highest average validation score was selected as optimal. The process was implemented using the GridSearchCV function in the scikit-learn library. The key tuned hyperparameters for the five models are listed in Table 3, while all other settings remained at their default values.

Table 3
Hyperparameters used for ML models

Model	Parameters	Values
ANN	Layer 1 (Dense)	Units: 64, Activation: 'relu', Input: (X_train.shape[1],)
	Layer 2 (Dense)	Units: 64, Activation: 'relu'
	Layer 3 (Dense)	Units: 32, Activation: 'relu'
	Layer 4 (Output)	Units: 1
CatBoost Regressor	iterations	1500
	learning_rate	0.05
	depth	6
	l2_leaf_reg	5
	loss_function	RMSE'
	eval_metric	MAE'
	task_type	GPU'
	early_stop-ping_rounds	50
	verbose	100
RandomForest Regressor	random_seed	42
	n_estimators	1500
	criterion	squared_error'
	random_state	42
	bootstrap	TRUE
	max_leaf_nodes	20
	max_depth	10
	n_jobs	-1
SVR	kernel	rbf'
	C	1000
	epsilon	1
	gamma	0.2
XGB Regressor	base_score	0.5
	booster	gbtree'
	n_estimators	1100
	objective	reg:squarederror'
	max_depth	3
	learning_rate	0.03
	tree_method	gpu_hist'
	predictor	gpu_predictor'

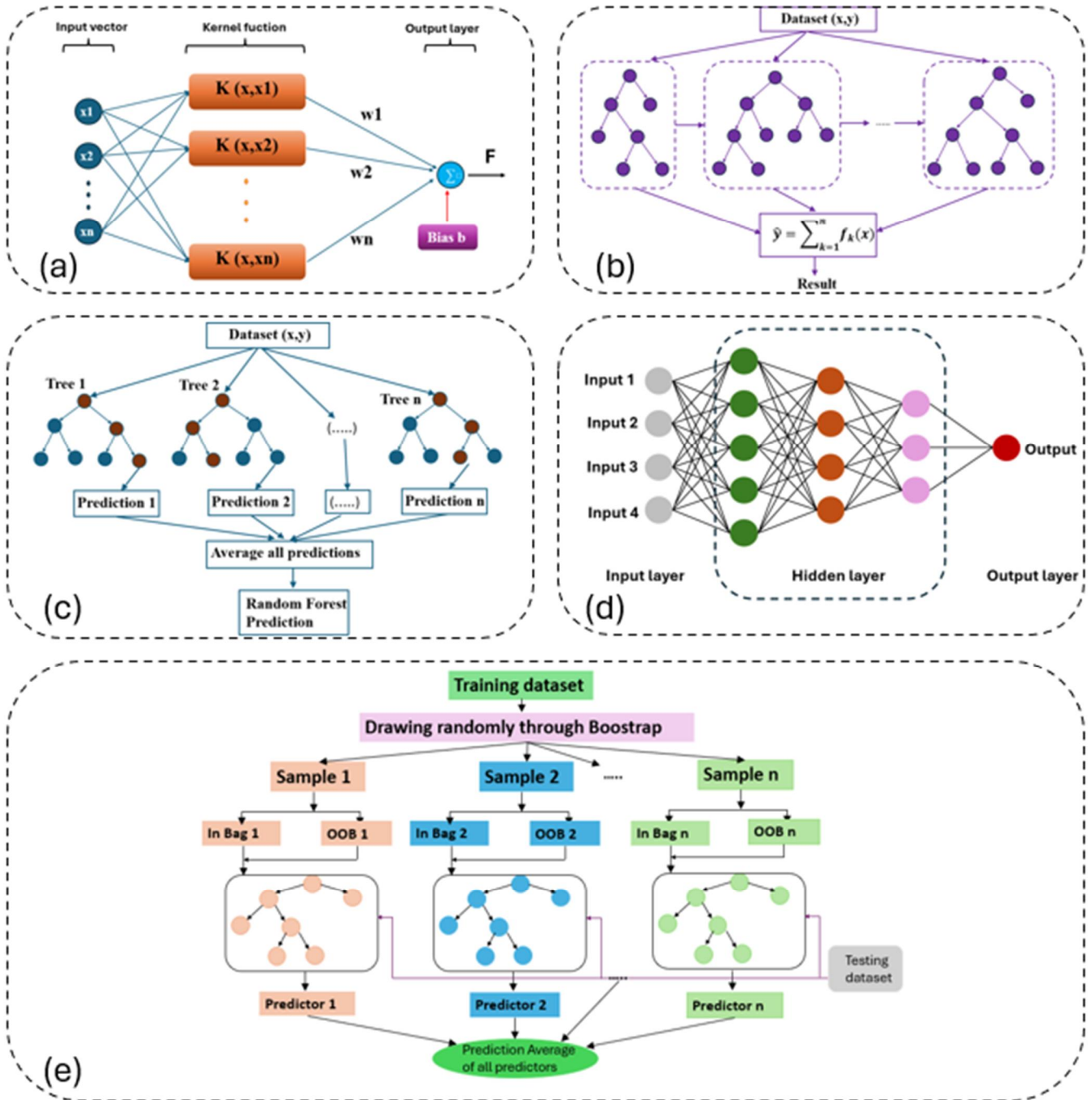


Fig. 5 Structures of ML models: a) ANN; b) SVR; c)XGBoost; d) CatBoost; e) RF

4. Analysis of predicted model performance

After determining the optimal hyperparameters for each algorithm, five machine learning models, including SVR, XGBoost, ANN, RF, and CatBoost, were developed using the training dataset. The predictive capability of these models was evaluated by comparing their outputs with the numerically obtained moment capacities of HSS–ECC composite beams. As depicted in Fig. 6, all five algorithms were able to reasonably estimate the flexural capacity of composite beams composed of HSS I-sections and ECC slabs. However, differences in predictive consistency were observed. The SVR and ANN models exhibited closer agreement with the reference values, while the RF model showed comparatively greater dispersion in its predictions. To further examine model performance, Fig. 7 illustrates scatter plots with linear regression fits between predicted and numerical results. In these plots, actual values are presented on the x-axis and predicted values on the y-axis. The solid diagonal line ($y = x$) represents perfect correspondence, whereas dashed lines corresponding to $\pm 10\%$ error bounds ($y = 0.9x$ and $y = 1.1x$) define acceptable prediction limits. A tighter clustering of points around the diagonal line indicates higher predictive accuracy and stability. Although all models captured the overall trend of the numerical data, the SVR model demonstrated the

strongest alignment with the reference results. Predictions from SVR, ANN, CatBoost, and XGBoost largely fell within the acceptable error margins, whereas the RF model showed several data points exceeding the defined tolerance range, suggesting comparatively lower robustness. For quantitative assessment, statistical performance indicators, including RMSE, MAE, MAPE, and R^2 , were computed for both training and testing datasets, as summarized in Table 4. In addition, radar and bar charts presented in Fig. 8 provide a visual comparison of these metrics across the five algorithms. These comparative analyses consistently indicate that SVR delivers the highest predictive accuracy and reliability in estimating the moment capacity of HSS–ECC composite beams.

A more detailed examination of Table 4 and Fig. 8 indicates that all developed machine learning models exhibit strong learning and predictive capabilities, with only minor differences attributable to the intrinsic mechanisms and limitations of each algorithm. High R^2 values were achieved for both training and testing datasets across all models, although the RF model recorded comparatively lower values. In terms of training performance, SVR and ANN demonstrated superior fitting ability, achieving the smallest error metrics (RMSE, MAE, and MAPE) on the training set. Meanwhile, XGBoost and CatBoost delivered closely comparable results across the evaluation criteria.

More notably, when assessing generalization performance on the testing dataset, SVR outperformed the remaining models. It produced the lowest prediction errors (MAE = 8.25, MAPE = 1.58, RMSE = 16.67) along with the highest coefficient of determination ($R^2 = 0.99$), indicating excellent agreement with the reference results. Overall, although all five models proved capable of estimating the moment capacity of HSS–ECC composite beams, SVR provided the most consistent balance between fitting accuracy and generalization performance. In contrast, the RF model showed comparatively weaker predictive capability, as

reflected by its higher RMSE, MAE, and MAPE values and the lowest R^2 among the evaluated algorithms. The ANN model exhibited strong accuracy during training; however, a slight reduction in performance was observed on the testing dataset, suggesting minor generalization limitations. To further assess model robustness, Taylor diagrams were employed. These diagrams offer a comprehensive graphical comparison by simultaneously incorporating the correlation coefficient, centered root-mean-square error, and standard deviation into a unified geometric representation [39].

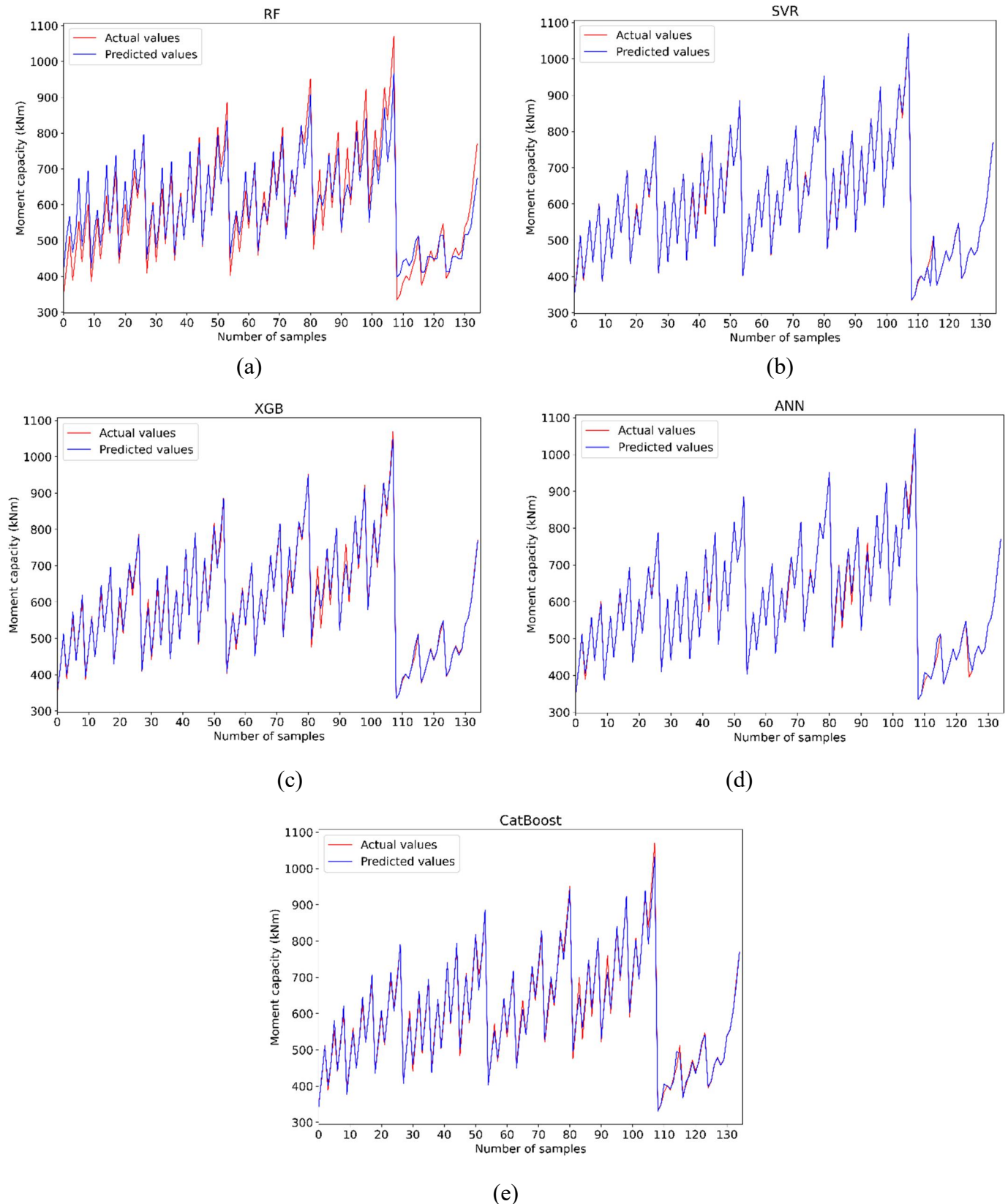


Fig. 6 Prediction values of ML models versus actual data: (a) RF; (b) SVR; (c) XGB; (d) ANN; (e) CatBoost

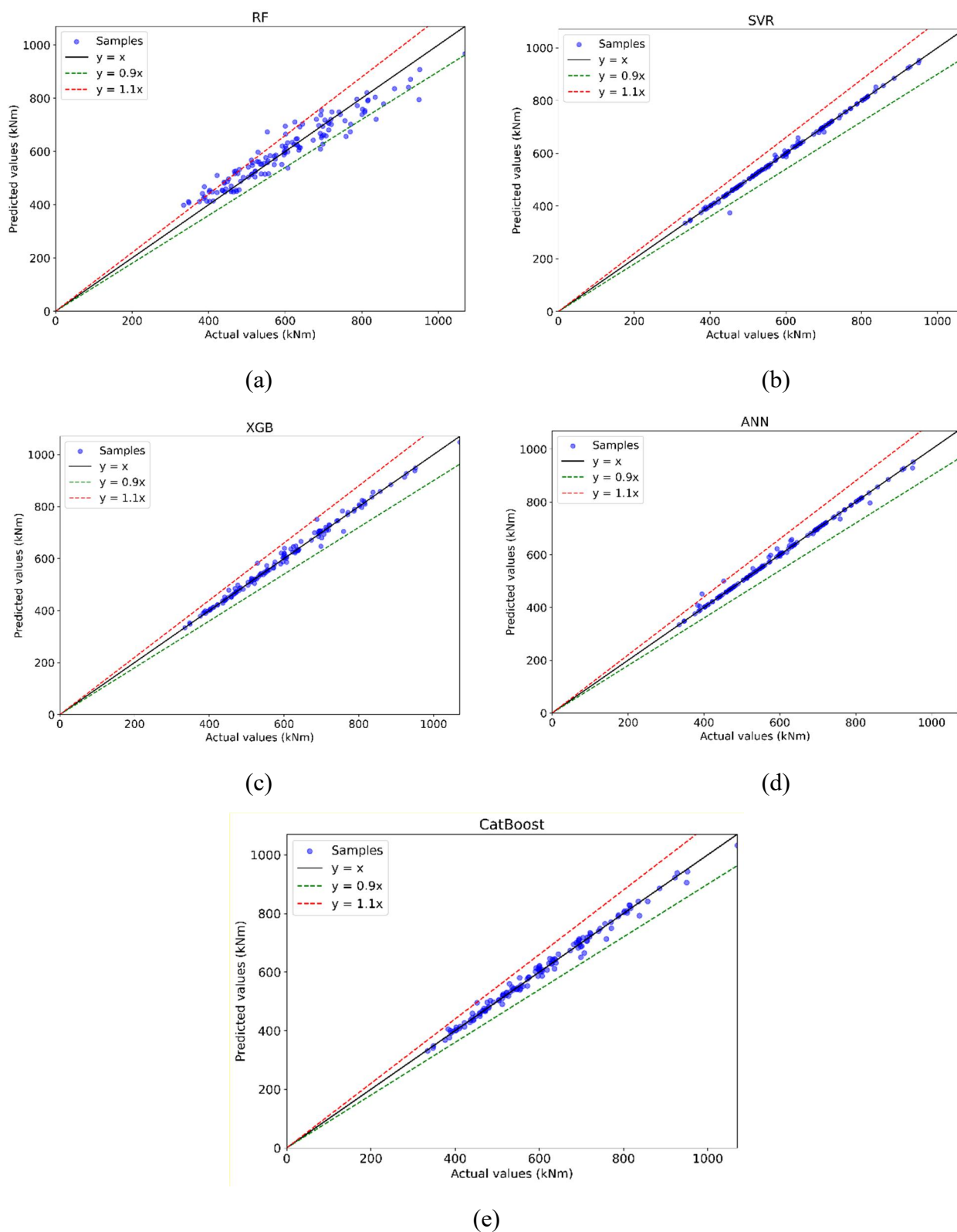
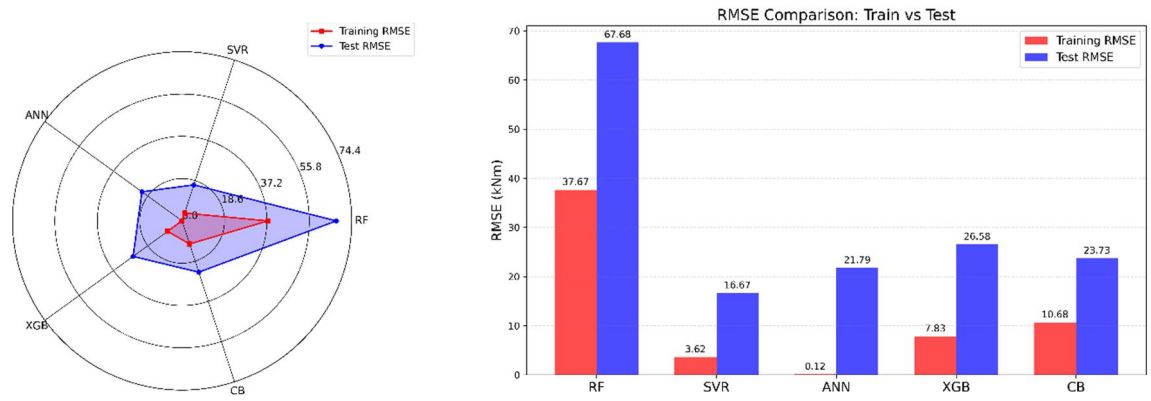


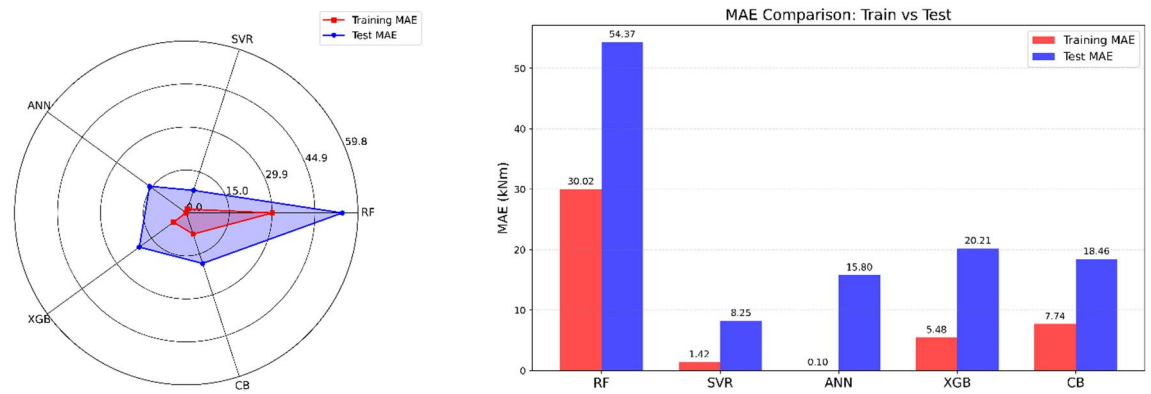
Fig. 7 Linear fitting plots of moment capacity of composite beams obtained by ML model (predicted value) and actual values: (a) RF; (b) SVR; (c) XGB; (d) ANN; (e) CatBoost

This approach allows performance comparisons by measuring how closely each model's results align with the actual data. Widely applied in assessing the behavior of composite materials, Taylor diagrams are particularly useful for analyzing predictive reliability. As depicted in Fig. 9, the purple dashed arcs represent centered RMSE, the radial axes indicate normalized standard

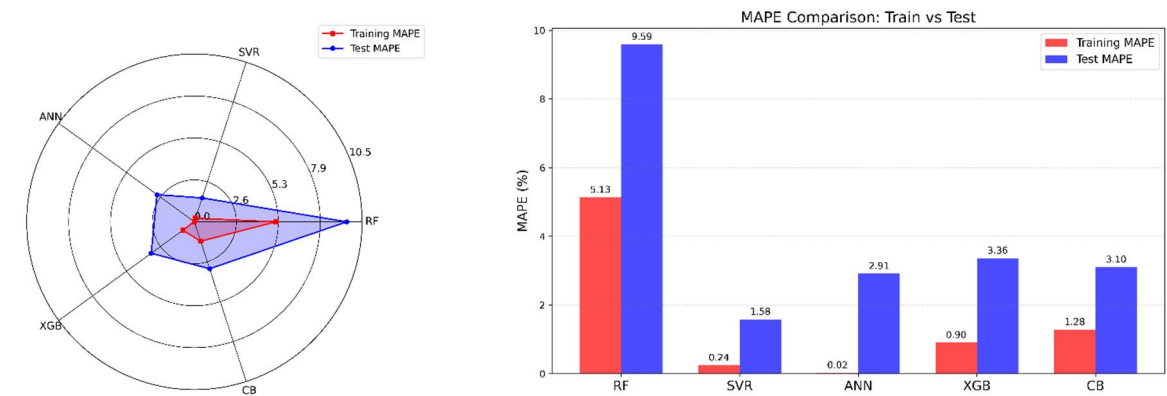
deviation, and concentric curves display correlation coefficients. Both training and testing diagrams confirm that SVR attains higher correlation, reduced deviation, and smaller centered RMSE compared with other models, indicating that SVR provides the most accurate and stable learning as well as predictive outcomes.



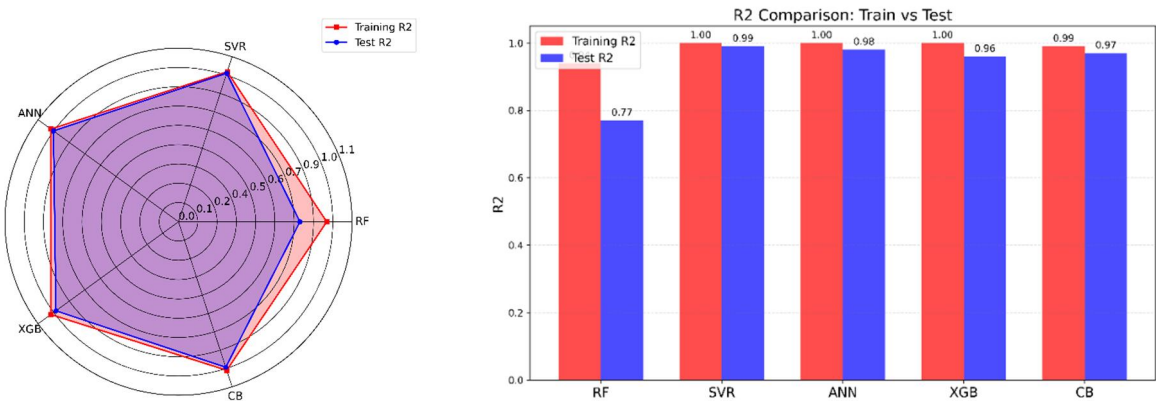
(a) RMSE



(b) MAE



(c) MAPE



(d) R^2

Fig. 8 Evaluation metrics of ML models

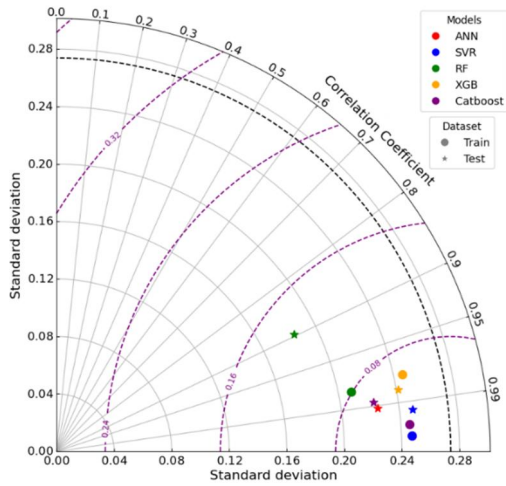


Fig. 9 Taylor diagram of ML models

Table 4
Evaluation metrics of ML models

Eva. Metr.	Dataset	RF	SVR	ANN	XGBoost	CatBoost
RMSE (kNm)	Train	33.67	3.62	0.12	7.83	10.68
	Test	67.68	16.67	21.79	26.58	23.73
MAE (kNm)	Train	30.02	1.42	0.10	5.48	7.74
	Test	54.37	8.25	15.08	20.21	18.46
MAPE (%)	Train	5.13	0.24	0.02	0.90	1.28
	Test	9.59	1.58	2.91	3.36	3.10
R ²	Train	0.94	1	1	1	0.99
	Test	0.77	0.99	0.98	0.96	0.97

5. Evaluation of input feature significance and correlation to the model output

The growing use of machine learning techniques in scientific research has highlighted the importance of interpretability approaches that can clarify the reasoning behind model predictions. Among the most commonly used

interpretability frameworks is Shapley Additive Explanations (SHAP), which offers a robust, theoretically grounded approach to assessing the contribution of individual input variables. In contrast to conventional sensitivity analysis methods, SHAP evaluates not only the isolated effect of each feature but also its combined and interaction effects, thereby mitigating issues associated with multicollinearity. This approach determines the relative significance of each input variable by calculating its mean marginal effect over all possible feature subsets. The strength of each variable's influence is measured using the absolute SHAP values, where higher magnitudes correspond to a more significant impact on the model's prediction [40–42]. In this study, SHAP analysis was carried out on the SVR model since it demonstrated the most accurate model in the preceding evaluations. Fig. 10(a) presents a summary plot in which each data point represents an individual sample, and its horizontal position corresponds to the SHAP value of the associated feature. Positive SHAP values signify that a feature contributes to increasing the predicted moment capacity, whereas negative values indicate a contribution that reduces the predicted value. Feature values are represented using a color scale, where red denotes higher values, blue indicates lower values, and intermediate colors correspond to values near the mean. The spread and concentration of points along the horizontal axis reflect the relative influence of each variable. Fig. 10(b) presents an overall evaluation of feature significance using a bar chart derived from the mean absolute SHAP values computed over the entire dataset. The horizontal axis represents the average magnitude of the SHAP values, whereas the vertical axis displays the input variables ordered from the highest to the lowest level of influence. As can be seen from the results in Fig. 10, it is revealed that HSS section height (h_a), ECC slab thickness (t_c), ECC slab width (b_{eff}), ECC compressive strength (f'_c), and HSS yielding strength (f_y) play dominant roles in predicting moment capacity of the HSS-ECC composite beams, in which h_a is the most influential feature. In contrast, the HSS web thickness (t_w), HSS flange thickness (t_f), and shear studs spacing (s_d) marginally contribute to the model's predictions. From an engineering standpoint, the SHAP ranking is consistent with composite-beam mechanics. Increasing the HSS section height (h_a) primarily increases the internal lever arm between tensile and compressive resultants, leading to a pronounced increase in moment capacity. The ECC slab thickness (t_c) and effective width (b_{eff}) directly affect the compressive force that can be developed in the slab and its contribution to the composite section capacity. Material strengths (ECC compressive strength f'_c and HSS yield strength f_y) further scale these compressive and tensile resultants, respectively, and their effects become more pronounced when combined with larger lever arms and slab dimensions. The comparatively smaller SHAP contribution of shear stud spacing (s_d) should be interpreted within the scope of the dataset and modelling assumptions. It may reflect the limited variation range considered and/or idealizations in the numerical modelling of composite action. Therefore, while SHAP provides a useful “priority map” for flexural capacity improvement within the studied domain, variables with lower SHAP importance may still govern other design checks (e.g., shear connection design, local stability, constructability) and should not be disregarded.

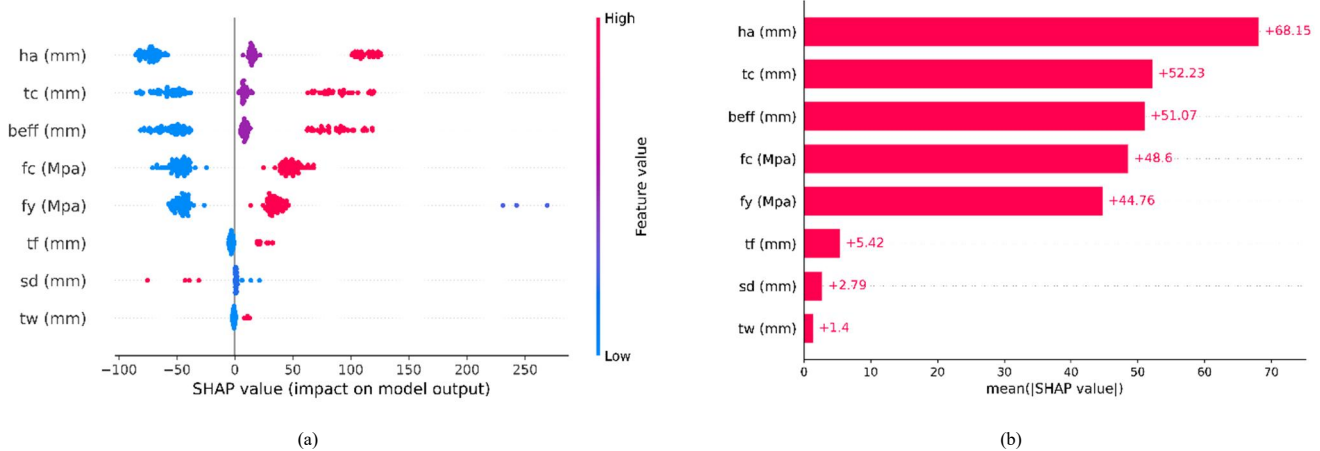


Fig. 10 SHAP plots of input features: (a) SHAP value; (b) mean SHAP value

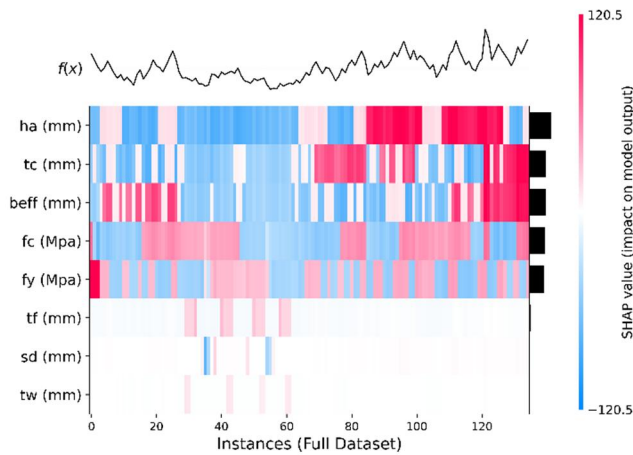


Fig. 11 SHAP heatmap of input features

In addition, SHAP values were further visualized using a clustered heatmap to enable a more integrated interpretation of feature contributions. In this representation, individual samples are arranged in the horizontal direction, while the corresponding predicted responses are shown in the vertical axis. The color intensity within the matrix denotes the SHAP value associated with each input variable, and hierarchical clustering is employed to group samples exhibiting similar contribution patterns. As illustrated in Fig. 11, the SHAP heatmap displays the predicted outputs above the matrix, a dashed gray line indicating the reference baseline, and a bar plot on the right summarizing the overall input features' importance. The variation of the prediction function $f(x)$ across samples reflects the cumulative influence of SHAP values on the model output. The results consistently demonstrate that the HSS section height (h_a) exerts the strongest influence on moment capacity predictions, whereas HSS web thickness (t_w) and HSS flange thickness (t_f) and shear studs spacing (s_d) contribute the least. Taken together, the SHAP summary plots and the heatmap generated from the SVR model provide a clear and interpretable explanation of the relative significance of input parameters, enhancing insight into their effects on the flexural behavior of composite beams employing HSS and ECC.

6. Conclusions

This study systematically investigates the flexural capacity of HSS–ECC composite beams using five machine learning algorithms, namely Extreme

Gradient Boosting (XGBoost), Support Vector Regression (SVR), Random Forest (RF), Artificial Neural Networks (ANN) and CatBoost. A database comprising 132 numerical samples data and 3 experimental data collected from the author's published studies was established. Eight influential input features were selected, preprocessed, and analyzed to develop robust predictive models. To improve interpretability and provide deeper insight into the prediction mechanism, the Shapley Additive Explanations (SHAP) method was employed to quantify the specific contribution of each input variable to the flexural capacity of HSS–ECC composite beams. The key findings derived from this analysis are summarized below:

1. The SVR and ANN algorithms demonstrate outstanding training performance, reflected by their minimal RMSE, MAE, and MAPE values. However, SVR further distinguishes itself on the testing dataset, achieving the lowest prediction errors (MAE, MAPE, and RMSE) together with the highest R^2 ($R^2 = 0.99$), indicating superior generalization ability. XGBoost and CatBoost yield similar and consistently stable results across all evaluation criteria, confirming their robustness. By comparison, the RF model delivers comparatively lower predictive accuracy, as evidenced by its higher error metrics and the smallest R^2 among the evaluated models.
2. Evaluation metrics comparisons reveal that the SVR model provides the most balanced performance in terms of accuracy and robustness. This observation is further supported by Taylor diagram analysis, which shows that the SVR model predictions are consistently closest to the actual results during both training and testing stages, confirming its superior learning capability.
3. SHAP-based interpretability analysis, including additive explanation plots and clustered heatmaps, identifies that the HSS section height (h_a), ECC slab thickness (t_c), effective ECC slab width (b_{eff}), ECC compressive strength (f'_c), and HSS yield strength (f_y) are the dominant parameters governing the predicted moment capacity of HSS–ECC composite beams, in which HSS section height (h_a) is the most influential feature. In contrast, the HSS web thickness (t_w), flange thickness (t_f), and shear studs spacing (s_d) exhibit a marginal influence on the model predictions.
4. The predictive reliability of the developed models is expected to be highest within the parameter ranges and loading configuration covered by the training data (Table 2; four-point bending setup). Application beyond these ranges constitutes extrapolation and may reduce accuracy. Therefore, the proposed ML models are recommended as decision-support tools for preliminary design and rapid assessment; further independent experimental datasets and reliability calibration are required before direct replacement of code-based design procedures.

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Appendix

No.	f_c (Mpa)	f_y (Mpa)	b_{eff} (mm)	t_c (mm)	h_a (mm)	t_r (mm)	t_w (mm)	s_d (mm)	M_u (kNm)
1	40	690	600	140	180	8	6	100	347.3
2	40	690	600	140	230	8	6	100	422.1
3	40	690	600	140	280	8	6	100	511.8
4	40	690	600	170	180	8	6	100	388.7
5	40	690	600	170	230	8	6	100	467.5
6	40	690	600	170	280	8	6	100	553.2
7	40	690	600	200	180	8	6	100	440.6
8	40	690	600	200	230	8	6	100	526.7
9	40	690	600	200	280	8	6	100	600.3
10	40	690	1000	140	180	8	6	100	386.4
11	40	690	1000	140	230	8	6	100	470.9
12	40	690	1000	140	280	8	6	100	560.6
13	40	690	1000	170	180	8	6	100	449.1
14	40	690	1000	170	230	8	6	100	530.7
15	40	690	1000	170	280	8	6	100	625.0
16	40	690	1000	200	180	8	6	100	521.0
17	40	690	1000	200	230	8	6	100	602.6
18	40	690	1000	200	280	8	6	100	692.9
19	40	690	1400	140	180	8	6	100	436.4
20	40	690	1400	140	230	8	6	100	514.1
21	40	690	1400	140	280	8	6	100	600.9
22	40	690	1400	170	180	8	6	100	514.1
23	40	690	1400	170	230	8	6	100	606.1
24	40	690	1400	170	280	8	6	100	694.6
25	40	690	1400	200	180	8	6	100	618.1
26	40	690	1400	200	230	8	6	100	698.1
27	40	690	1400	200	280	8	6	100	787.2
28	40	960	600	140	180	8	6	100	409.4
29	40	960	600	140	230	8	6	100	503.7
30	40	960	600	140	280	8	6	100	606.6
31	40	960	600	170	180	8	6	100	441.0
32	40	960	600	170	230	8	6	100	538.8
33	40	960	600	170	280	8	6	100	645.2
34	40	960	600	200	180	8	6	100	490.5
35	40	960	600	200	230	8	6	100	596.9
36	40	960	600	200	280	8	6	100	681.4
37	40	960	1000	140	180	8	6	100	444.5
38	40	960	1000	140	230	8	6	100	554.9

39	40	960	1000	140	280	8	6	100	632.5
40	40	960	1000	170	180	8	6	100	503.7
41	40	960	1000	170	230	8	6	100	596.9
42	40	960	1000	170	280	8	6	100	740.6
43	40	960	1000	200	180	8	6	100	571.6
44	40	960	1000	200	230	8	6	100	673.3
45	40	960	1000	200	280	8	6	100	788.3
46	40	960	1400	140	180	8	6	100	483.6
47	40	960	1400	140	230	8	6	100	599.2
48	40	960	1400	140	280	8	6	100	711.9
49	40	960	1400	170	180	8	6	100	573.9
50	40	960	1400	170	230	8	6	100	700.9
51	40	960	1400	170	280	8	6	100	816.5
52	40	960	1400	200	180	8	6	100	706.7
53	40	960	1400	200	230	8	6	100	756.1
54	40	960	1400	200	280	8	6	100	885.5
55	70	690	600	140	180	8	6	100	402.5
56	70	690	600	140	230	8	6	100	480.1
57	70	690	600	140	280	8	6	100	571.6
58	70	690	600	170	180	8	6	100	468.6
59	70	690	600	170	230	8	6	100	549.1
60	70	690	600	170	280	8	6	100	638.3
61	70	690	600	200	180	8	6	100	535.3
62	70	690	600	200	230	8	6	100	629.6
63	70	690	600	200	280	8	6	100	703.8
64	70	690	1000	140	180	8	6	100	459.0
65	70	690	1000	140	230	8	6	100	553.7
66	70	690	1000	140	280	8	6	100	636.0
67	70	690	1000	170	180	8	6	100	544.5
68	70	690	1000	170	230	8	6	100	628.5
69	70	690	1000	170	280	8	6	100	722.2
70	70	690	1000	200	180	8	6	100	636.5
71	70	690	1000	200	230	8	6	100	720.5
72	70	690	1000	200	280	8	6	100	815.4
73	70	690	1400	140	180	8	6	100	521.0
74	70	690	1400	140	230	8	6	100	599.7
75	70	690	1400	140	280	8	6	100	687.7
76	70	690	1400	170	180	8	6	100	622.7
77	70	690	1400	170	230	8	6	100	719.3
78	70	690	1400	170	280	8	6	100	813.6
79	70	690	1400	200	180	8	6	100	771.1
80	70	690	1400	200	230	8	6	100	857.3
81	70	690	1400	200	280	8	6	100	951.6
82	70	960	600	140	180	8	6	100	476.1
83	70	960	600	140	230	8	6	100	575.6
84	70	960	600	140	280	8	6	100	698.6
85	70	960	600	170	180	8	6	100	529.0
86	70	960	600	170	230	8	6	100	631.4
87	70	960	600	170	280	8	6	100	744.1
88	70	960	600	200	180	8	6	100	592.4
89	70	960	600	200	230	8	6	100	713.6
90	70	960	600	200	280	8	6	100	802.1
91	70	960	1000	140	180	8	6	100	522.3

92	70	960	1000	140	230	8	6	100	639.4
93	70	960	1000	140	280	8	6	100	759.0
94	70	960	1000	170	180	8	6	100	600.9
95	70	960	1000	170	230	8	6	100	714.2
96	70	960	1000	170	280	8	6	100	834.9
97	70	960	1000	200	180	8	6	100	691.7
98	70	960	1000	200	230	8	6	100	801
99	70	960	1000	200	280	8	6	100	922.3
100	70	960	1400	140	180	8	6	100	591.1
101	70	960	1400	140	230	8	6	100	694.6
102	70	960	1400	140	280	8	6	100	808.5
103	70	960	1400	170	180	8	6	100	695.8
104	70	960	1400	170	230	8	6	100	807.9
105	70	960	1400	170	280	8	6	100	927.5
106	70	960	1400	200	180	8	6	100	837.8
107	70	960	1400	200	230	8	6	100	949.9
108	70	960	1400	200	280	8	6	100	1070.1
109	40	690	600	140	180	8	6	200	334.7
110	40	690	600	140	180	8	6	75	348.5
111	70	690	600	140	180	8	6	200	383.5
112	70	690	600	140	180	8	6	75	401.4
113	40	960	600	140	180	8	6	200	389.9
114	40	960	600	140	180	8	6	75	422.1
115	70	960	600	140	180	8	6	200	452.0
116	70	960	600	140	180	8	6	75	511.8
117	40	690	600	140	180	10	6	100	376.1
118	40	690	600	140	180	12	6	100	402.5
119	70	690	600	140	180	10	6	100	435.3
120	70	690	600	140	180	12	6	100	471.5
121	40	960	600	140	180	10	6	100	442.2
122	40	960	600	140	180	12	6	100	464.6
123	70	960	600	140	180	10	6	100	512.9
124	70	960	600	140	180	12	6	100	547.4
125	40	690	600	140	180	10	8	100	395.0
126	40	690	600	140	180	10	10	100	412.3
127	70	690	600	140	180	10	8	100	458.3
128	70	690	600	140	180	10	10	100	479.6
129	40	960	600	140	180	10	8	100	458.9
130	40	960	600	140	180	10	10	100	472.7
131	70	960	600	140	180	10	8	100	537.1
132	70	960	600	140	180	10	10	100	556.6
133	36.34	759	600	140	180	8	6	100	607
134	44.5	759	600	170	180	8	6	100	692
135	72.6	759	600	140	180	8	6	100	770

PRELIMINARY STUDY ON LEANING COLUMN EFFECT CAUSING COLLAPSE OF TEMPORARY SUPPORTS DURING GROUTING IN TAIWAN

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ABSTRACT

In reinforced concrete structures, temporary supports frequently fail during slab grouting instead of after completion. This indicates that the initial design strength of the temporary support is greater than the actual weight of the newly grouted concrete. This study investigates the failure mechanism of temporary support collapses during concrete grouting, based on LeMessurier's theory of the leaning column effect and previous research by Peng et al. on the load-bearing capacity and the leaning column effect in single-story, double-story, and combined scaffold temporary supports. The findings indicate that the collapse mechanism in these temporary supports during concrete grouting is primarily driven by the leaning column effect, which significantly reduces the critical loads of the overall structural system. The leaning column will induce the leaning column effect in the compression structural system, thereby significantly reducing the critical load of the overall structural system. This study examines the three types of temporary supports and explains how the failure mechanism of the leaning column effect influences the reduction of critical loads on the overall structures. Moreover, this study collected actual collapse instances of the three previously mentioned types of temporary supports to illustrate how the failure mechanism associated with the leaning column effect results in the collapse of the temporary support during the grouting process. Proposals for improving assemblies of three temporary supports were also made. When designing temporary supports, it's essential to change the unrestrained boundary condition of the structural system to a restrained one to eliminate the leaning column effect.

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1. Introduction

The setup of temporary supports in construction varies based on the headroom of buildings. In Taiwan, three typical setups are used: a single-story temporary support, a double-story temporary support, and a combined assembly of tubular steel scaffolds with other supports (referred to as a combined scaffold temporary support). For the headroom below 4 m, single-story temporary supports, typically using tubular steel adjustable shores or wooden shores, are predominant, as shown in Fig. 1. Double-story temporary supports are often chosen for cost savings for the headroom between 4 and 7 m, primarily utilizing tubular steel adjustable shores, as illustrated in Fig. 2.



Fig. 1 A single-story temporary support on a construction site in Taiwan

When the headroom exceeds 7 m, temporary supports typically employ a combination of tubular steel scaffolds with wooden shores or tubular steel adjustable shores, which is referred to as a "combined scaffold temporary support" for simplicity. The steel scaffold, a standard material, has a unit height of 1.7 m. After stacking the scaffolds several stories vertically, if the gap between the top of the scaffold and the bottom of the slab is less than 1.7 m, contractors fill this space with wooden shores or tubular steel adjustable shores, creating a hybrid assembly, as shown in Fig. 3.



Fig. 2 A double-story temporary support on a construction site in Taiwan

Steel structure design codes consider live loads, dead loads, seismic forces, and wind loads. [1–4] During construction, the primary external load is the weight of grouted fresh concrete, with secondary loads including rebar, formwork, personnel, equipment vibrations, etc. In Taiwan, investigations into construction accidents show that temporary support collapses are not linked to earthquakes or typhoons. As a result, current design practices typically do not consider seismic or wind forces.

Since temporary supports are temporary structures that are removed immediately after construction, their importance is often overlooked. To date, the temporary support design in Taiwan typically follows steel structure design

codes, applying the ASD concept with increased safety factors. For smaller construction projects, temporary supports are often built based on the experience of on-site workers.

The setups for temporary support differ from those for traditional steel structures. Temporary supports commonly use pin connections, while steel structures rely on high-tension bolts and welding. Furthermore, because of variations in construction accuracy and the construction environment, it is unsuitable to directly apply steel structure design specifications for designing temporary supports.

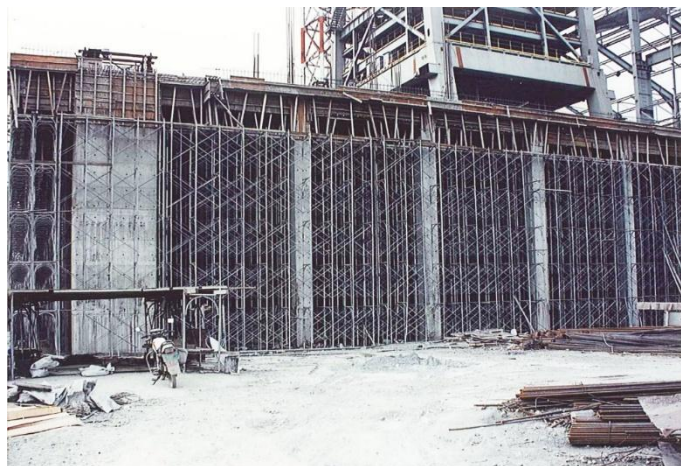


Fig. 3 A combined scaffold temporary support on a construction site in Taiwan

Collapses of temporary supports are not uncommon. For example, on January 28, 1997, a single-story temporary support collapsed during slab grouting at the box culvert on Second Expressway in Kaohsiung, Taiwan, resulting in one death and one severe injury (Fig. 4). On December 17, 2006, during grouting at the Linkou residential project in Taipei, Taiwan, a double-story temporary support collapsed, killing five workers beneath the fallen reinforced concrete slab (Fig. 5). On October 4, 1998, another incident took place at the Chiayi Hospital construction site. During grouting, a temporary support system that combined scaffolds and tubular steel adjustable shores collapsed, resulting in one fatality and six injuries (Fig. 6).

Reviewing these collapse cases and earlier studies [5] shows that temporary support failures primarily occur during concrete pouring. This indicates a connection between these failures and the design of the temporary supports. For example, if a professional engineer determines that the structural strength of temporary support is 100 tonnes, using a safety factor of 2 results in a design strength of 50 tonnes. This indicates that the temporary support must bear 50

tonnes of grouted fresh concrete.

However, if the temporary support collapses when only half of the slab is grouted, the weight of the grouted fresh concrete is approximately 25 tonnes. This indicates that the weight of the grouted fresh concrete is only one-fourth (i.e., 25/100) of the initially calculated strength of the temporary support. Further investigation is needed to understand why the initially calculated strength of the temporary support differs significantly from the actual weight of the grouted fresh concrete. We could start by examining how the leaning column effect influences the strength of the temporary support. [6]

A considerable amount of research has been conducted on steel scaffolds. For instance, Jake and Chan (2020) [7] employed the second-order analysis to investigate the mechanical behavior of scaffolding structures, confirming how uneven subsidence affects their ultimate load capacity. In a separate study, Jake and Chan (2019) [8] conducted a direct analysis to examine the ultimate load capacity of scaffolding structures featuring asymmetrical joints. Liu et al. (2010) [9] performed load tests on scaffolding structures reinforced with large X-braces, demonstrating the relationship between the X-brace and the load-bearing capacity of the scaffolds. Peng et al. (2021) [10] examined how inclined top and bottom boundary conditions influence the ultimate load of scaffolding structures.



Fig. 4 Collapse of single-story temporary support at the box culvert on Second Expressway in Kaohsiung, Taiwan

Tubular steel adjustable shore remains intact and undamaged



Fig. 5 Collapse of double-story temporary support at the Linkou residential project in Taipei, Taiwan

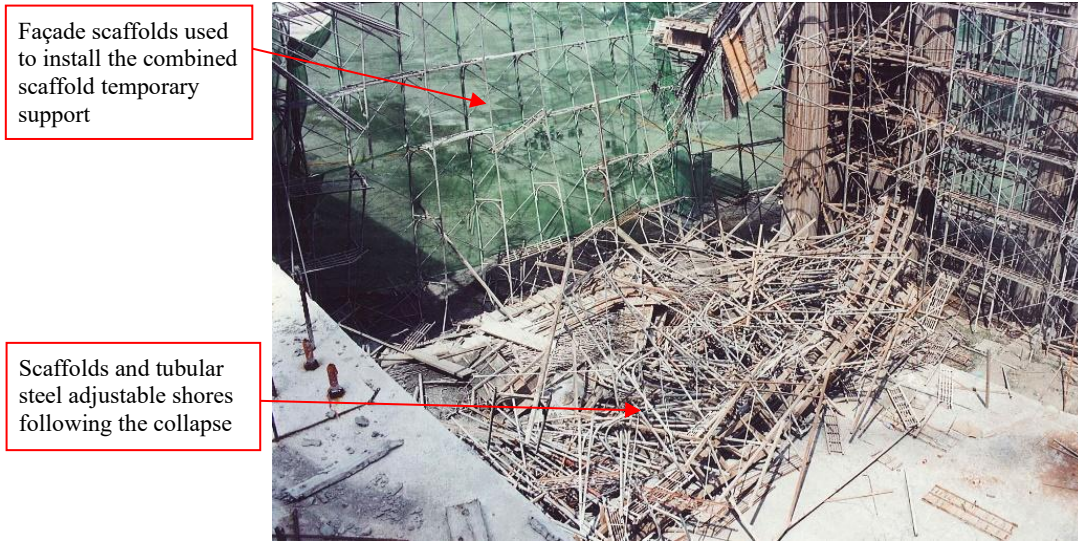


Fig. 6 Collapse of a combined scaffold temporary support at the girder of a hospital construction project in Chiayi, Taiwan

For research on steel scaffolds used in conjunction with other shores, Peng et al. (2020) [11] conducted full-scale outdoor loading tests on systems that combine steel scaffolds with wooden shores. This study compared the load-bearing capacity and failure modes of scaffolding structures with those of combined scaffolding assemblies. Earlier studies by Peng et al. (1998) [12] and Peng et al. (1996) [13] examined the mechanical behavior of these combined scaffolding assemblies. Their findings suggested that wooden shores or tubular steel adjustable shores positioned above the scaffolds could function as leaning columns. The longer these columns are, the more they reduce the overall critical load capacity of the combined scaffolding assemblies.

Research on temporary supports for single-story and double-story assemblies is less extensive than studies on steel scaffolds. Among the few studies on single-story assemblies, Peng (2013) [14] investigated the mechanical behavior of individual tubular steel adjustable shores, emphasizing their column buckling curve and reinforcement within structural systems. Peng (2012) [15] examined the ultimate load capacities of wooden shore systems under various top and bottom boundary conditions and those of combined systems that include both wooden shores and tubular steel adjustable shores. Peng (2002) [16] explored the theoretical relationship between single-story assemblies and the leaning column effect, tentatively concluding that merely increasing the number of leaning columns does not significantly enhance the critical loads of the entire single-story systems.

Peng (2010) [17] examined the critical loads of various temporary supports for wooden double-story assemblies and their reinforcement configurations. Additionally, Peng (2002) [18] conducted a preliminary theoretical exploration of the relationship between double-story assemblies and the leaning column effect. Similar to single-story assemblies, the research indicated that merely adding more leaning columns in double-story assemblies does not significantly enhance the total critical load.

Previous studies on steel scaffolds indicate that most focus solely on analyzing and testing the mechanical behavior of basic scaffolding assemblies, while few investigate combined scaffolds with top shores. These studies revealed that the leaning column effect reduces the critical load of combined scaffolds with top shores. Likewise, the leaning column effect impacts both single-story and double-story assemblies with unrestrained boundaries, and their overall critical loads do not increase with the addition of more leaning columns. The leaning column effect offers essential insight into why temporary supports frequently fail during concrete grouting, especially when the original design load of these supports far exceeds the actual load from the grouted fresh concrete.

2. Leaning column effect

The American Institute of Steel Construction (AISC) defines a *leaning column* as "a column designed to carry gravity loads only, with connections that are not intended to provide resistance to lateral loads" [1]. In simple terms, columns within a system that have pinned connections at both their top and bottom ends and do not possess lateral restraints on their boundary conditions are referred to as leaning columns. Below is a brief explanation of how the leaning column effect occurs.

A perfectly straight column, when subjected to axial load, will buckle once

this load reaches the column's critical load (P_{cr}). The critical load P_{cr} depends on the material, cross-sectional shape, length, and boundary conditions of the column. The critical load of the column is represented by Eq. (1). [19]

$$P_{cr} = \frac{\pi^2 EI}{(KL)^2} = \frac{P_e}{K^2}, \quad (1)$$

where

L : Length of the column;

E : Young's modulus of the column;

I : Moment of inertia of the column section;

K : Effective length factor of the column and

P_e : Euler buckling load $P_e = \pi^2 EI / L^2$ with $K=1.0$.

If the boundary conditions of a column change, its critical load will also change. For example, as illustrated in Fig. 7(a), for Column AB, the calculated effective length factor K_{AB} is 2. Substituting this effective length factor into Eq. (1) results in the critical load of this column, $P_{AB,cr1} (= P_e/2^2) = P_e/4$.

Fig. 7(b) shows an anti-symmetric frame. During compression, the effective length factor K_{AB} for Column AB in this frame remains 2, just like for Column AB in Fig. 7(a). Substituting the effective length factor into Eq. (1) produces the critical load for Column AB, $P_{AB,cr2} = P_e/4$, as demonstrated in Eq. (2). Eq. (3) illustrates the total critical load of the frame in Fig. 7(b) at the point of buckling, $P_{cr2} = P_e/2$.

$$P_{AB,cr2} = \frac{\pi^2 EI}{(2L)^2} = \frac{P_e}{(2)^2} = \frac{P_e}{4} = P_{AB,cr1} \quad (2)$$

$$\Sigma P_{cr2} = 2P_{AB,cr2} = 2 \frac{P_e}{4} = \frac{P_e}{2} \quad (3)$$

LeMessurier [6] was the first to introduce the concept of the leaning column effect. In a compressed structural system with unrestrained boundaries, columns with pinned connections at both ends (i.e., leaning columns) create the leaning column effect. This leaning column effect reduces the critical load of non-leaning compressed columns.

LeMessurier introduced a modified effective length factor K' to design columns affected by a leaning column effect within a structural system. The modified K' in the LeMessurier formula is represented in Eq. (4). [20]

$$K' = \sqrt{\frac{P_e}{P_l} \times \frac{\sum P}{\sum P_{eK}}}, \quad (4)$$

where

K' : Modified effective length factor of the column that provides sideways resistance;

P_e : Euler buckling load;

P_i : Column axial force contributing to sidesway resistance;

ΣP : Axial loads on every column in a story;

ΣP_{ek} : Euler loads for all columns in a story provide sidesway resistance for the frame, which is evaluated using the effective length derived from the nomograph.

The leaning column effect in the LeMessurier formula is illustrated by the frame in Fig. 7(c). Figs. 7(b) and 7(c) showcase two different types of portal frames. The only difference is the connection at point C of column CD on the right side; all other assumptions remain the same. At point C, the top of column CD changes from a rigid joint in Fig. 7(b) to a hinged joint in Fig. 7(c). In the system with an unrestrained boundary illustrated in Fig. 7(c), column CD acts as a leaning column.

In the frame depicted in Fig. 7(c), after undergoing compression loads, the leaning column effect increases the effective length factor K_{AB} of column AB. According to the LeMessurier formula (Eq. 4), $K_{AB} = 2.0$ increases to $K'_{AB} = 2\sqrt{2}$ as calculated in Eq. (5). Subsequently, the critical load of column AB is reduced to $P_{AB,cr3} = P_e/8$, as indicated in Eq. (6). The critical load for the entire frame is $\Sigma P_{cr3} = P_e/4$ at buckling, as shown in Eq. (7) in Fig. 7(c).

$$K'_{AB} = \sqrt{\frac{P_e \cdot \Sigma P}{P_1 \cdot \Sigma P_{ek}}} = \sqrt{\frac{P_e \cdot \frac{2P}{P} \cdot \frac{P_e}{(\frac{P_e}{2} + \frac{P_e}{\infty})^2}}{\frac{P_e}{P} \cdot \frac{P_e}{4}}} = \sqrt{\frac{P_e \cdot 2P}{P \cdot \frac{P_e}{4}}} = 2\sqrt{2} \quad (5)$$

$$P_{AB,cr3} = \frac{P_e}{(2\sqrt{2})^2} = \frac{P_e}{8} \quad (6)$$

$$\Sigma P_{cr3} = 2P_{AB,cr3} = 2 \cdot \frac{P_e}{8} = \frac{P_e}{4} \quad (7)$$

Eq. (8) provides a comparison of the total critical loads between frame (c) and frame (b) as depicted in Fig. 7. Because of the leaning column effect, the ratio of ΣP_{cr3} to ΣP_{cr2} is 1/2, indicating that the overall critical load of the frame in Fig. 7(c) is half that in Fig. 7(b).

$$\Sigma P_{cr3} / \Sigma P_{cr2} = (P_e/4) / (P_e/2) = 1/2 \quad (8)$$

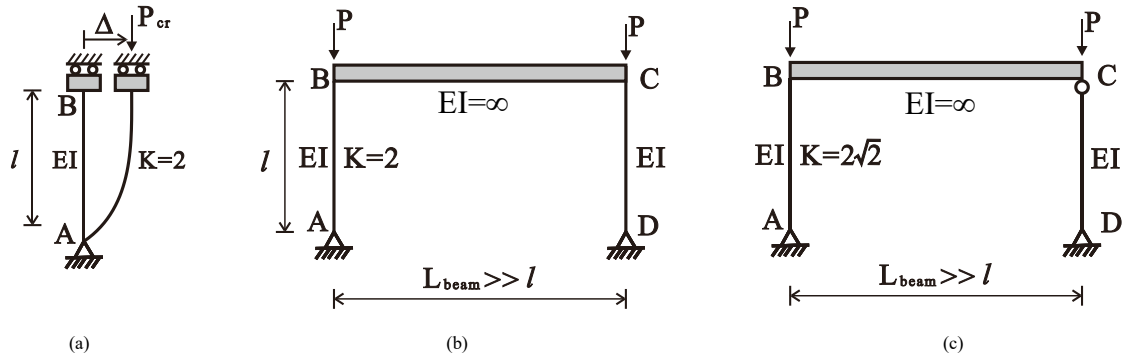


Fig. 7 K values of compression members and leaning columns in different structures

3. Relationship between the leaning column effect and temporary support failures

3.1. Single-story temporary support

Fig. 8 shows the arrangement of single-story temporary support using tubular steel adjustable shores at construction sites in Taiwan. The top of the tubular steel adjustable shore directly supports the wooden formwork of the slab, while the bottom rests on the ground. Since neither end is reinforced, these tubular steel adjustable shores cannot withstand bending moments at the top and bottom and are considered pinned ends in the analysis. If the structural system is unrestrained at the boundaries, each isolated tubular steel adjustable shore can be viewed as a leaning column. Additionally, construction crews employ horizontal ties to connect multiple tubular steel adjustable shores in two

perpendicular directions on the horizontal plane. These tied tubular steel adjustable shores can be regarded as an equivalent strong column, as illustrated in Fig. 8.

Fig. 9 is a schematic diagram depicting a two-dimensional simplified model that simulates a single-story temporary support at a construction site. It illustrates the relationship between a rigid slab, seven leaning columns, and one equivalent strong column. Fig. 10 explains the buckling behavior between leaning columns and the equivalent strong column under load in the single-story temporary support. According to stability theory, when a structural system undergoes buckling and reaches a state of neutral equilibrium, the critical load P_{cr} remains constant. Based on the small displacement theory, the horizontal displacement of the slab in the structural system can take any value at this load point.

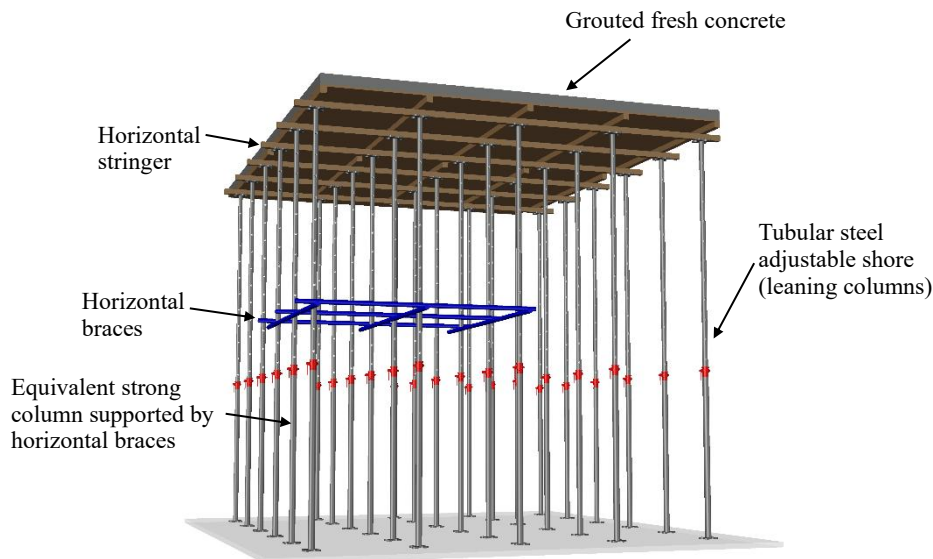


Fig. 8 Schematic diagram of leaning columns and equivalent strong columns in a single-story temporary support

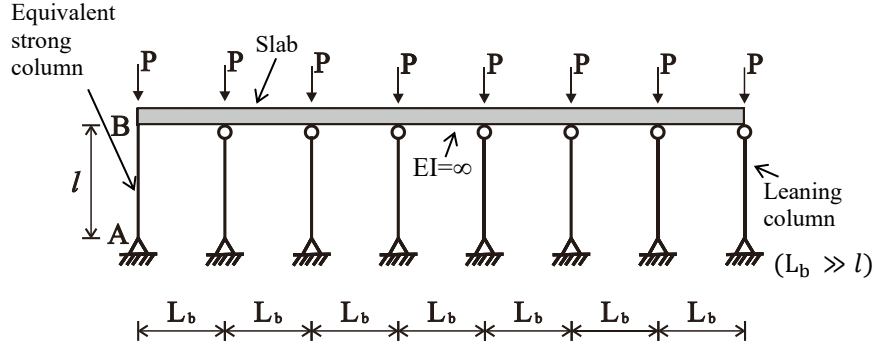


Fig. 9 Schematic diagram of a simplified model for single-story temporary support with unrestrained boundaries, showing leaning columns and an equivalent strong column

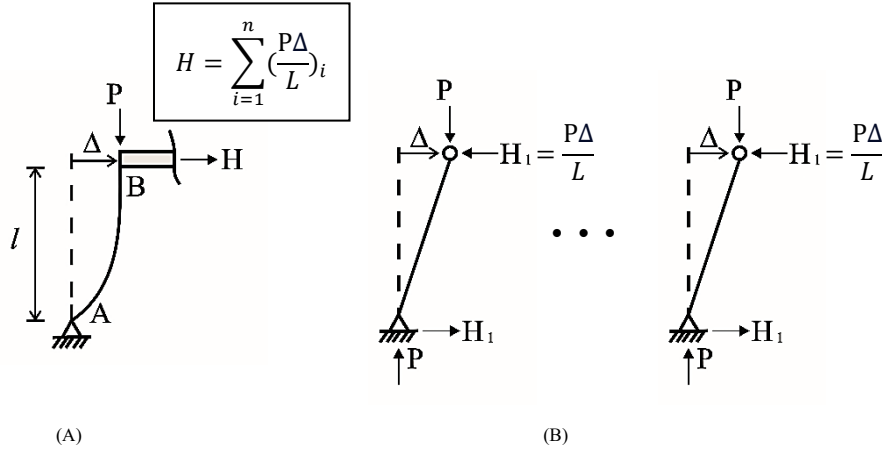


Fig. 10 Schematic diagram of a horizontal force induced by leaning columns in a simplified model for single-story temporary support with unrestrained boundaries

Fig. 10 shows the force equilibrium relationship of each column in Fig. 9 after being loaded. Fig. 10 also shows that the horizontal force H_i , caused by the leaning column effect in the simplified model of a single-story temporary support, is $P\Delta/L$. As shown in Fig. 10, the equivalent strong column AB generates a horizontal force H at its top due to the leaning column effect. This horizontal force H represents the total of all horizontal forces generated by all leaning columns, as expressed in Eq. (9):

$$H = \sum_{i=1}^n (\frac{P\Delta}{L})_i = 7(\frac{P\Delta}{L}) \quad (9)$$

In Eq. (9), P represents the buckling load P_{cr} , which remains constant when the structural system reaches buckling. L denotes the length of the column, and n refers to the number of leaning columns, which is seven. In a state of neutral equilibrium, the horizontal displacement Δ can assume any value; thus, as it increases, the horizontal force H also increases. As the horizontal force H applied to the top of the equivalent strong column AB increases, the critical load of column AB decreases. Hence, an increase in the number of leaning columns within the structural system leads to a larger total horizontal force resulting from buckling deformation, which subsequently reduces the critical load of the equivalent strong column [16].

In the example of seven leaning columns and one equivalent strong column from Fig. 9, the modified effective length factor for the equivalent strong column AB is $K'_{AB} = 4\sqrt{2}$, as shown in Eq. (10). Substituting $K'_{AB} = 4\sqrt{2}$ into Eq. (1) yields the critical load of the equivalent strong column AB, $P_{e,cr4}$, as indicated in Eq. (11). Consequently, the total critical load of the structural system can be calculated as $P_e/4$, as shown in Eq. (12).

$$K'_{AB} = \sqrt{\frac{P_e}{P_1} \cdot \frac{\Sigma P}{\Sigma P_{ek}}} = \sqrt{\frac{P_e}{P} \cdot \frac{8P}{(\frac{P_e}{2} + 7 \cdot \frac{P_e}{\infty})}} = 4\sqrt{2} \quad (10)$$

$$P_{AB,cr4} = \frac{P_e}{(4\sqrt{2})^2} = \frac{P_e}{32} \quad (11)$$

$$\Sigma P_{cr4} = 8 \cdot P_{AB,cr4} = 8 \cdot \frac{P_e}{32} = \frac{P_e}{4} \quad (12)$$

As shown in Eq. (13), comparing the overall critical load of the rigid frame in Fig. 9 with that in Fig. 7(b) indicates that, because of the leaning column effect, the ratio of ΣP_{cr4} from Eq. (12) to ΣP_{cr2} from Eq. (3) is 1/2. This means that the overall critical load of the rigid frame in Fig. 9 is half that in Fig. 7(b).

$$\Sigma P_{cr4} / \Sigma P_{cr2} = (P_e/4) / (P_e/2) = 1/2 \quad (13)$$

The result of the calculation for Eq. (13) matches that of Eq. (8), indicating that the structure with one leaning column shown in Fig. 7(b) and the structure with seven leaning columns depicted in Fig. 9 both have the same overall critical load. As illustrated in Fig. 9, merely increasing the number of leaning columns to seven in the single-story temporary support does not significantly enhance the overall critical load of the structural system.

Alternatively, if we calculate the total critical load of the structure shown in Fig. 9 using the critical load $P_{AB,cr1}$ ($= P_e/4$) for a single column AB, as illustrated in Fig. 7(a), the total critical load for Fig. 9 will be $\Sigma P'_{cr4}$ ($= 2P_e$), as indicated in Eq. (14). If a design safety factor of 2 is applied, $\Sigma P'_{cr4,design}$ ($= P_e$), is calculated as demonstrated in Eq. (15). The actual critical load ΣP_{cr4} in Eq. (12) is one-fourth of the design critical load $\Sigma P'_{cr4,design}$ in Eq. (15), which is calculated as indicated in Eq. (16).

$$\Sigma P'_{cr4} = 8 \cdot (P_e/4) = 2P_e \quad (14)$$

$$\Sigma P'_{cr4,design} = \Sigma P'_{cr4} / S.F. = 2P_e / 2 = P_e \quad (15)$$

$$\Sigma P_{cr4} / \Sigma P'_{cr4,design} = (P_e/4) / (P_e) = 1/4 \quad (16)$$

Considering the fresh concrete grouted on the single-story temporary support illustrated in Fig. 9, the leaning column effect reveals that the structure attains its critical load when the grouted concrete reaches one-fourth of the design load. This leads to an immediate buckling collapse due to minor disturbances affecting the single-story temporary support.

In summary, columns with pinned ends can be regarded as leaning columns in the single-story temporary support with unrestrained boundaries. During buckling, the leaning column effect significantly reduces the overall critical load of the structural system. However, if the single-story temporary support is adjusted to fit the restrained boundaries, the leaning column effect will disappear, as shown in Fig. 11. The effective length factor of the equivalent strong column AB will change from ≥ 1 to ≤ 1 . The original 'leaning columns', with pinned ends and unrestrained boundaries, will have their effective length factor changed from ∞ to 1.0, significantly enhancing the overall critical load of the structural system.

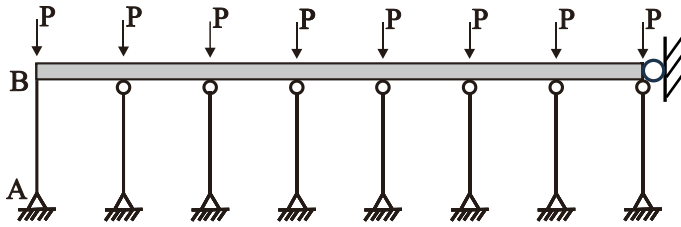


Fig. 11 Schematic diagram of a simplified model with restraint boundary conditions for single-story temporary support to enhance load-bearing capacity

3.1.1. Collapse case of single-story temporary support and leaning column effect

Fig. 12 illustrates the installation of temporary support using single-story tubular steel adjustable shores in the collapse area of the vehicular box culvert across the Second Expressway in Kaohsiung, Taiwan. Fig. 13 shows that the collapsed region of the vehicular box culvert's top slab was 20 m long, 9 m wide, and 1.1 m thick, featuring a temporary support height of 5.3 m. The total weight of the concrete slab supported by the temporary support was approximately 475 tonnes ($= 2.4 \times 20 \times 9 \times 1.1$). The vehicular box culvert was built in two stages, as

illustrated in Fig. 12. The right side was completed first, while work on the left side started one month later. The single-story temporary support on the left side collapsed during the grouting process, as shown in Fig. 4. Fig. 12 also shows the concrete grouting path of the slab on top of the vehicular box culvert.

This project used a single-story temporary support with tubular steel adjustable shores, each measuring 5.3 m in length. These steel shores were neither welded nor bolted at their top and bottom ends, which makes them effectively pinned joints for analysis. If the boundaries of the assembly system are unrestrained, these independent tubular steel adjustable shores can be regarded as 'leaning columns.' Additionally, several tubular steel adjustable shores were linked by multiple horizontal braces, forming a cohesive structure. These braced tubular steel adjustable shores can be regarded as one 'equivalent strong column,' as illustrated in Figs. 12 and 13.

The vertical walls on both sides of the left box culvert in this project have been completed, offering lateral support for the top slab of the temporary support. However, in the perpendicular direction, there were no side walls to provide lateral support for the top slab, as shown in Fig. 14, which illustrates the possible direction of slab displacement. The movement of the top slab in this single-story temporary support can cause the leaning column effect in the structural system, significantly reducing the overall critical load. When global buckling and instability occur, the structure may experience a rapid, domino-effect collapse following a minor disturbance, while most components remain intact until a failure happens. As illustrated in Fig. 4, following the collapse of the temporary support, the outermost tubular steel adjustable shore, which shifted outward, remained intact and was not crushed.

If the engineers missed the reduction in the critical load of the overall structure caused by the leaning column effect when calculating the temporary support strength for the single-layer assembly in this project, it is highly likely that during the grouting process, the weight of the fresh concrete on the top slab would reach the critical load, resulting in structural instability. At this point, the impact force from the pumped fresh concrete on top of the temporary support could cause a minor disturbance, potentially triggering a domino-like collapse of the entire single-story temporary support.

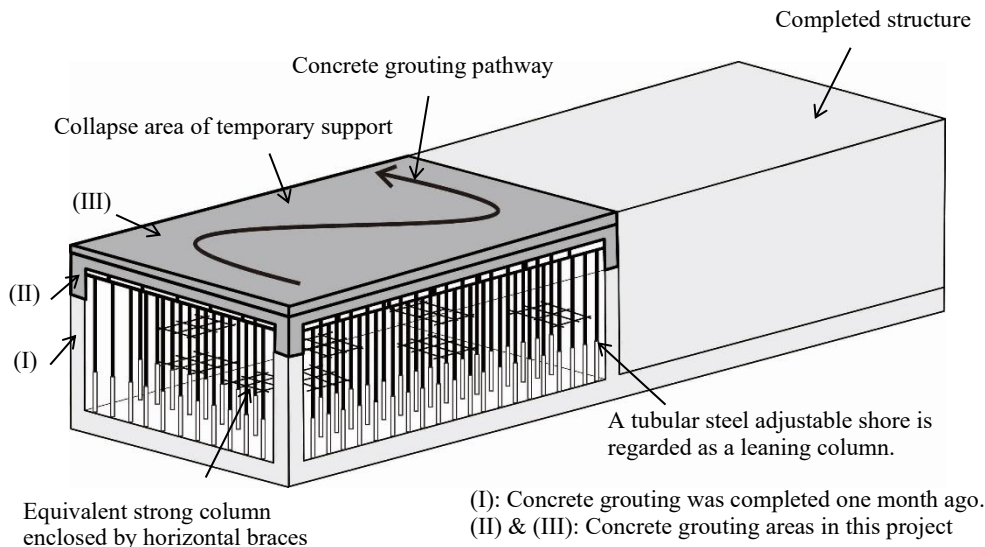


Fig. 12 Schematic diagram depicting the single-story temporary support in the collapse area of the box culvert across the Second Expressway in Kaohsiung, Taiwan

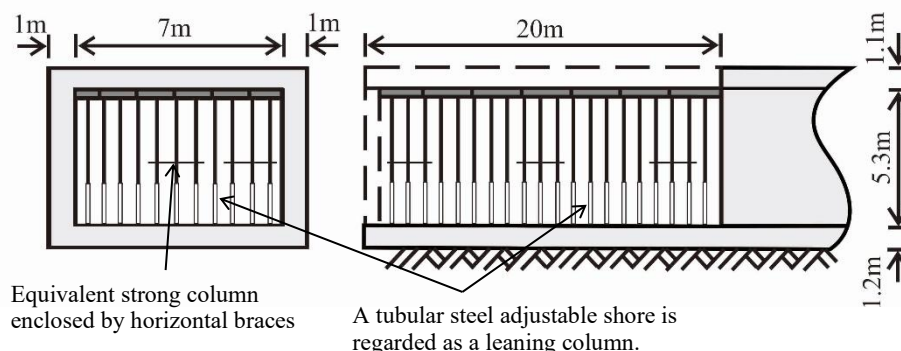


Fig. 13 Front and side views of the single-story temporary support in the collapse area of the box culvert across the Second Expressway in Kaohsiung, Taiwan

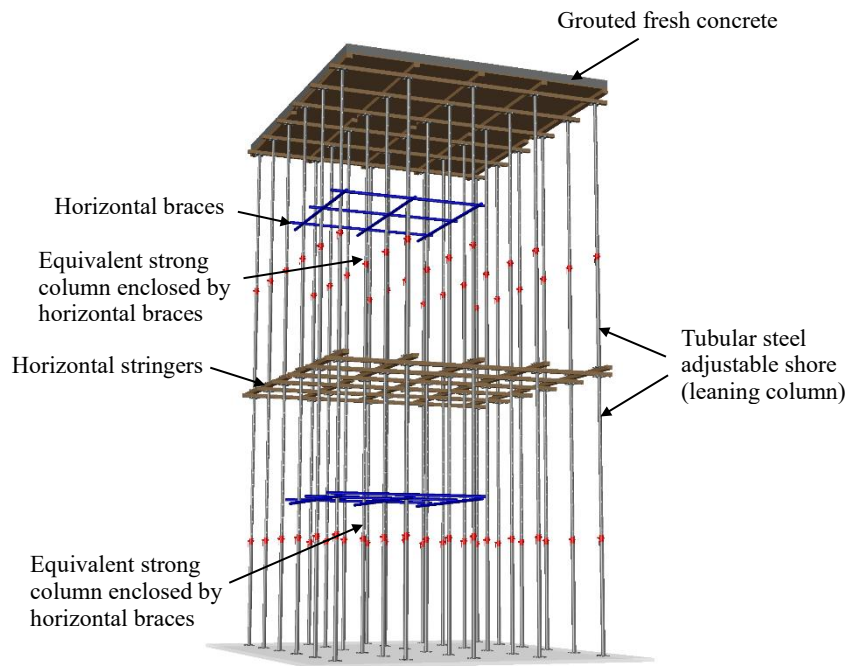


Fig. 16 Schematic diagram showing the arrangement of leaning columns and equivalent strong columns in a double-story temporary support

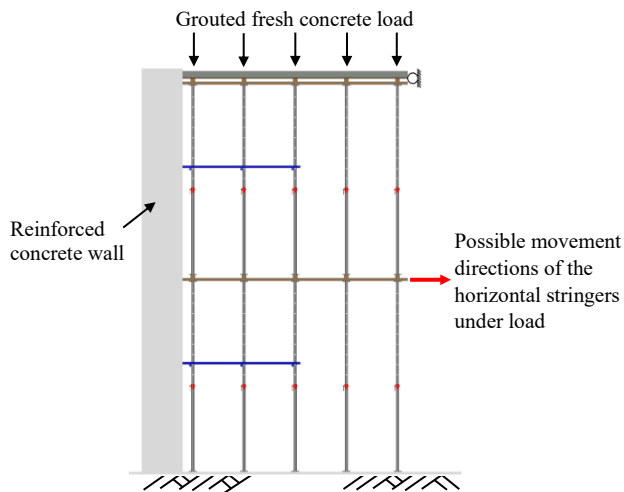


Fig. 17 Schematic diagram of the lateral displacement of horizontal stringers in double-story temporary support under load

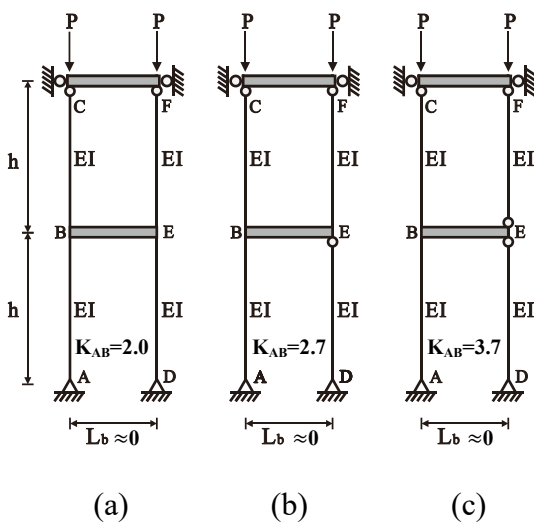


Fig. 18 The effective length factor K_{AB} for equivalent strong columns with different numbers of leaning columns in double-story temporary supports

Transforming unrestrained boundary conditions to restrained ones is more complex for double-story temporary supports than for single-story ones. The intermediate horizontal wooden stringers in double-story temporary supports are 3.6 m long and are typically not connected by opposite joints on-site. As a result, even if the ends of the horizontal stringers, which undergo no displacement, are directly braced against hardened reinforced concrete (RC) walls or columns, these horizontal stringers may shift inward toward the center of the unconnected area when the vertical shores are loaded. This inward horizontal movement of the stringers causes the leaning column effect in the double-story temporary support.

Many years ago in Taiwan, numerous collapses occurred due to the use of double-story temporary supports in residential construction projects. Recognizing the harmful mechanism of the leaning column effect, the Ministry of Labor, as the authority on occupational safety, prohibited the use of double-story temporary supports in urban residential construction. As a result, the temporary support failures in residential construction projects in Taiwan have significantly decreased.

3.2.1. Collapse case of double-story temporary support and leaning column effect

The Linkou residential project in Taipei, Taiwan, utilized a top-down construction method, and Fig. 19 shows the arrangement of double-story temporary supports. The temporary supports utilized tubular steel adjustable shores, and the arrangement of the upper and lower tubular steel adjustable shores resembles that of single-story temporary support. Concrete grouting is set to commence at the building's far end, as illustrated in Fig. 19. The temporary support collapsed instantly during the building's one-sided grouting process, as illustrated in Fig. 5.

The building had a headroom of 595 cm. The double-story temporary support utilized tubular steel adjustable shores, with the lower story height at 334.5 cm and the upper story height at 180.5 cm, as shown in Figs. 19 and 20. In this area, the slab thickness was measured at 20 cm, the beam depth was 60 cm, and the girder depth was 100 cm. The average equivalent thickness of the slab was calculated to be approximately 30 cm, and the total weight of the concrete in the collapsed slab area was about 104 tonnes ($= 2.4 \text{ tonnes/m}^3 \times 18\text{m} \times 8\text{m} \times 0.3\text{m}$).

The double-story temporary support for this project employs horizontal wooden stringers. Insufficient horizontal lateral support at these stringers caused them to shift horizontally after grouting, as depicted in Fig. 21. The moving of the stringers resulted in the leaning column effect within the structural system, significantly reducing the overall critical load of the double-story temporary support and ultimately leading to a collapse.

The failure of the structural system, caused by instability from the leaning column effect, is distinct from the overall failure of the structure due to the yielding of individual components. The double-story temporary support

collapsed from buckling on the construction site, as shown in Fig. 5. Fig. 5 shows that the outermost tubular steel adjustable shores that fell to the ground remain intact and undamaged. This type of failure aligns with the scenario

illustrated in Fig. 4, where the outermost tubular steel adjustable shores in the single-story temporary support remained intact during the instability and collapse.

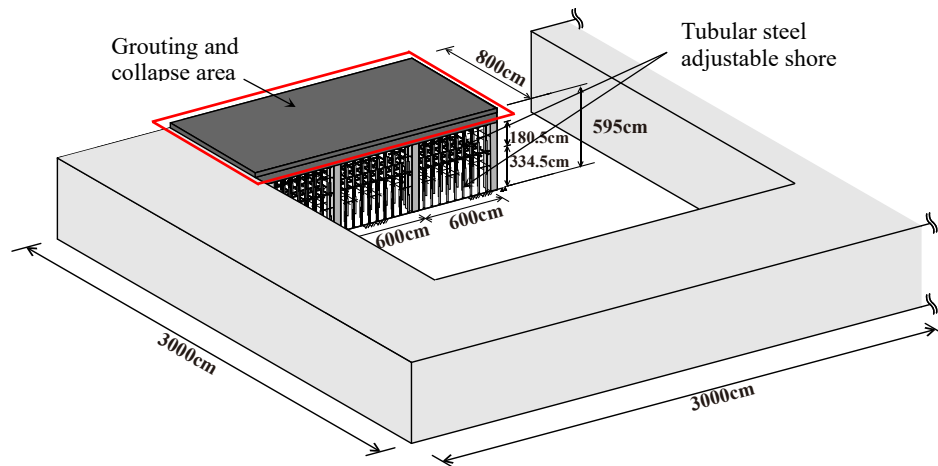


Fig. 19 Schematic diagram of double-story temporary support at the Linkou residential project in Taipei, Taiwan

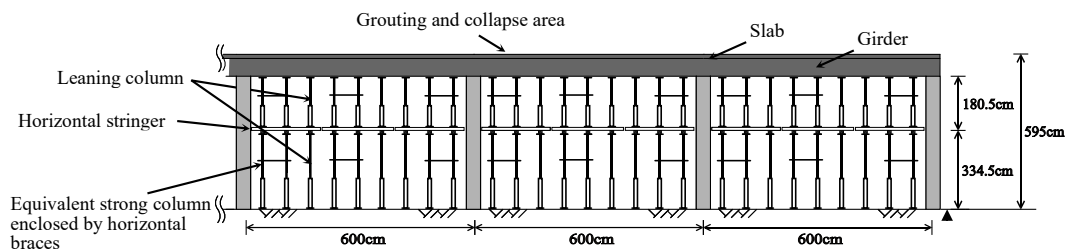


Fig. 20 Front view of the double-story temporary support in the collapse area at the Linkou residential project in Taipei, Taiwan

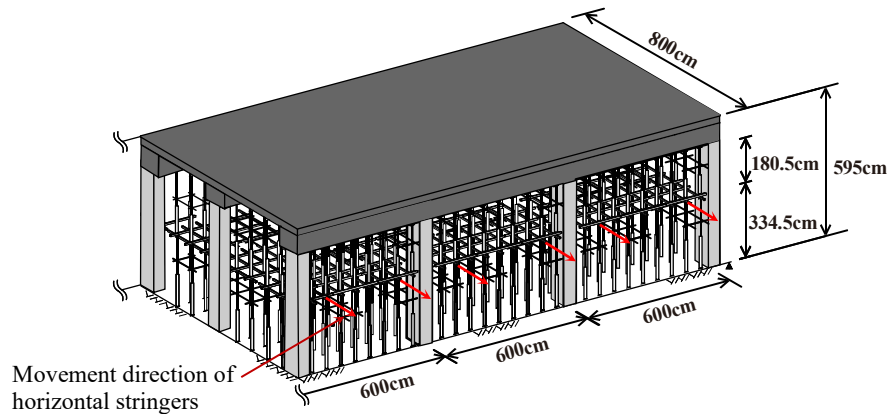


Fig. 21 Schematic diagram of the movement direction of horizontal stringers in a double-story temporary support collapse zone at the Linkou residential project in Taipei, Taiwan

3.2.2. Improvement strategy

It is advised that this project substitutes the double-story temporary support with a straightforward scaffolding system to remove the leaning column effect. Once the grouted fresh concrete is loaded onto the two-story temporary support, the wooden horizontal stringers connecting the upper and lower supports shift outward, causing a leaning column effect. This construction project employed a top-down construction method. Nevertheless, it is practically difficult to reinforce the horizontal stringers in the center of the double-story temporary supports with lateral supports. Additionally, the wooden horizontal stringers are 3.6 m long, and due to cost considerations, these stringers were not connected using opposite joints at the construction site. Therefore, even if the outermost ends are reinforced with lateral supports, the unconnected horizontal stringers will likely shift inward under load, which still leads to the leaning column effect. Therefore, the best approach is to substitute the support material by eliminating the double-story temporary support and employing a steel scaffolding structure that extends straight from the ground to the slab's underside.

3.3. Combined scaffold temporary support

In Taiwan, when the headroom of a building exceeds 7 m, construction contractors often utilize the temporary support made of tubular steel scaffolds in conjunction with other shores (i.e., combined scaffold temporary support), as illustrated in Fig. 22. Fig. 22 illustrates that the overall combined scaffold temporary support consists of two parts: the lower part comprises multiple stories of steel scaffolds, while the upper part includes tubular steel adjustable shores or wooden shores.

Previous studies by Peng et al. [11–13] have demonstrated that the top and bottom ends of tubular steel adjustable shores and wooden shores in the upper section do not resist bending moments. These shores resemble leaning columns in analysis. In a structural system with an unrestrained boundary, leaning columns under compression can lead to the leaning column effect, which decreases the overall critical load of the combined scaffold temporary support. The critical load of the combined system is less than that of a solely installed steel scaffold system.

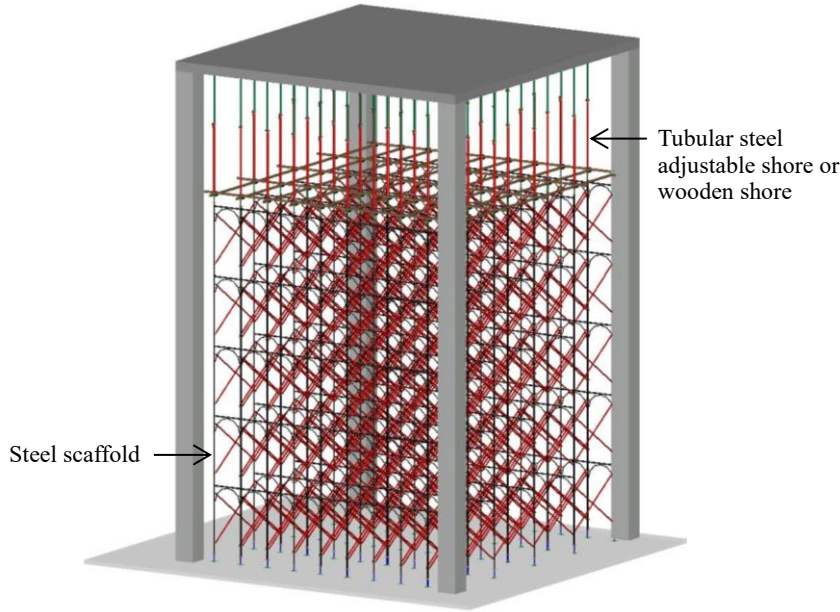


Fig. 22 Schematic diagram of combined scaffold temporary support

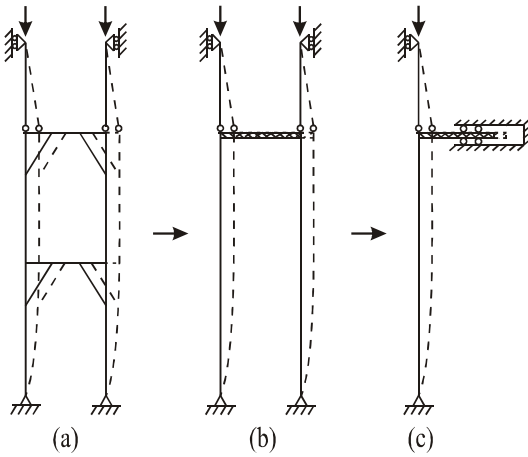


Fig. 23 Simplification process of the double-story combined scaffold temporary support model

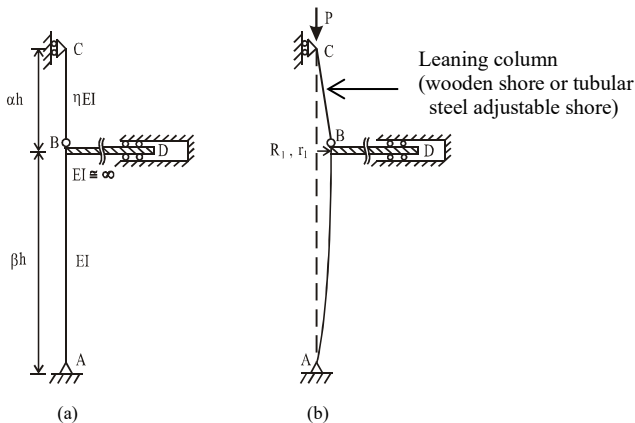


Fig. 24 Parameter definitions and leaning column in the simplified model of combined scaffold temporary support

To examine the mechanical behavior of the combined scaffold temporary support, we can analyze an individual unit from it, as shown in Fig. 22. For simplicity, consider a two-story individual unit of combined scaffold temporary support, as shown in Fig. 23(a). Fig. 23 illustrates the simplification process of a 2D, two-story combined scaffold temporary support model. First, the internal diagonal braces of the scaffolds in Fig. 23(a) are removed to simplify the scaffold stiffness, as shown in Fig. 23(b). Secondly, research by Peng et al. (1998) [12] indicates that the anti-symmetric structural behavior of the

structures shown in Figs. 23(a) and 23(b) under load enables a further simplification of Fig. 23(b) into Fig. 23(c).

Fig. 23(c) illustrates the final simplified model of the two-story combined scaffold temporary support. In this simplified model, the connection between the scaffolds and the upper tubular steel adjustable shores or wooden shores is assumed to permit only horizontal movement. This assumption arises from observing that horizontal stringers are generally fully installed at construction sites, giving them a stiffness greater than that of scaffolds, tubular steel adjustable shores, or wooden shores. In this simplified model, the horizontal stringers can initially be regarded as a rigid body experiencing only horizontal displacement [12].

Fig. 24(a) defines analysis parameters for the simplified model in Fig. 23(c), while Fig. 24(b) illustrates the leaning column effect in this model. In Fig. 24(a), the equivalent stiffness of the scaffolds in the lower AB section is represented as $EI/(\beta h)$. Additionally, the equivalent stiffness of the tubular steel adjustable shore or the wooden shore in the upper BC section is denoted as $\eta EI/(\alpha h)$.

The horizontal stringer BD in Fig. 24(b) can only move horizontally, so the simplified model has one degree of freedom. According to the research conducted by Peng et al. (1998) [12], the critical load of the simplified model of the combined scaffold temporary support, influenced by the leaning column effect, is expressed in Eq. (18) [12]:

$$P_{cr} = \frac{3 \frac{\alpha}{\beta^3} \frac{EI}{h^2}}{1 + \frac{6\alpha}{5\beta}} \quad (18)$$

where

β is the number of scaffolding stories

h is the height of a single scaffolding story (1.7 m)

α is a parameter ranging between 0 and 1

η is a parameter ranging between 0 and 1

In Fig. 24(a), when α and β are both set to 1, the leaning column effect is illustrated through the simplified model of the combined scaffold temporary support depicted in Fig. 25(a). The slope-deflection method can be used to calculate the effective length factor K_{AB} of column AB in the structure shown in Fig. 25(a). The calculated effective length factor K_{AB} for column AB is 2.69 [12].

Fig. 25(b) shows a portal frame from the American Institute of Steel Construction (AISC) design code for steel structures, illustrating the leaning column effect [1, 20]. The effective length factor K_{AB} for column AB in this frame, calculated with the slope-deflection method, is 2.69. Thus, Figs. 25(a) and 25(b) display the same K_{AB} . Additionally, in Fig. 7(c), the modified K'_{AB} ($= 2\sqrt{2}$), derived from Eq. (4) (the LeMessurier formula), serves as an approximate solution for design purposes. Fig. 25 compares two examples that illustrate why the upper tubular steel adjustable shores or wooden shores in a combined scaffold serve as leaning columns. It also clarifies how this leaning column effect impacts the critical load of the combined scaffold temporary support.

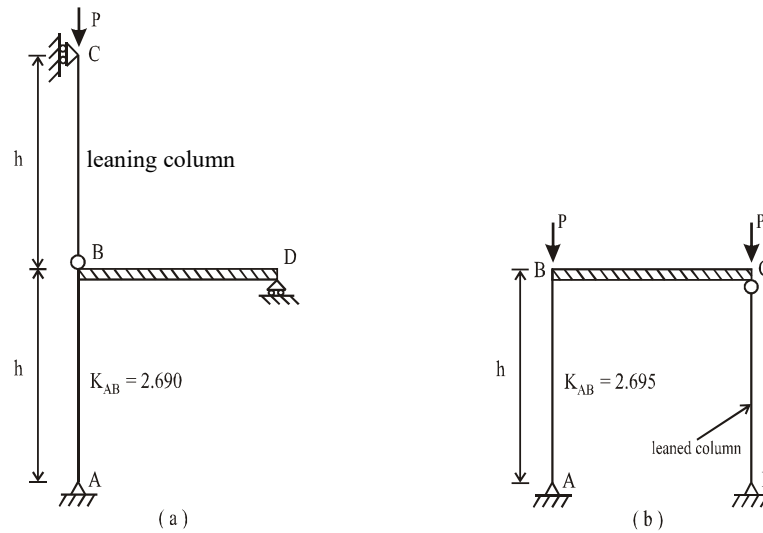


Fig. 25 The leaning column effect relationship between the simplified model of combined scaffold temporary support and the portal frame [12]

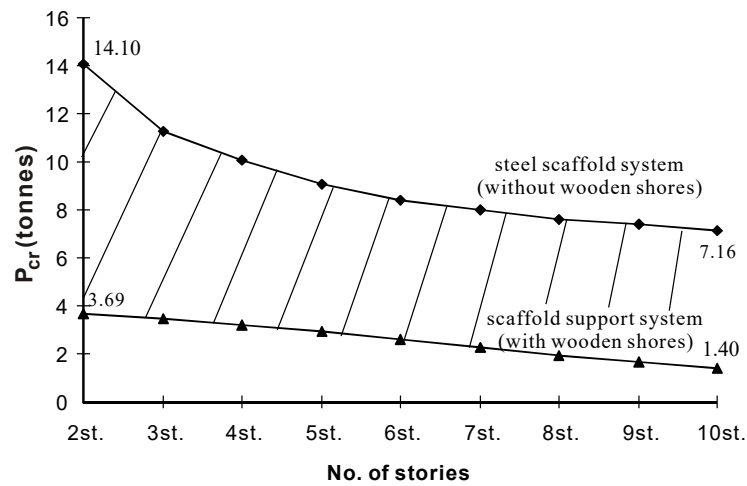


Fig. 26 A comparison of critical loads between scaffold systems with and without wooden shores at the top [13]

According to research by Peng et al. (1996) [13], a comparison of the critical load P_{cr} was conducted using the second-order elastic analysis for two structural systems: (1) a scaffold system without wooden shores and (2) a scaffold system with top wooden shores measuring 1.7 m in length. As shown in Fig. 26, the critical load for a two-story scaffold system without wooden shores is 14.10 tonnes. However, when a 1.7 m long wooden shore is added to the top of the scaffolds, the critical load decreases to 3.69 tonnes. The ratio of the critical load for the two-story scaffold system with wooden shores to that of the two-story scaffold system without wooden shores is approximately 0.3 ($=3.69/14.1$). For a ten-story scaffold system with wooden shores, the critical load ratio compared to one without wooden shores is approximately 0.2 ($=1.40/7.16$) [13].

Fig. 26 shows the differences in critical loads between the scaffold system without wooden shores, which is unaffected by the leaning column effect, and the scaffold system with wooden shores, which is affected by the leaning column effect. The critical load ratios for two-story and ten-story scaffold systems, both with and without wooden shores, are 0.3 and 0.2, respectively. This highlights the significant influence of the leaning column effect on the critical load of combined scaffold temporary supports.

Additionally, when calculating the critical load, it is noted that for scaffold systems exceeding five stories, the ratio of critical loads between systems with and without wooden shores tends to stabilize at a constant value of approximately 0.2. This suggests that, given the same number of stories, the critical load of a scaffold system without wooden shores can be used to estimate the critical load for a scaffold system with wooden shores during the design process. This parameter is necessary because conducting the loading test of the scaffold system without wood shores is easier. The loading test value of the scaffold system without wood shores is multiplied by this parameter to determine the critical load of the scaffold system with wood support.

Additionally, the loading test results for the scaffold system with wooden shores indicate that the wooden shores remained undamaged when the system collapsed due to buckling. Thus, using tubular steel adjustable shores instead of wooden shores results in a similar leaning column effect that impacts the results.

3.3.1. Collapse case of combined scaffold temporary support and leaning column effect

The entrance hall of the hospital in Chiayi, Taiwan, features four girders that enclose an open area designed to support section steels and skylights, as shown in Fig. 27. A combined scaffold temporary support under the three girders consists of steel scaffolds topped with tubular steel adjustable shores. The upper level of this combined scaffold temporary support includes 3.6 m tubular steel adjustable shores, while the lower level consists of two-story steel scaffolds that measure 3.4 m in height. The overall height of this temporary support, including the thickness of the horizontal stringers and U-shaped adjustment seats, is 7.2 m.

The hospital entrance girder is 8.4 m high, 1.2 m deep, and 0.8 m wide. In the collapse area, it measures 18 m long, as shown in Fig. 28. The combined scaffold temporary support assists the reinforced concrete girder, which weighs approximately 41.5 tonnes ($= 2.4 \text{ tonnes/m}^3 \times 0.8 \text{ m} \times 1.2 \text{ m} \times 18 \text{ m}$). In the combined scaffold temporary support, these upper tubular steel adjustable shores can be considered leaning columns. Likewise, the structural system is influenced by the leaning column effect. As shown in Fig. 26, the critical load of the two-story scaffold system with top shores is approximately 0.3 ($=3.69/14.1$) times that of the two-story scaffold system without top shores. The designer may mistakenly evaluate the bearing capacity of the latter structural system while assessing the bearing capacity of the former, which could potentially lead to collapse.

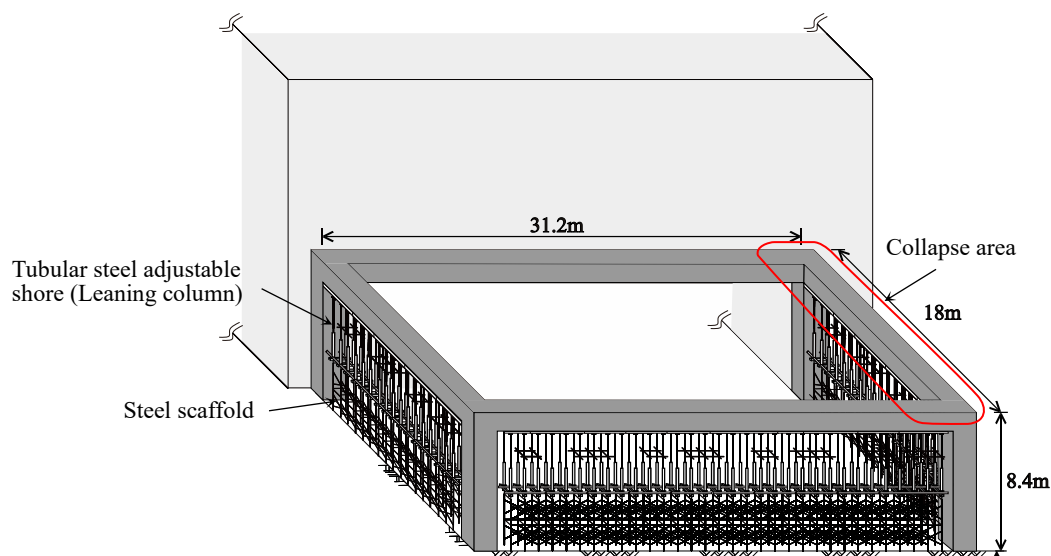


Fig. 27 Schematic diagram of a combined scaffold temporary support at the entrance hall of the hospital in Chiayi, Taiwan

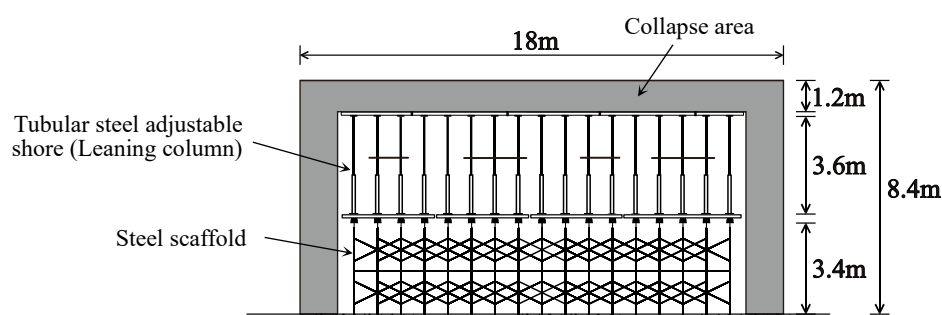


Fig. 28 Front view of the combined scaffold temporary support collapse area at the entrance hall of the hospital in Chiayi, Taiwan

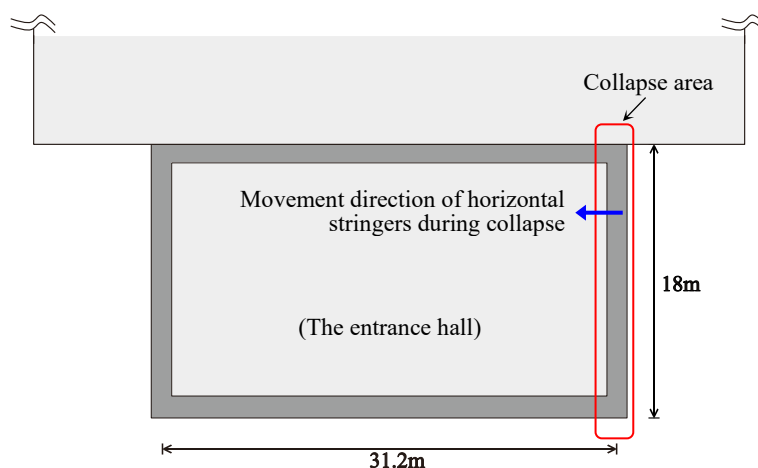


Fig. 29 Schematic diagram of the movement direction of horizontal stringers of temporary support during collapse at the entrance hall of the hospital in Chiayi, Taiwan

After grouting the side girder at this construction site, the combined scaffold temporary support buckled when the grouted fresh concrete reached its critical load. The vibrations from pumping fresh concrete were transmitted to the combined scaffold temporary support. This instability causes rapid deformation, ultimately resulting in collapse, as shown in Fig. 6. Fig. 29 illustrates the movement direction of the horizontal stringers in the combined scaffold temporary support during its collapse.

3.3.2. Improvement strategy

To prevent the combined scaffold temporary support from collapsing, it is advisable to replace the scaffold system with top shores with one without top shores. In simple terms, this means taking out the tubular steel adjustable shores from the upper section of the combined scaffold temporary support and directly extending the scaffold installation down to the bottom of the slabs and beams.

Height differences can be adjusted using the upper and the lower jake bases of scaffolds, effectively eliminating the influence of the leaning column effect.

Fig. 26 shows that using only the steel scaffolding assembly, without the top shores, significantly increases the total critical load of the temporary supports compared to the scaffold system with top shores. Furthermore, to ensure that this scaffold system without top shores can withstand lateral impact forces and to further enhance its overall critical load capacity, it is advisable to add a third row of scaffolds to the existing two-row configuration, resulting in a three-row scaffolding setup.

4. Conclusion

This study investigated the failure mechanisms that lead to the collapse of single-story, double-story, and combined scaffold temporary supports during

concrete grouting at construction sites in Taiwan. The research is based on the theory of the leaning column effect proposed by LeMessurier and the prior studies conducted by Peng et. al. concerning the relationship between temporary supports and the leaning column effect in various temporary support systems. This study finds that the leaning column effect is the primary failure mechanism responsible for the collapse of single-story, double-story, and combined scaffold temporary supports during concrete grouting, as determined through analyses and calculations grounded in stability theory. This paper further examines actual failure cases involving three types of temporary supports: single-story, double-story, and combined scaffold temporary supports in Taiwan. It concludes that the leaning column effect sufficiently explains these failures. The paper proposed several strategies to enhance critical load capacity and prevent collapses, emphasizing that the most effective approach is to eliminate the leaning column effect in structural systems. Professional engineers should modify the boundary conditions of single-story, double-story, and combined scaffold temporary supports from unrestrained to restrained. In other words, the boundary with lateral displacement is replaced by the boundary without lateral displacement. Additionally, it is advisable to avoid double-story temporary support to reduce the risk of collapse.

Acknowledgement

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Conversion factors

1 kg = 1 kgf
 1 kgf = 9.807 N
 1 tonne = 9.807 kN
 9.807 kN = 1000 kg

Reference

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RESEARCH ON THE COLLABORATIVE LOAD-BEARING MECHANISM BETWEEN ROOF SLAB AND SOCKET-TEMPLATE SCAFFOLD

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ABSTRACT

With the rapid development of large-span architectural facilities, the application of scaffolding support systems has progressively increased. Due to the inherent initial imperfections of scaffolding and the complex mechanical behaviors at connection joints, higher stability requirements are imposed on scaffolding systems. Current analyses of scaffolding support systems predominantly focus on their spatial overall stability, while insufficient attention has been paid to the contribution of the collaborative load-bearing interaction between roof slabs and scaffolding to structural reliability. This study investigates the collaborative load-bearing mechanism between roof slabs and scaffolding, using the Yongchuan District Convention and Exhibition Center project as an engineering case. Through finite element modeling and analysis with ANSYS software under various erection parameters, the moment-rotation curves of connection joints are compared with existing experimental data, validating the reliability of semi-rigid connections in the numerical model. Additionally, considering the stochastic nature of geometric initial imperfections, the influence of two types of geometric initial imperfections on the load-bearing capacity distribution of the collaborative system is quantitatively analyzed. The conclusions provide a rational design basis for roof slab-scaffolding collaborative load-bearing systems and establish a foundation for similar future engineering projects.

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Stochastic initial imperfections;
Semi-rigid connections;
Load-bearing capacity

1. Introduction

Under the rapid development of China's economy, the quality of life has been significantly improved, accompanied by continuous advancements in urban modernization. Various types of architectural structures have emerged, particularly in industrial construction, where large-span public buildings, such as airport terminals, high-speed railway stations, auditoriums, theaters, and museums, have been increasingly constructed ^[1]. Consequently, the application of scaffolding as a support system has expanded accordingly. In practice, on-site construction processes rarely involve independent operation of the scaffolding support system alone. Instead, it is more rational to analyze the scaffolding system and the upper concrete floor slab as an integrated structure from an engineering perspective.

This study adopts the Yongchuan District Convention and Exhibition Center project as the engineering context to conduct finite element simulation analyses of critical construction challenges within the project. Due to the limited bearing capacity of the project's foundation, a mobile crane was deployed on the first-floor slab for hoisting operations. In this scenario, the slab, as a hardened structural component, participates in load-bearing, while the scaffolding serves as a slab-type support system (as shown in Fig. 1). The upper structure consists of a concrete floor used for construction activities and safety measures, whereas the lower scaffolding support system remains non-removable until the completion of the upper construction. The upper concrete floor and the temporarily installed scaffolding support system below collectively bear the construction loads from above.



Fig. 1 Schematic diagram of the collaborative load-bearing system between the floor slab and scaffolding



Fig. 2 Engineering accidents caused by scaffold collapse

Due to its advantages of excellent convenience, high construction efficiency, and cost-effectiveness, scaffolding has been widely utilized in building construction processes. However, engineering accidents frequently occur during scaffolding usage due to multiple factors. Existing data indicate that approximately two-thirds of engineering accidents occur during construction phases, with the majority caused by scaffolding collapses [2], as shown in Fig. 2. This has prompted extensive research by scholars through experimental and numerical methods to investigate the structural performance of scaffolding systems [3–7]. Analysis of recent accident cases reveals that the primary causes lie in insufficient strength or stiffness of scaffolding support systems and weak overall structural stability. Therefore, research on the load-bearing capacity distribution of the roof slab-scaffolding collaborative system holds significant engineering importance.

Scaffolding support systems demonstrate advantages such as high strength and expansive planar dimensions [8]. Primarily composed of vertical members and horizontal members, these temporary structural components require rapid assembly and reconfiguration, imposing stringent demands on the mechanical properties of connection joints. Extensive research has been conducted on the influence of joint stiffness on the load-bearing capacity of scaffolding. Results indicate that semi-rigid connections significantly affect the mechanical behavior of primary structures. Consequently, this study adopts semi-rigid connections for scaffolding joints.

Numerous investigations have focused on semi-rigid connections [9–15]. Wang [16] proposed a theoretical computational model for modular structures incorporating semi-rigid internal connections. This model facilitated the derivation of structural lateral stiffness formulas, revealing the force transfer mechanism at modular joints. They further developed a simplified calculation method employing equivalent linear stiffness, providing universality for modular structures with semi-rigid or rigid connections. Alqattan [17] examined the mechanical performance and load-bearing capacity of scaffolding systems, particularly comparing disc-lock and bowl-lock scaffolding joints through full-scale tests. Their experimental analysis of bearing capacity and deformation characteristics demonstrated that disc-lock joints exhibit superior resistance to minor bending loads compared to bowl-lock joints, highlighting their enhanced anti-bending properties. Additionally, disc-lock scaffolding demonstrated exceptional load-bearing capacity. Pienko [18] conducted comprehensive experimental studies on disc-lock scaffolding joints. Utilizing specialized loading machinery and custom-designed clamps, they applied normal forces, shear forces, bending moments, and torsional loads to evaluate joint performance. Experimental data revealed that these joints behave as hinged connections under horizontal bending and torsion while exhibiting semi-rigidity under vertical bending. Prabhakaran [19] investigated joint looseness in scaffolding systems, developing an algorithm to simulate nonlinear loading/unloading behavior and moment-rotation curves incorporating joint slackness. They implemented various approximations of moment-rotation curves in simplified scaffolding models.

Furthermore, geometric initial imperfections inevitably affect scaffolding during practical applications. These imperfections constitute a critical factor requiring careful consideration when investigating the load-bearing capacity distribution of roof slab-scaffolding collaborative systems. Zhang [20] investigated the load-bearing capacity of scaffolding systems, noting that their reliability is inferior to conventional building structures due to the mechanical complexity of joint connections and stochasticity induced by initial imperfections. To facilitate the analysis of random geometric imperfections, they developed a finite element-based algorithm capable of automatically incorporating stochastic geometric imperfections. The established numerical model demonstrated close alignment with actual conditions, revealing that buckling modes in real-world scenarios are intrinsically correlated with the random distribution of geometric imperfections. Chandrangsul [21] systematically measured geometric initial imperfections in supported scaffolding systems across multiple construction sites to evaluate their stability impacts. Measured parameters included curvature deviations of vertical members, frame warping, and load eccentricities between upper supports and U-head screw jacks. These measurements were statistically analyzed through histograms and fitted probability distribution functions. Zhao [22] examined the buckling capacity of socket-type formwork scaffolding under stochastic bending stiffness and load-bearing conditions. By incorporating joint random initial imperfections and varying loading patterns, they investigated the stochastic distribution of stability performance in disc-lock scaffolding systems. Results demonstrated that lateral constraints at the top region act as primary supports during the initial loading stages. Mercier [23] addressed discrepancies between actual initial imperfections in cold-formed hollow tube sections and buckling coefficients specified in European steel structure design codes. To resolve this inconsistency, they proposed an innovative design methodology for beam columns without lateral torsional buckling, introducing novel buckling coefficients and interaction

equations. Zhang [24] quantitatively characterized initial imperfections in scaffolding steel tubes, establishing column curves under varying initial curvatures and residual stresses. Building upon this foundation, they derived reference values for global initial imperfections in tubes with different initial bending configurations.

Scaffolding support systems are frequently employed in the construction of large-span structures, where both joint stiffness and geometric initial imperfections significantly influence structural load-bearing capacity. This study investigates the collaborative load-bearing mechanism between roof slabs and scaffolding, using the Yongchuan Convention and Exhibition Center project as an engineering case study. Specifically, a numerical model integrating concrete floor slabs and socket-template scaffolding systems is developed to analyze their stability and load-bearing capacity.

First, a finite element model of the collaborative load-bearing system is established using ANSYS software, with validation performed on the semi-rigid connections within the numerical model. Subsequently, two types of geometric initial imperfections are considered: joint positioning errors and initial curvature of vertical members. Finally, parametric analyses are conducted by varying key construction parameters such as planar dimensions and height-to-thickness ratios of both the roof slab and scaffolding system. These investigations aim to elucidate the collaborative load-bearing behavior between the components and reveal their underlying mechanical mechanisms.

2. Establishment of finite element numerical model

2.1. Selection of numerical model elements

The stiffness of scaffolding connection joints significantly affects load-bearing capacity. Therefore, semi-rigid connections should be simulated to accurately evaluate the influence of joint stiffness [25]. The COMBIN39 element is selected to model semi-rigid connections. COMBIN39 is a unidirectional element with nonlinear generalized force-displacement capability, allowing input of customized load-displacement curves. This element is applicable to all analysis types, as shown in Fig. 3. The axial option represents a tension-compression element, with each node possessing up to three degrees of freedom: translations along the x, y, and z axes. Bending or torsional effects are not considered. The torsional option functions as a pure rotational element, also representing a tension-compression unit, with three rotational degrees of freedom per node: rotations about the x, y, and z axes. Bending or axial loading is excluded. Structural members between nodes are typically modeled using beam elements [25]. The BEAM188 element is employed to simulate vertical and horizontal scaffolding members. BEAM188 is a three-dimensional beam element suitable for analyzing slender to moderately stubby/thick beam structures, as illustrated in Fig. 4. Based on Timoshenko beam theory, this element accounts for shear deformation effects. Each node possesses six or seven degrees of freedom, including translations along the x, y, and z axes and rotations about these axes, with the seventh degree of freedom (warping magnitude) being optional. This element is particularly effective for linear analyses, large rotation, and large strain nonlinear analyses.

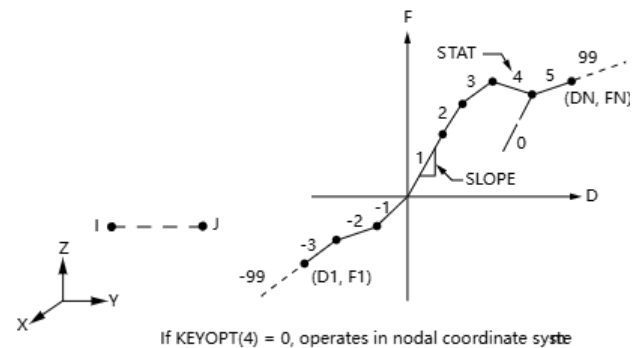
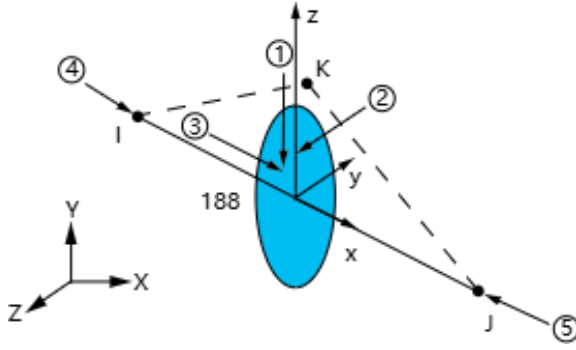
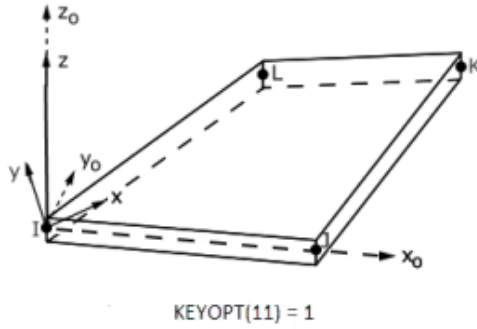
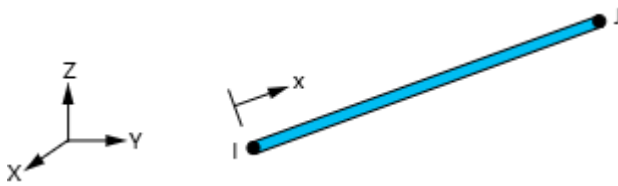


Fig. 3 COMBIN39 Geometry [26]

Fig. 4 BEAM188 Geometry^[26]Fig. 5 SHELL181 Geometry^[26]Fig. 6 LINK180 Geometry^[26]

The roof slab is modeled using SHELL181 elements. SHELL181 is a four-node element suitable for analyzing thin to moderately thick shell structures, as illustrated in Fig. 5. Each node possesses six degrees of freedom: translations along the x, y, and z axes and rotations about these axes. This element is particularly effective for linear analyses, large rotation, and large strain nonlinear applications. Nonlinear analyses account for shell thickness variation and support multi-layered material configurations.

Reinforcing steel bars within the floor slab are simulated with LINK180 elements. LINK180 is a rod element with broad engineering applications, capable of modeling trusses, cables, links, springs, and similar components, as shown in Fig. 6. Each node has three degrees of freedom: translations along the x, y, and z axes in the nodal coordinate system. Similar to pin-jointed structures, this element does not resist bending moments while incorporating capabilities for plasticity, creep, rotation, large deflection, and large strain.

2.2. Configuration of numerical model parameters

The parameters of the numerical model for the roof slab-scaffolding collaborative system were configured in compliance with industry standards for steel tubes, couplers, and components. Initial settings included a scaffolding member spacing of 1.2 m between vertical and horizontal members, with a planar configuration of 6 spans in the X-direction, 6 steps in the Y-direction, and 6 layers in the Z-direction. The vertical and horizontal members were modeled using BEAM188 elements, characterized by steel tubes with a diameter of 48 mm, wall thickness of 3.5 mm, elastic modulus $E=2.0 \times 10^{11}$ Pa, Poisson's ratio $\mu=0.3$, and density $\rho=7850$ kg/m³.

The roof slab is designed as a square with edge length $T_{size}=11.8$ m $\times$ 11.8 m and thickness $T_{size}=170$ mm. The concrete material is simulated using a multilinear isotropic hardening model (MISO) to capture its nonlinear behavior, with a compressive strength of 35 MPa, elastic modulus $E=3.4 \times 10^{10}$ Pa, Poisson's ratio $\mu=0.2$, and density $\rho=2500$ kg/m³. The reinforcement within the slab consists of steel bars with a cross-sectional area of 5.5 cm², spaced proportionally to the slab dimensions. The steel reinforcement is modeled using a bilinear isotropic hardening model (BISO), representing nonlinear elastic-plastic stress-strain behavior including plastic deformation, with elastic modulus $E=2.0 \times 10^{11}$ Pa, Poisson's ratio $\mu=0.3$, and density $\rho=7850$ kg/m³. The numerical model integrating these parameters, as illustrated in Fig. 7, ensured a realistic representation of the collaborative system's mechanical behavior while adhering to structural engineering specifications.

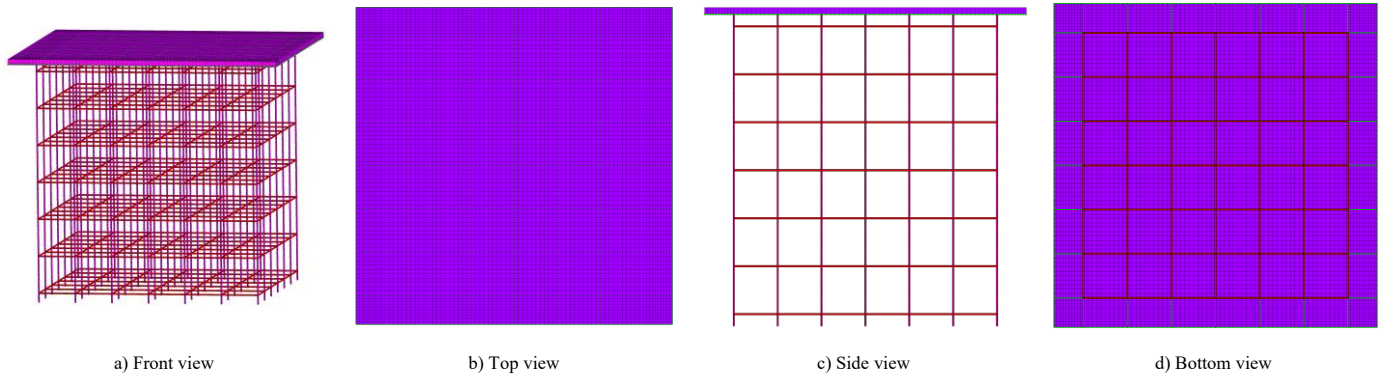


Fig. 7 Numerical model of the collaborative load-bearing system between the top plate and scaffolding

2.3. Simulation of semi-rigid connections in numerical model nodes

To validate the effectiveness of semi-rigid connections in the numerical model, comparative analyses between numerical simulations and experimental results were conducted^[27]. As the scaffolding in this study simulates socket-template scaffolding, bending performance experiments were specifically performed on socket-template connection joints. The bending tests were executed in both positive and negative directions. 'P' denotes loading in the positive direction, while 'N' represents loading in the negative direction. Vertical members were fixed to the test apparatus, and upward loads were applied to horizontal members via a hydraulic jack. Moment-rotation curves at the joints were monitored, with relative rotations during loading measured by

displacement transducers. Moments were calculated as the product of applied forces and lever arms. Negative directional loading was achieved by inverting the horizontal member orientation. Finite element analysis-derived moment-rotation curves were compared with experimental results, as shown in Fig. 8, which validated the reliability of bending behavior simulations in the numerical model. Fig. 9 presents a comparison of the failure modes derived from FEA and the experiment. As shown in the figure, the FEA model can accurately predict the failure mode of socket connections. Based on these comparative outcomes, the numerical model is confirmed to accurately replicate bending phenomena observed in physical experiments, thereby ensuring the validity and authenticity of experimental data. The comprehensive experimental protocol is delineated in^[27].

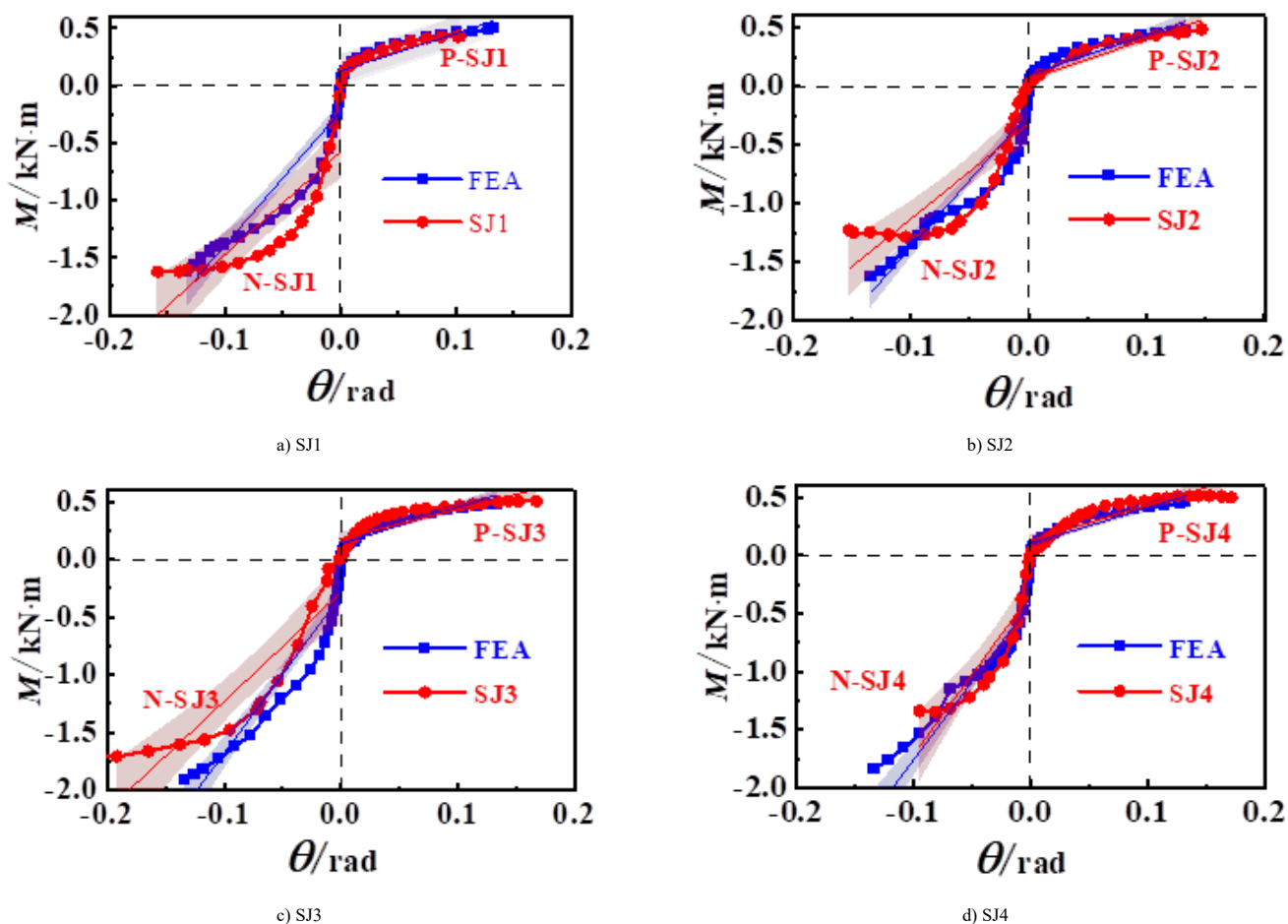


Fig. 8 Finite element simulation results and test results moment-rotation curve

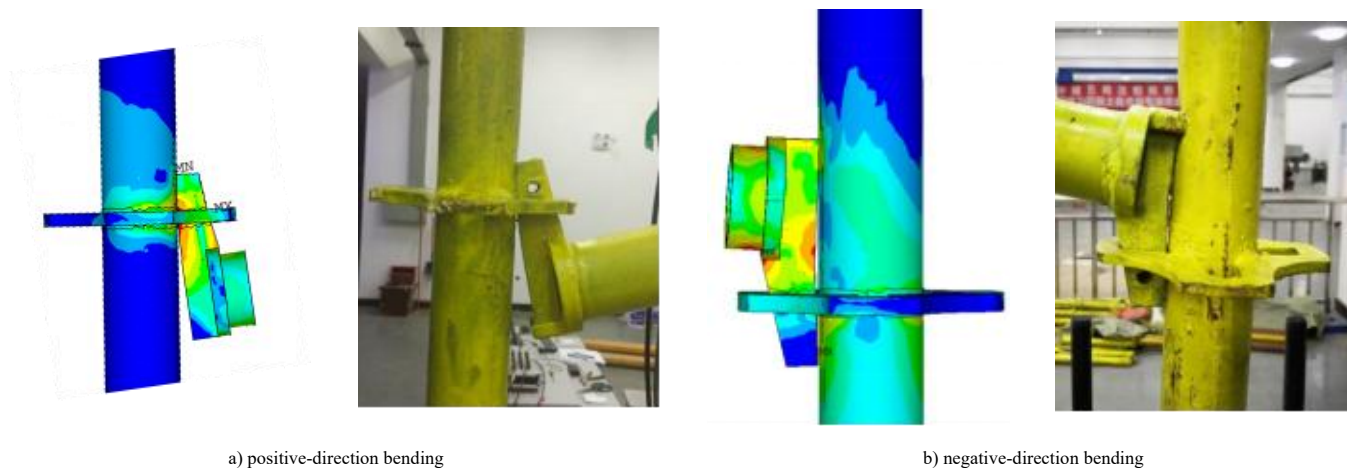


Fig. 9 Comparative analysis of failure mode

Table 1
Quantitative error analysis

Specimen	Bending Direction	RMSE (kN·m)	MAE (kN·m)	R ²
SJ1	positive	0.350	0.302	0.462
	negative	0.064	0.059	0.753
SJ2	positive	0.205	0.185	0.822
	negative	0.052	0.0453	0.914
SJ3	positive	0.227	0.173	0.817
	negative	0.079	0.069	0.681
SJ4	positive	0.245	0.184	0.720
	negative	0.097	0.081	0.604

To objectively evaluate the accuracy of the numerical model for semi-rigid joints, a quantitative error analysis was conducted on the moment-rotation curves of four specimens under both positive and negative bending directions. First, the data output from the finite element simulation were aligned to the rotation coordinate sequences of the corresponding experimental data using linear interpolation. Based on this alignment, the root means square error (RMSE), mean absolute error (MAE), and coefficient of determination (R²) were calculated, with the results summarized in Table 1.

As can be seen from Table 2, the simulation accuracy under negative bending is generally high for all specimens, with R² values ranging from 0.604 to 0.914. In particular, specimen SJ2 achieved an R² of 0.914 under negative bending, with both RMSE and MAE below 0.053 kN·m, indicating that the numerical model can reproduce the mechanical behavior of the joint in the main bearing direction (negative bending) with high precision.

Under positive bending, except for SJ1, all specimens exhibited R² values greater than 0.72, demonstrating reasonable predictive capability. The relatively

lower R^2 for SJ1 in positive bending is likely attributable to complex factors not accounted for in the model, such as slight initial gaps or asymmetric contact during positive loading. The quantitative metrics confirm that the semi-rigid joint model established in this study reliably captures the core mechanical performance of the joints, providing a sufficient basis for subsequent stability analysis of the collaborative structural system. The model does not explicitly consider complex behaviors such as hysteresis or gaps; this simplification is considered reasonable and efficient for parametric studies focused on monotonic loading and ultimate bearing capacity.

A crucial aspect in establishing the numerical model of the collaborative load-bearing system between the roof slab and the scaffolding in this study is

the accurate simulation of the mechanical behavior of the semi-rigid connections. To achieve this, the nonlinear spring element COMBIN39 was introduced to model the joints. The moment-rotation curve of specimen SJ4 was assigned as the real constants for the COMBIN39 element, aiming to validate the accuracy of the numerical model. Furthermore, the "CP" command in ANSYS was utilized to couple the translational degrees of freedom at the connections between the horizontal and vertical members, as illustrated in Fig. 10. The torsional stiffness data were derived from the finite element analysis results of the aforementioned joints, enabling the establishment of a comprehensive scaffolding model using the ANSYS finite element analysis software.

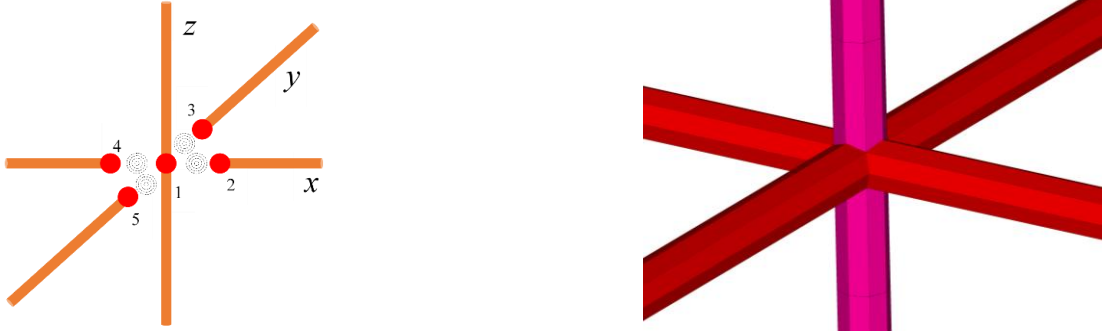


Fig. 10 Details of semi-rigid connections and physical model

2.4. Boundary conditions and connections for the roof slab in the numerical model

The roof slab was simulated using SHELL181 elements to capture its in-plane and out-of-plane mechanical behavior. After completing the scaffolding modeling in the ANSYS finite element software, four key points were established at the top height of the vertical scaffold poles. Based on these points, a square plane was generated, positioned directly above the scaffolding in the vertical direction. Subsequently, the "WPOFF" command was applied to offset the working plane, and two successive partitioning operations were performed along the width and length directions of the plane, thereby subdividing the slab plane into a regular rectangular mesh. The intersecting points of the roof slab mesh were geometrically aligned with the corresponding top nodes of the vertical scaffold poles below. To further account for the contribution of reinforcement to the bending stiffness of the concrete slab, LINK180 elements—simulating the reinforcement layout—were modeled along the partitioning lines in both the width and length directions after the generation of the concrete shell element mesh. This approach equivalently represents the bending restraint provided by the reinforcement to the roof slab, ensuring that the nodes of the LINK180 elements fully coincide with the nodes of the SHELL181 elements at the corresponding locations.

A critical aspect of the numerical model involves the connection mechanism between the scaffolding and the roof slab. To enhance collaborative load-bearing behavior and improve computational accuracy, a shared-node coupling method was implemented^[28], as illustrated in Fig. 11. During model construction, nodes of the roof slab and top nodes of the scaffolding were aligned within the same plane. The ANSYS NUMMRG command was employed to merge these nodes, forming shared nodes where at least one node is common between adjacent elements. This nodal sharing accommodates both simple connections and complex coupling relationships. When multiple elements share nodes, their displacement, stress, and strain fields remain kinematically compatible. This treatment integrates the roof slab and

scaffolding into a unified system that collectively resists external loads, thereby accurately reflecting their collaborative load-bearing interaction.

It must be acknowledged that adopting shared-node connections between the two components represents a theoretical simplification for mechanistic investigation. The primary objective is to explore the influence of random geometric initial imperfections under the idealized assumption of full compatibility. This assumption inevitably overestimates the connection stiffness between the roof slab and the scaffolding, which constitutes a limitation of this study. However, the focus of this work lies in revealing the macro-level influence patterns of geometric initial imperfections and various construction parameters on the collaborative load-bearing system. In subsequent research, more precise spring elements will be employed to simulate the interface connection. The present simplified assumption serves as a necessary foundation for establishing the initial framework of this study.

This study applied a series of boundary conditions to the established numerical model using ANSYS finite element analysis software. Boundary conditions define the behavioral constraints of a physical system at spatial boundaries during simulation. In numerical modeling, boundaries typically refer to the external regions of the computational domain, governing the evolution or stabilization of physical quantities. As shown in Fig. 12 (highlighted by red frames), displacement constraints in the x, y, and z directions were imposed on all four edges of the roof slab.

The solution process in this study employs ANSYS static analysis combined with geometric large deformation theory to investigate the nonlinear instability of the model. The loading method involves applying a surface load on top of the roof slab. To efficiently handle a large number of random samples, a load-controlled method is adopted for incremental loading. Sufficient substeps are set to ensure solution stability during the nonlinear stage. When the solver fails to meet the equilibrium convergence criteria at a certain load increment, the model is considered to have reached its ultimate state. The total reaction force from the last converged substeps is then defined as the ultimate bearing capacity of that sample.

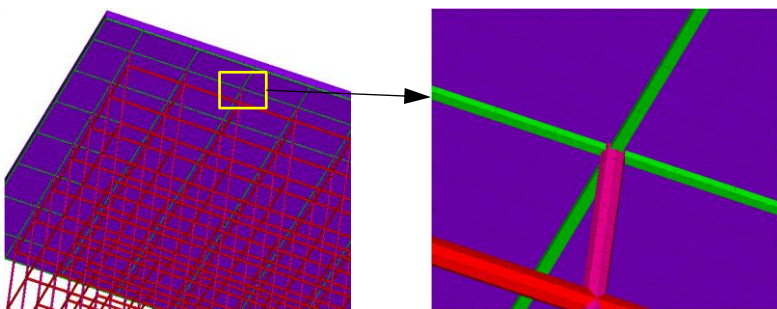


Fig. 11 Connection and details between the roof slab bottom surface and scaffolding

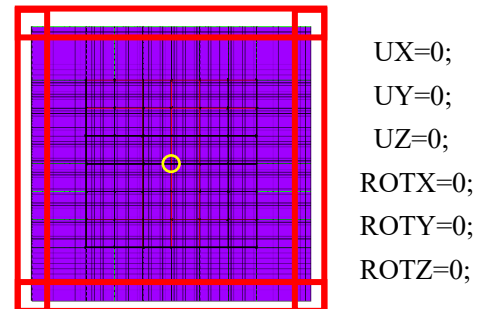


Fig. 12 Boundary conditions of the roof slab

3. Numerical model with stochastic initial imperfections

As a temporary structural system, scaffolding support systems are notably characterized by the development of geometric imperfections during repeated usage. Initial imperfections refer to various deviations existing in actual structural members prior to loading, which differ from idealized computational models. These imperfections may originate from fabrication processes, including member initial eccentricity and local imperfections.

This study considers two types of geometric initial imperfections: joint positioning errors and initial curvature of vertical members. The stochastic nature of initial imperfections significantly influences the load-bearing capacity of scaffolding systems. A numerical model incorporating stochastic geometric imperfections has been developed to comprehensively investigate their impacts on structural load-bearing capacity.

3.1. Joint positioning errors

Joint positioning errors refer to the maximum positional deviation of scaffolding nodes from their idealized locations due to inaccuracies during construction processes, such as operational practices or technical limitations. These deviations alter the stress distribution within the scaffolding system, potentially compromising the connection strength between adjacent members and reducing structural integrity. For analytical quantification, a vertical member was selected, with a top-view schematic of one node illustrated in Fig. 13. The actual node position (red) deviates radially from the ideal position (center of the dashed circle) by a distance Φ . The horizontal deviation Δx is

randomly generated within the range $[-\Phi_{\max}, \Phi_{\max}]$ using a uniform distribution, while the vertical deviation Δy is calculated as $\Delta y = \sqrt{(\Phi^2 - \Delta x^2)}$. This defines the actual node coordinates as $(x_0 + \Delta x, y_0 + \Delta y)$.

Cases a) and b) in Fig. 13 differ in their Φ definitions, Case a): All Φ values are fixed at $\Phi_{\max}$, representing maximum permissible deviations. Case b): Φ values are stochastically determined by Δx and Δy , simulating real-world random error distributions. The maximum allowable positioning error $\Phi_{\max}$ is defined as $H/1000$, where H denotes the overall height of the scaffolding system. The selection of $\Phi_{\max}$ is based on the stipulation in Clause 7.4.10 of the Chinese industry standard Technical Standard for Safety of Disk-Buckle Steel Tubular Scaffolding in Construction (JGJ/T 231-2021), which states that after the erection of the scaffolding, the vertical deviation of the uprights should not exceed $1/500$ of the support height. Therefore, $\Phi_{\max}$ in this study is defined as $H/1000$. This stochastic modeling approach captures the inherent randomness of construction imperfections, enabling a realistic assessment of their structural consequences.

To better approximate real-world conditions, this study employs random Φ values to establish the numerical model, which accounts for the stochasticity in both the direction and magnitude of joint positioning errors. The "EGEN" command was utilized to regenerate new nodes for connections. Fig. 14 illustrates the numerical model with joint positioning errors, demonstrating that vertical members no longer maintain perfectly straight configurations. In this model, node positions are constrained within a circle of radius $\Phi_{\max}$. Notably, joint positioning errors exhibit no significant effect on joint stiffness, as the vertical member segments between two adjacent connection nodes remain linear.

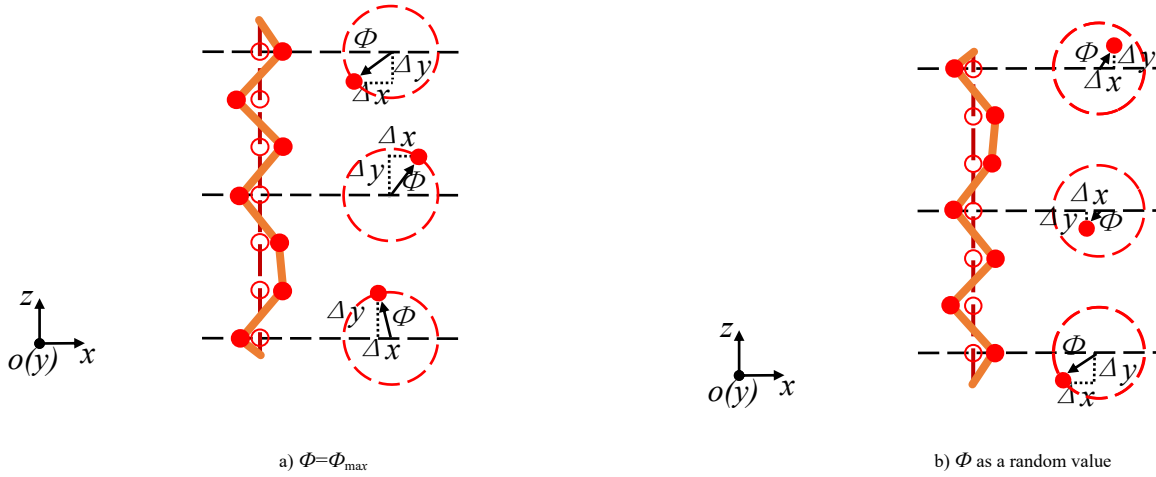


Fig. 13 Schematic of joint positioning errors

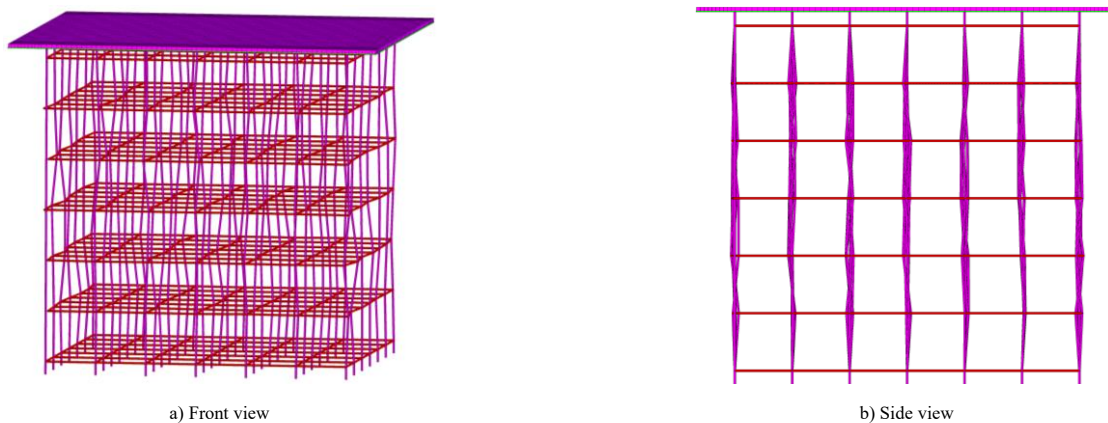


Fig. 14 Numerical model with joint positioning errors

3.2. Initial curvature of vertical members

In addition to joint positioning errors, another type of geometric initial imperfection is the initial curvature of vertical members. This curvature may arise from manufacturing defects, transportation-induced deformations, or construction practices, resulting in non-ideal linearity of structural members. Conventional load-bearing capacity analyses typically model vertical members

as perfectly straight, neglecting such initial imperfections. This study assumes the initial curvature follows a sinusoidal profile, as shown in Fig. 15, with maximum deflection v_{mo} occurring at the mid-height of the vertical member. To systematically account for bending direction, a local coordinate system^[29] is established: the x-axis is aligned with the longitudinal axis of the vertical member, and the y-axis is perpendicular to the member axis (horizontal direction). This coordinate system configuration is illustrated in Fig. 16.

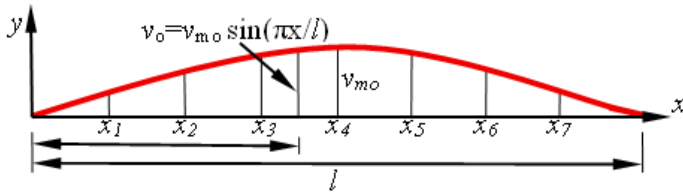


Fig. 15 Sinusoidal initial curvature

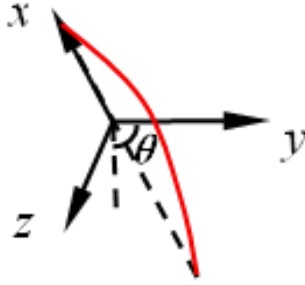


Fig. 16 Local coordinate system

The maximum bending amplitude is denoted as $v_{mo, \max}$, and defined as $l/1000$, where l represents the geometric length between two connection nodes of the vertical member. The selection of this value references Literature [21], which is based on a statistical study of measured initial geometric imperfections in scaffold uprights at construction sites. After considering various uncertainties, $l/1000$ is deemed a conservative and practically acceptable value in engineering, providing a robust data basis for the random parameter. As shown in Fig. 17, v_{mo} can be set as a constant value $v_{mo, \max}$, or as a random value uniformly distributed between 0 and $v_{mo, \max}$. To thoroughly investigate the influence of initial curvature on the load-bearing capacity distribution of the roof slab-scaffolding system, this study adopts stochastic v_{mo} values. Furthermore, a random angle θ uniformly distributed within $0-2\pi$ radians is generated to rotate the local coordinate system about the x -axis. This methodology comprehensively accounts for the dual stochasticity of initial curvature, encompassing both random bending amplitudes and random bending directions.

In summary, the vertical members of the scaffolding are modeled by replacing straight lines with curved splines generated via the BSPLIN command, using the bending amplitude at each layer's midpoint as input parameters. This approach establishes a numerical model with stochastic initial curvature, as shown in Fig. 18. Both the stochastic direction and magnitude of initial curvature are simultaneously incorporated. Notably, while adjacent member segments exhibit curvature, the node positions remain consistent with their idealized locations. It is worth emphasizing that the looping command functionality in ANSYS finite element analysis software enables the autonomous creation of vertical members with initial curvature characteristics [30]. This methodology not only streamlines the modeling process but also enhances operational efficiency, facilitating its adoption by researchers and engineers.

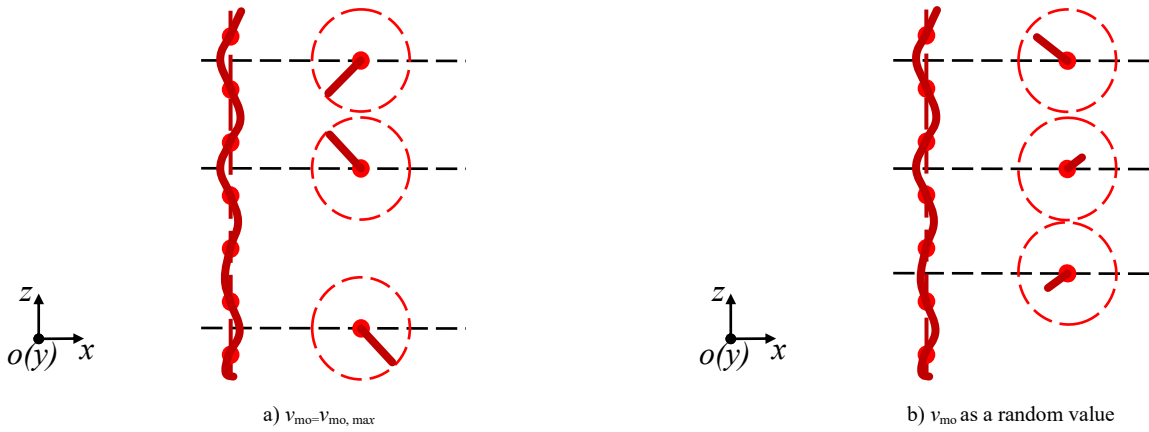


Fig. 17 Schematic of initial curvature in vertical members

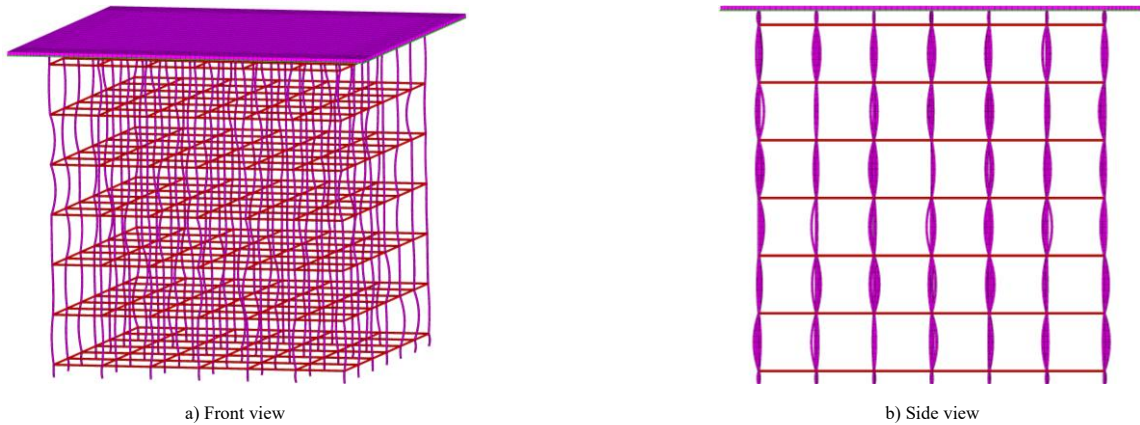


Fig. 18 Numerical model with random initial curvature

4. Influence of joint positioning errors on scaffolding stability load-bearing capacity

This section investigates the effects of joint positioning errors on the stability load-bearing capacity of the roof slab-scaffolding collaborative system. Using ANSYS finite element software, numerical models incorporating joint positioning errors were established and subjected to buckling analysis. A

surface load was applied to the upper surface of the roof slab to determine the system's load-bearing capacity. The maximum joint positioning error $\Phi_{\max}$ was set to $H/1000$ and $H/100$, with 500 iterations computed to enhance the stochastic representation of initial imperfections. Here, $\Phi_{\max}$ is magnified by a factor of ten to evaluate the performance of the collaborative load-bearing system under defect conditions that exceed the code-permissible limits. This approach employs a more adverse defect amplitude for sensitivity analysis,

thereby facilitating an examination of the system's safety reserve. To further explore the impact of joint positioning errors, parametric analyses were conducted by adjusting the planar dimensions and height of the scaffolding. Comparative studies of load-bearing capacity variations under different configurations reveal the sensitivity of structural performance to positioning errors, thereby informing anti-defect design optimization. The load-bearing

capacity characteristics of scaffolding with joint positioning errors are illustrated in Fig. 19. Table 2 summarizes the nonlinear buckling analyses conducted on the roof slab-scaffolding collaborative load-bearing system considering joint positioning errors, discussing the influence of erection parameters on the bearing capacity. For all working conditions, 500 random simulations were performed.

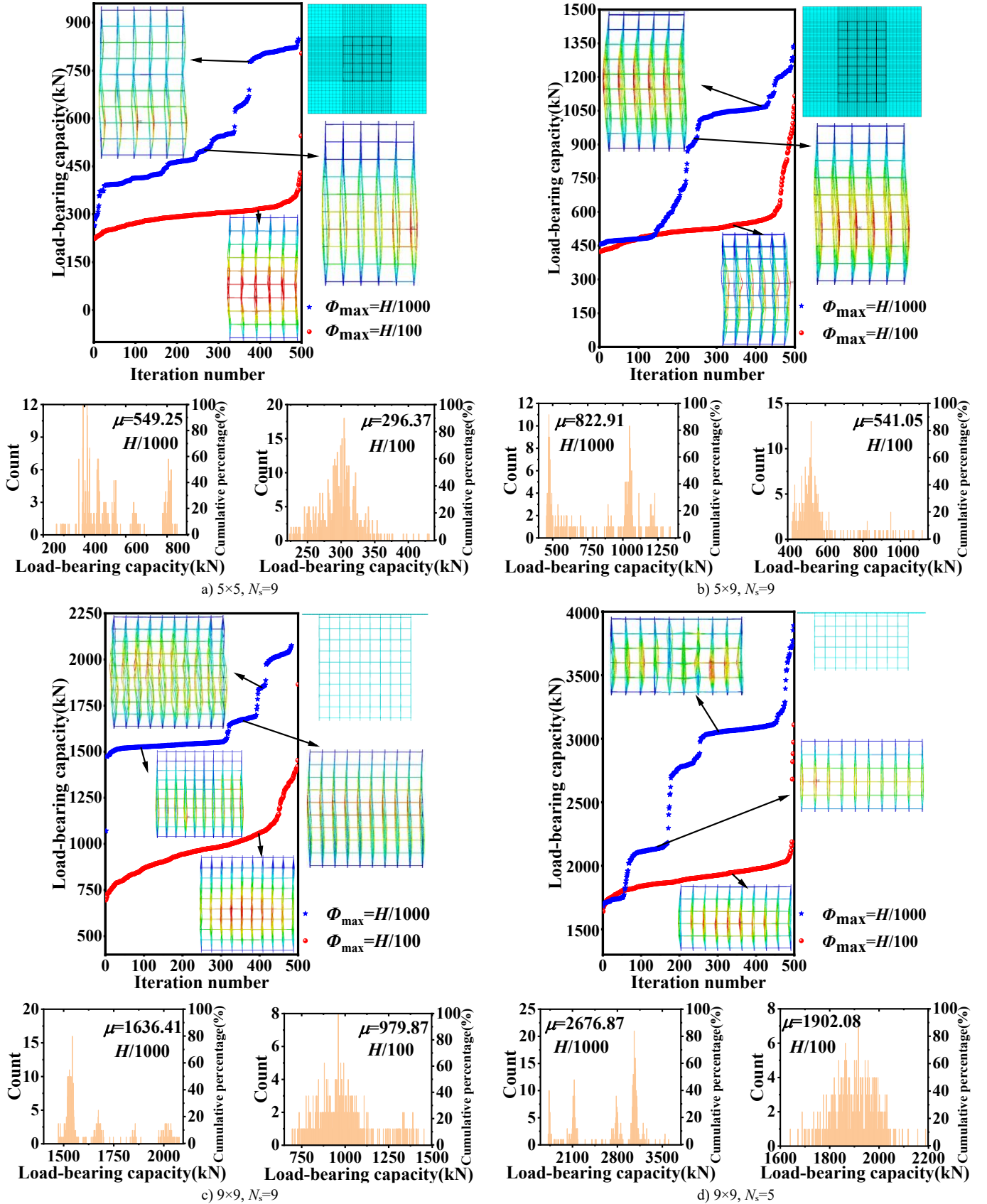


Fig. 19 Joint positioning errors on the stochastic distribution of load-bearing capacity

Table 2

Numerical parameter matrix under the condition of joint positioning errors in the vertical members

Research Section	Variable Type	Variable Parameter Values	Fixed Parameter Values	Initial Imperfection Amplitude	Objective
4.1	Scaffolding Plan Dimensions	5×5, 5×9, 9×9,	$N_s=9$	$\Phi_{\max}=H/1000$ $\Phi_{\max}=H/100$	Comparison of the Influence of Plan Dimensions
4.2	Scaffolding Number of Stories	$N_s=9, N_s=5$	9×9	$\Phi_{\max}=H/1000$ $\Phi_{\max}=H/100$	Comparison of the Influence of Height

4.1. Influence of different plan dimensions of scaffolds on the distribution of bearing capacity

This study investigates the effects of scaffolding planar dimensions on load-bearing capacity distribution under varying geometric imperfections. The roof slab was configured with dimensions $T_{\text{size}}=14.8\text{m}\times 14.8\text{m}$ and thickness $Th_{\text{size}}=90\text{mm}$, while the scaffolding height was fixed at $N_s=9$ layers. Three planar configurations (5×5, 5×9, and 9×9) were analyzed to evaluate structural performance. Finite element results demonstrate that enlarging planar dimensions consistently enhances load-bearing capacity, regardless of joint positioning error magnitude.

Under maximum joint positioning error from $\Phi_{\max}=H/1000$ to $H/100$, significant capacity reductions were observed. For the 5×5 configuration, the dominant buckling mode capacity decreased from 412.5 kN to 304.5 kN, representing a 26.2% reduction (108 kN loss). In contrast, the 9×9 configuration exhibited a more pronounced decline, with capacity dropping from 1543.5 kN to 960.5 kN (583 kN loss, 37.8% reduction). This trend highlights that larger planar dimensions amplify the detrimental effects of joint positioning errors, as node deviations propagate stress redistributions across broader regions, exacerbating instability in adjacent members and reducing overall system integrity.

Notably, unidirectional expansion (e.g., 5×5 to 5×9) also improves load-bearing performance, though asymmetric configurations introduce new vulnerabilities. Under $\Phi_{\max}=H/100$, the 5×5 system displays uniformly distributed maximum displacements, while the 5×9 system concentrates displacement at the mid-span of the long side (50% span), revealing heightened sensitivity in weak-axis regions. This phenomenon underscores the necessity of reinforcing mid-span flexural resistance in non-symmetric designs to mitigate localized buckling risks.

Mechanistically, while larger planar dimensions enhance structural redundancy, they simultaneously broaden error propagation pathways, intensifying sensitivity to geometric imperfections. For instance, expanding from 5×5 to 9×9 increases load-bearing capacity by 164% under ideal conditions but amplifies curvature-induced instability under defects. These findings emphasize the critical balance between dimensional scaling and precision control in scaffolding design, particularly for systems exceeding 6×6 configurations, where error mitigation strategies must prioritize both global stability and localized reinforcement.

4.2. Influence of scaffolding height on load-bearing capacity distribution

This study examines the impact of scaffolding height on load-bearing capacity distribution under stochastic joint positioning errors. The roof slab's planar dimensions and thickness remained constant, while the scaffolding planar configuration was fixed at 9×9 with two height configurations: 5-layer and 9-layer systems. Generally, taller scaffolding systems exhibit reduced

stability due to amplified bending moments and lateral displacements at elevated heights, a trend exacerbated by joint positioning errors.

Under severe joint positioning errors ($\Phi_{\max}=H/100$), the 5-layer system demonstrated a load-bearing capacity increase from 960.5 kN to 1916.5 kN, a 99.53% enhancement, compared to the 9-layer system. The taller 9-layer configuration predominantly exhibited global buckling, characterized by bending-like deformation along the height due to load eccentricity induced by joint errors, resulting in synchronized buckling of multiple members and extensive instability zones. In contrast, the 5-layer system displayed local buckling, where reduced height enhanced member stiffness, concentrating error effects on specific regions. Stress accumulation in critical vertical members led to localized mid-span bending in the shorter system.

For moderate joint positioning errors ($\Phi_{\max}=H/1000$), multi-modal buckling was observed. The dominant buckling mode capacity increased from 1543.5 kN (9-layer) to 3062.5 kN (5-layer), representing a 98.41% improvement. In the 9-layer system, maximum displacements were concentrated in the upper-middle regions (60–80% height), while the 5-layer system exhibited minimal localized buckling near the base (0–30% height). These results highlight the disproportionate impact of joint positioning errors on taller scaffolding systems, emphasizing the critical role of height-dependent design in balancing load redistribution and defect sensitivity. Taller systems require stringent error controls to mitigate global instability, whereas shorter systems benefit from localized stiffness but demand targeted reinforcement at stress-concentration zones.

5. Influence of initial curvature of vertical members on scaffolding stability load-bearing capacity

This section investigates the effects of initial curvature in vertical members on the stability load-bearing capacity of the roof slab-scaffolding collaborative system. Buckling analyses were conducted using ANSYS finite element software on models incorporating the initial curvature of vertical members. A surface load was applied to the upper surface of the roof slab to determine the system's load-bearing capacity. Given the stochastic distribution of initial curvature amplitudes (v_{mo} , max) in practical scenarios, two cases were analyzed: $v_{\text{mo}, \text{max}}=l/1000$ and $v_{\text{mo}, \text{max}}=50l/1000$, where l represents the geometric length between connection nodes. To comprehensively investigate the influence mechanisms of initial curvature, parametric analyses were performed by varying key construction parameters, including scaffolding planar dimensions, height, and roof slab dimensions and thickness. Each numerical model underwent 500 computational iterations to ensure statistical robustness and enhance the reliability of the findings. To investigate the influence of various parameters on the bearing capacity under the condition of initial curvature in the vertical members through nonlinear buckling analysis, a numerical experiment matrix was designed as shown in Table 3. For all working conditions, 500 random simulations were performed.

Table 3

Numerical parameter matrix under the condition of initial curvature in the vertical members

Research Section	Variable Type	Variable Parameter Values	Fixed Parameter Values	Initial Imperfection Amplitude	Objective
5.1	Scaffolding Plan Dimensions	4×4, 6×6, 9×9	$N_s=9$ $T_{\text{size}}=14.8\text{m}\times 14.8\text{m}$ $Th_{\text{size}}=90\text{mm}$	$v_{\text{mo}, \text{max}}=l/1000, v_{\text{mo}, \text{max}}=50l/1000$	Comparison of the Influence of Plan Dimensions
5.2	Scaffolding Number of Stories	$N_s=3, N_s=6, N_s=9$	6×6 $T_{\text{size}}=14.8\text{m}\times 14.8\text{m}$ $Th_{\text{size}}=90\text{mm}$	$v_{\text{mo}, \text{max}}=l/1000, v_{\text{mo}, \text{max}}=50l/1000$	Comparison of the Influence of Height
5.3	Roof Slab Plan Dimensions	$T_{\text{size}}=8.8\text{m}\times 8.8\text{m}$ $T_{\text{size}}=11.8\text{m}\times 11.8\text{m}$ $T_{\text{size}}=14.8\text{m}\times 14.8\text{m}$	6×6 $N_s=6$ $Th_{\text{size}}=90\text{mm}$	$v_{\text{mo}, \text{max}}=l/1000, v_{\text{mo}, \text{max}}=50l/1000$	Comparison of the Influence of Plan Dimensions
5.4	Roof Slab Thickness	$Th_{\text{size}}=90\text{mm}$ $Th_{\text{size}}=170\text{mm}$ $Th_{\text{size}}=250\text{mm}$	6×6 $N_s=6$ $T_{\text{size}}=11.8\text{m}\times 11.8\text{m}$	$v_{\text{mo}, \text{max}}=l/1000, v_{\text{mo}, \text{max}}=50l/1000$	Roof Slab Thickness

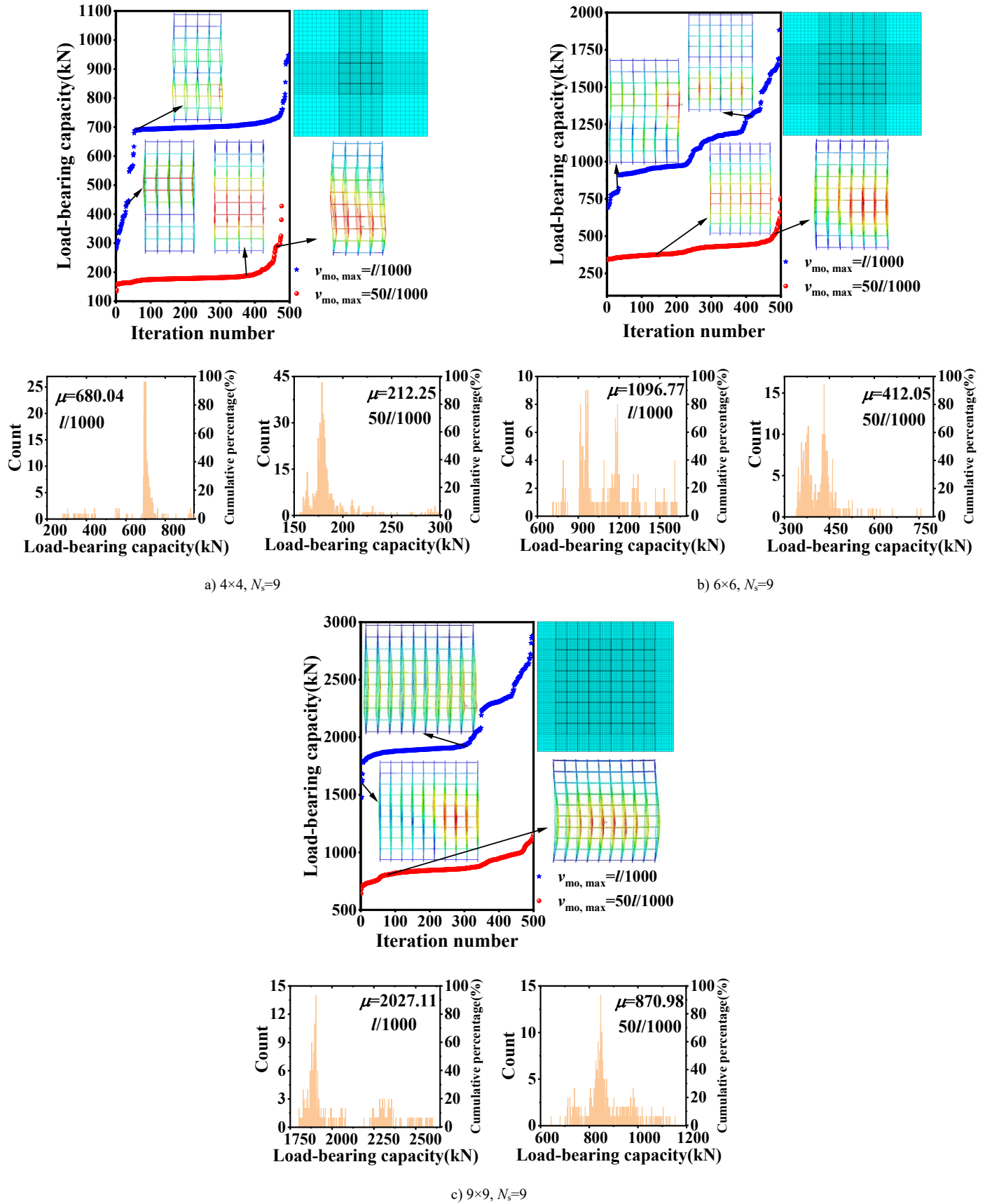


Fig. 20 Stochastic distribution of load-bearing capacity under different planar dimensions of scaffolding

5.1. Influence of scaffolding planar dimensions on load-bearing capacity distribution

This study investigates the effects of initial curvature in vertical members on the stability and load-bearing capacity of the roof slab-scaffolding collaborative system under varying planar dimensions. Three configurations (4×4 , 6×6 , and 9×9) were analyzed with a fixed scaffolding height of $N_s=9$ layers, roof slab dimensions of $T_{psize}=14.8\text{m} \times 14.8\text{m}$, and thickness

$T_{hsize}=90\text{mm}$. Buckling analyses revealed stochastic load-bearing capacity distributions, as illustrated in Fig. 20.

Comparative analysis of the 4×4 and 9×9 configurations demonstrates that larger planar dimensions significantly enhance load-bearing capacity. For the 4×4 system (Fig. 20a), the load-bearing capacity under $v_{mo, \max}=l/1000$ ranges between 300 kN and 1000 kN, while increasing the initial curvature to $v_{mo, \max}=50l/1000$ reduces this range to 100–300 kN. This indicates that greater initial curvature concentrates load on highly bent members, inducing localized

failure and diminishing global capacity. Notably, the load-bearing capacity curves $v_{mo, max} = l/1000$ exhibit discontinuous segments with gaps (Fig. 20 a–c), suggesting the coexistence of multiple distinct buckling modes. Since the initial imperfections are randomly distributed, these discontinuities are closely related to the buckling modes, with each buckling mode corresponding to a distinct dominant frequency.

Further quantitative analysis shows that the load-bearing capacity distribution ranges expand with planar dimensions: 667.91 kN for 4×4 , 1003.54 kN for 6×6 , and 1405.83 kN for 9×9 . This trend implies that larger systems

exhibit reduced sensitivity to initial curvature, as enhanced structural redundancy allows broader load redistribution. Mechanistically, initial curvature amplifies stress concentrations in critical members, accelerating localized buckling, while larger planar dimensions (9×9) mitigate these effects through distributed force transfer. The observed mode transitions and distribution broadening underscore the importance of system-scale design in counteracting curvature-induced instability, particularly for small-scale configurations requiring stringent curvature control ($v_{mo, max} \leq l/1000$).

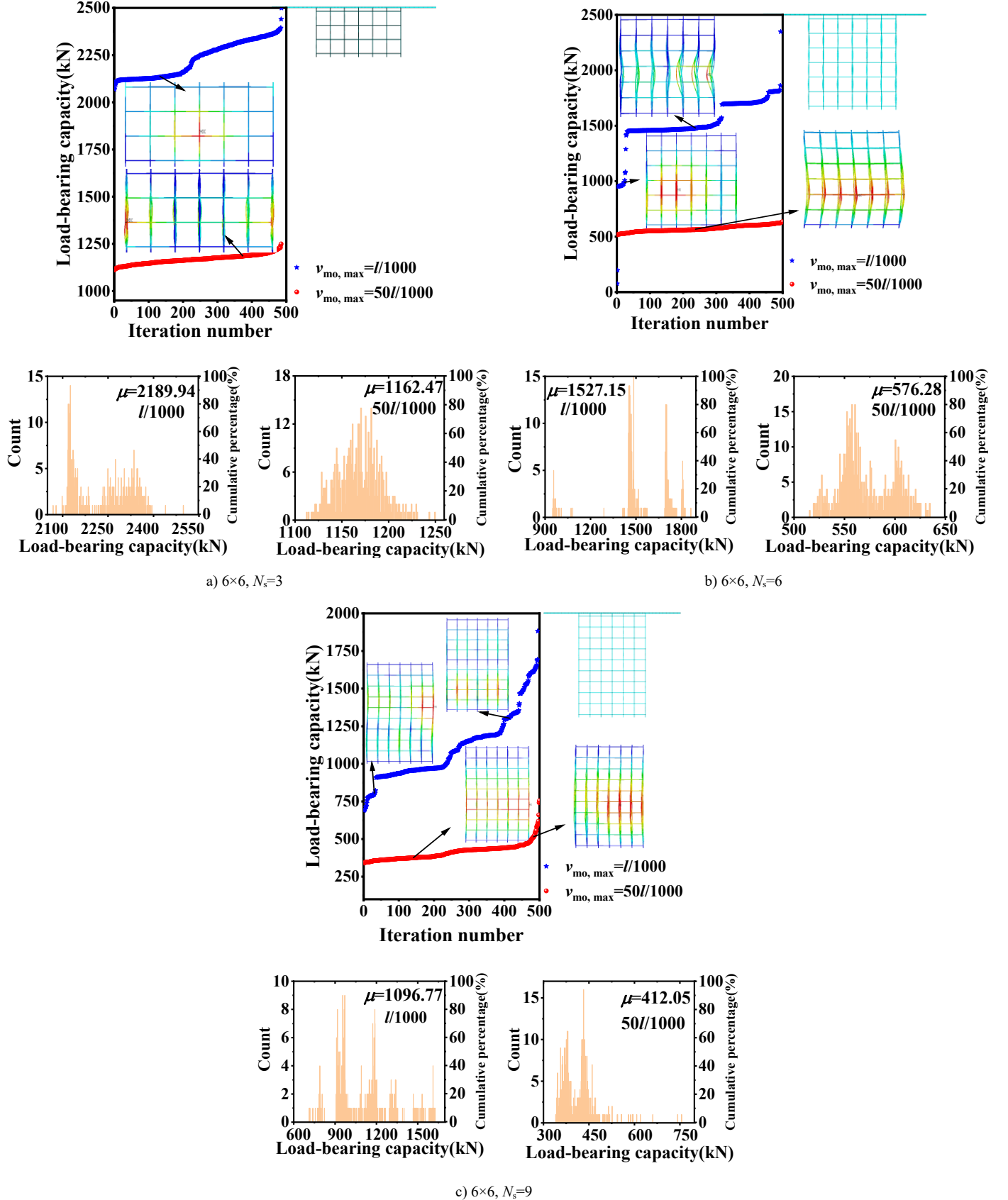


Fig. 21 Stochastic distribution of load-bearing capacity under different scaffolding heights

5.2. Influence of scaffolding height on load-bearing capacity distribution

This study examines the effects of vertical member initial curvature on the load-bearing capacity distribution of the roof slab-scaffolding collaborative system under varying heights. The scaffolding planar dimensions were fixed at 6×6 , with three height configurations defined by the number of layers: $N_s = 3, 6$, and 9 . The roof slab dimensions ($T_{size} = 14.8\text{m} \times 14.8\text{m}$) and thickness ($T_{size} = 90\text{mm}$) remained constant. Load-bearing capacity distributions derived from buckling analyses are illustrated in Fig. 21.

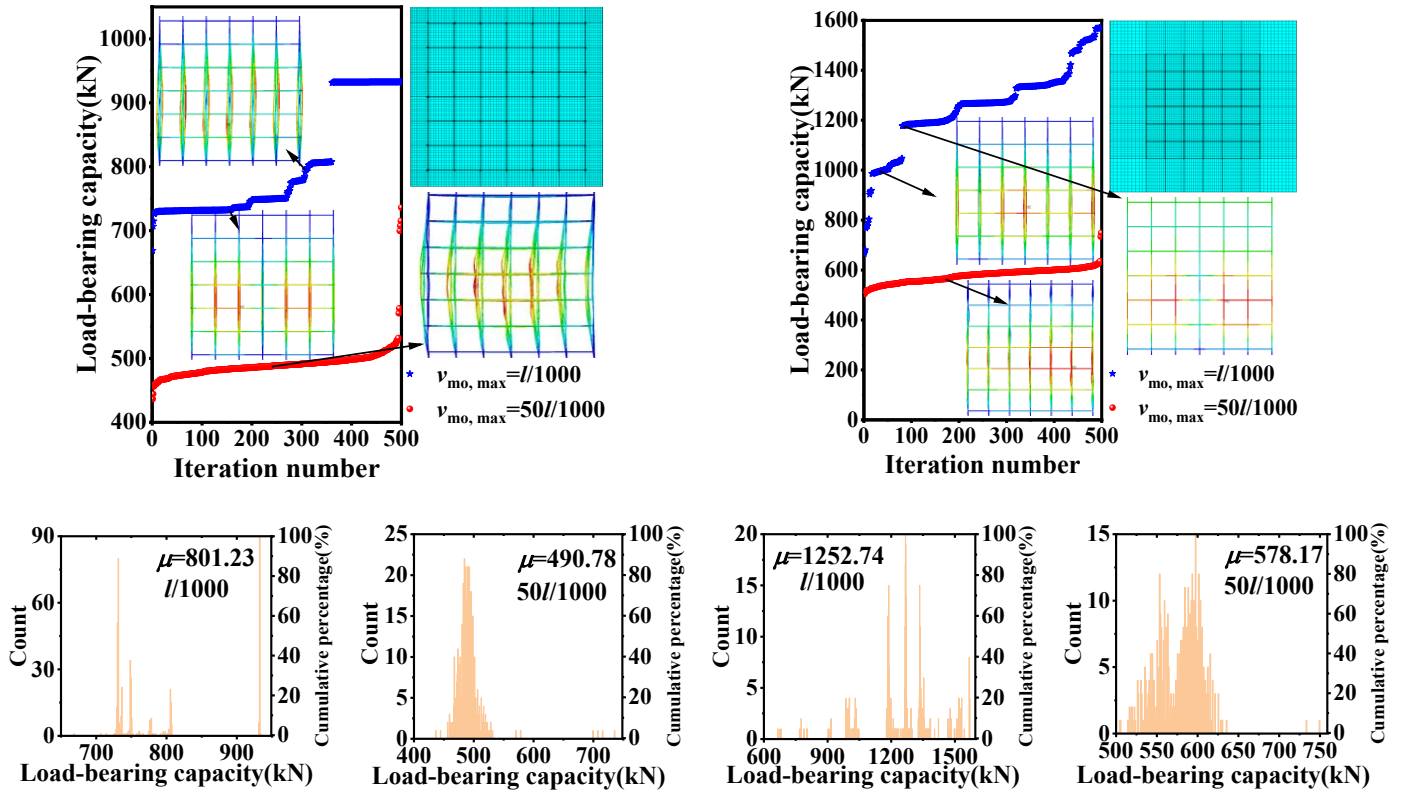
Comparative analysis of 3-layer and 9-layer systems reveals a clear inverse relationship between scaffolding height and stability load-bearing capacity. Under severe initial curvature ($v_{mo, \max} = 50/1000$), all configurations exhibit single-curve capacity distributions, indicating mono-modal buckling behavior. For the 3-layer system, buckling predominantly occurs locally, driven by stress concentration at highly curved members due to stronger nodal constraints that direct loads through limited load paths. In contrast, the 9-layer system undergoes global buckling, as loads redistribute across numerous members, leading to system-wide instability.

Under moderate initial curvature ($v_{mo, \max} = l/1000$), multi-modal buckling emerges, evidenced by discontinuous capacity curves. Each segment corresponds to a distinct buckling mode, with peak regions in normal

distribution curves representing dominant failure modes. The 3-layer system demonstrates higher tolerance to initial curvature, with localized buckling confining damage to specific regions, whereas the 9-layer system's broader load redistribution exacerbates sensitivity to geometric imperfections. Mechanistically, shorter systems leverage localized stiffness to mitigate curvature effects, while taller systems amplify imperfections through cumulative load-path deviations. These findings underscore the critical role of height-dependent design in balancing structural efficiency and defect resilience, particularly for tall scaffolding requiring enhanced global stability measures.

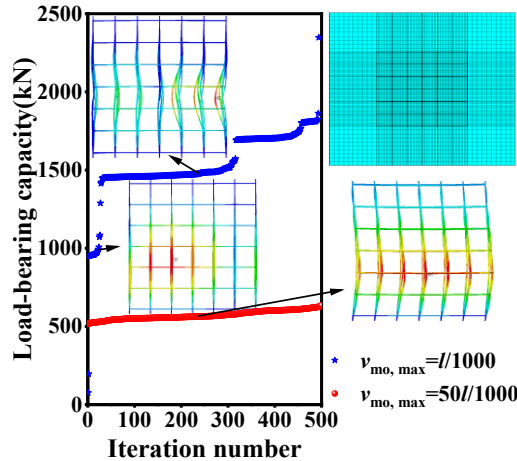
5.3. Influence of roof slab planar dimensions on load-bearing capacity distribution

Building upon previous investigations into scaffolding parameters, this section evaluates the effects of vertical member initial curvature on load-bearing capacity distribution under varying roof slab planar dimensions. Three slab sizes were analyzed: $T_{size} = 8.8\text{m} \times 8.8\text{m}$, $T_{size} = 11.8\text{m} \times 11.8\text{m}$, and $T_{size} = 14.8\text{m} \times 14.8\text{m}$, with a constant thickness $T_{size} = 90\text{mm}$. The scaffolding configuration was fixed at $6 \times 6 \times 6$ (planar dimensions $\times$ layers). Load-bearing capacity distributions under these parametric variations are illustrated in Fig. 22.



a) $T_{size} = 8.8\text{m} \times 8.8\text{m}$

b) $T_{size} = 11.8\text{m} \times 11.8\text{m}$



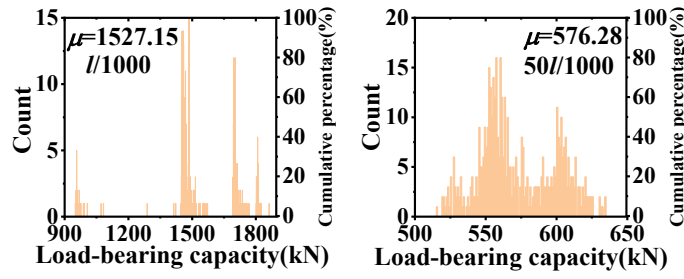


Fig. 22 Stochastic distribution of load-bearing capacity under different roof slab planar dimensions

For the 8.8m×8.8m slab (Fig. 22 a), the dominant buckling mode under v_{m0} , $\max=l/1000$ exhibits a load-bearing capacity of 801.23 kN, which decreases by 38.75% to 490.78 kN when initial curvature increases to 50l/1000. This instability manifests as localized buckling of mid-height vertical members, reflecting concentrated load transfer through limited load paths. In contrast, the 11.8m×11.8m slab (Fig. 22 b) shows capacity reductions of 53.84% under the same curvature increase, with dominant buckling occurring at corner members due to asymmetric load redistribution. The 14.8m×14.8m slab (Fig. 22 c) experiences a 62.84% capacity reduction, characterized by global overturning instability as edge members overload, compromising system-wide stability.

Mechanistically, smaller slabs (e.g., 8.8m×8.8m) exhibit heightened sensitivity to initial curvature due to constrained load distribution, where localized buckling dominates. Intermediate slabs (11.8m×11.8m) demonstrate improved defect tolerance through multi-member load sharing, though corner regions remain vulnerable. Larger slabs (14.8 m × 14.8 m) suffer systemic instability from edge member overloading, where initial curvature amplifies stress imbalances across expansive geometries. These findings highlight a critical trade-off: while larger slabs enhance load distribution breadth, they paradoxically increase susceptibility to global failure modes under geometric imperfections, necessitating targeted reinforcement strategies for edge zones in large-span systems.

To further investigate the effects of roof slab planar dimensions on the stability of the collaborative system, a load-displacement analysis was conducted on a critical node located at the center of the roof slab (marked by a yellow circle in Fig. 11). The resulting load-displacement curves are shown in

Fig. 23.

For small initial curvature (v_{m0} , $\max=l/1000$), the curves exhibit linear elastic behavior, with initial stiffness values of 0.372×10^3 N/m (8.8 m slab), 0.691×10^3 N/m (11.8 m slab), and 1.025×10^3 N/m (14.8 m slab). This uniformity in stiffness arises from synchronized load transfer among vertical members and effective collaborative interaction between the roof slab and scaffolding. Under large initial curvature (v_{m0} , $\max=50l/1000$), nonlinear stiffness degradation occurs. For the 8.8 m slab (Fig. 23 a), the initial stiffness ranges between 0.113×10^3 N/m and 0.164×10^3 N/m, while similar trends are observed for the 11.8 m and 14.8 m slabs. This variability reflects disrupted load transfer caused by stress localization in bent members, reducing system stiffness proportionally to curvature magnitude.

All curves demonstrate progressive displacement under increasing load, reaching peak points that correspond to buckling modes. Multiple peaks indicate multi-modal instabilities, with peak load magnitudes inversely correlated to initial curvature. For example, the 8.8 m slab shows a 38.7% reduction in peak load when curvature increases from $l/1000$ to $50l/1000$. The 14.8 m slab exhibits a 62.8% reduction, highlighting heightened sensitivity to imperfections in larger (14.8m) systems. Post-peak behavior stabilizes to horizontal plateaus, indicating residual load-bearing capacity after instability. These results quantify the dual role of slab dimensions: smaller (8.8m) systems prioritize localized stiffness but are vulnerable to curvature-induced stress concentrations, whereas larger (14.8m) systems enhance load distribution at the cost of global stability under geometric imperfections.

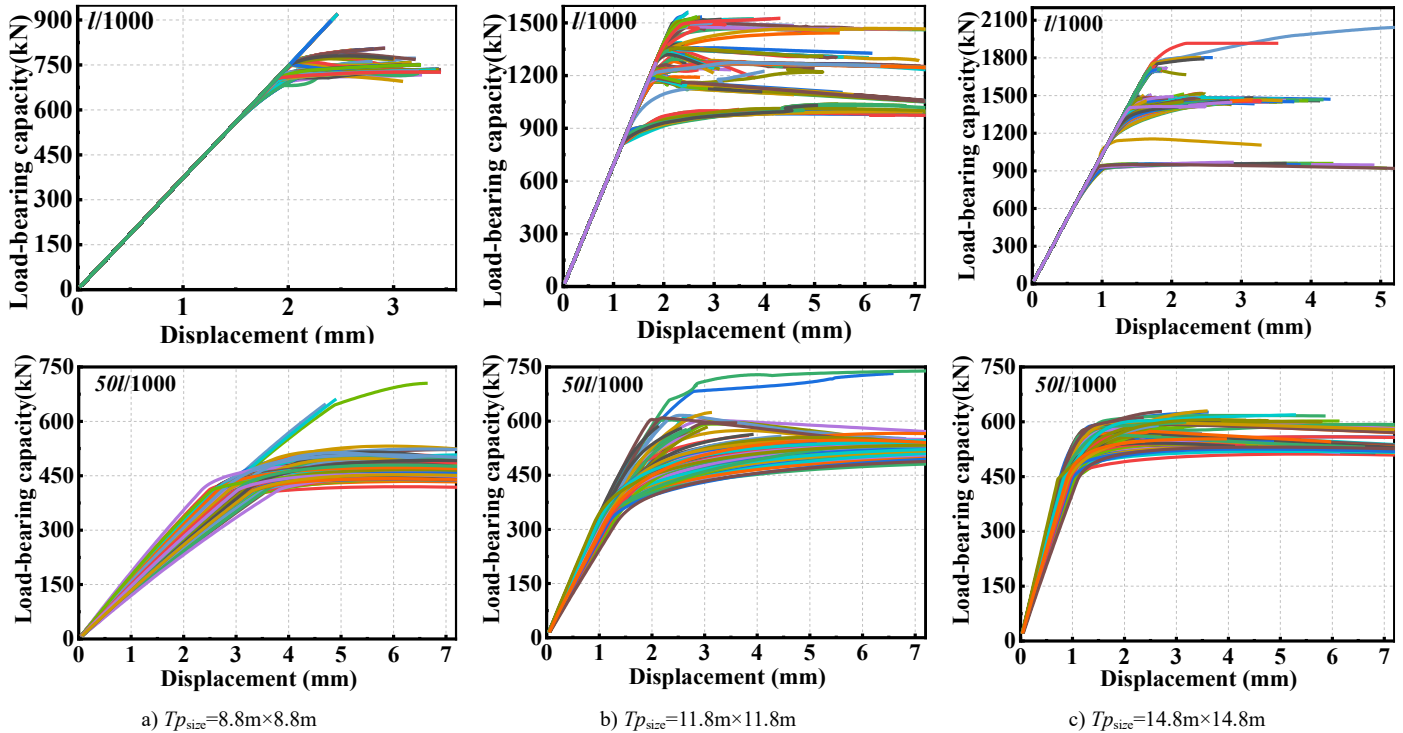


Fig. 23 Load-displacement curves

5.4. Influence of roof slab thickness on load-bearing capacity distribution

This section examines the impact of vertical member initial curvature on

the load-bearing capacity distribution of the roof slab-scaffolding collaborative system under varying slab thicknesses. The scaffolding configuration was maintained at 6×6 (planar dimensions × layers), with a constant roof slab

planar size of $T_{size}=11.8m \times 11.8m$. Three slab thicknesses ($Th_{size}=90mm$, $170mm$, and $250mm$) were analyzed to evaluate their adaptive capacity to geometric imperfections. Load-bearing capacity distributions derived from buckling analyses are illustrated in Fig. 24.

Under moderate initial curvature (v_{mo} , $max=l/1000$), all thickness configurations exhibit multi-modal buckling behavior, with dominant mode capacities varying within a narrow range of 25%, indicating comparable stability across thickness variations. However, under severe curvature (v_{mo} ,

$max=50l/1000$), the load-bearing capacities of all models converge to approximately 550 kN, regardless of slab thickness. This uniformity arises because large initial curvature critically reduces the axial stiffness of vertical members, shifting failure mechanisms to scaffolding-dominated buckling in the lower half of the structure. Notably, even significant slab thickening (e.g., from 90 mm to 250 mm) fails to mitigate this instability, as geometric imperfections in scaffolding members govern system collapse.

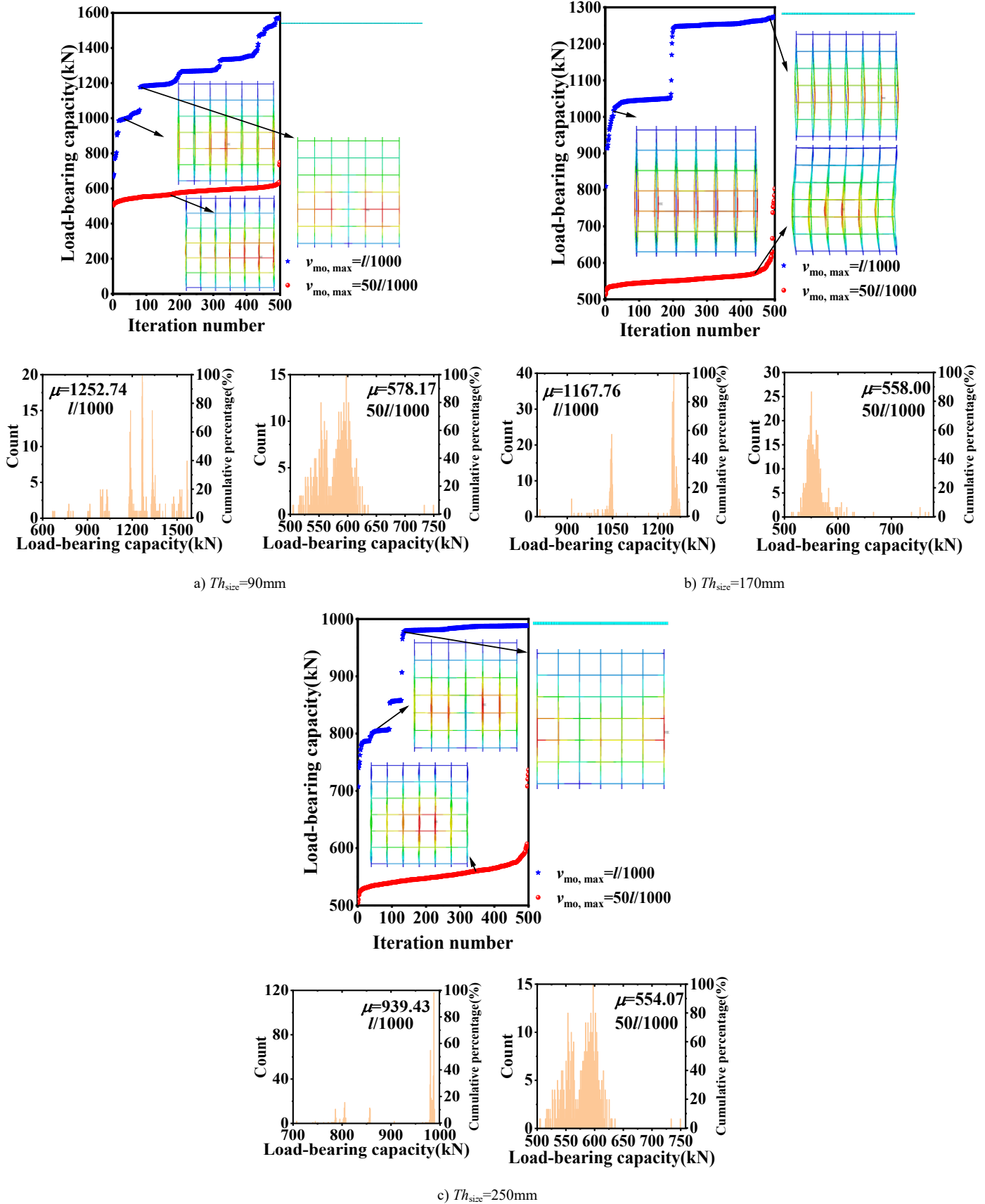


Fig. 24 Stochastic distribution of load-bearing capacity under different roof slab thicknesses

Quantitative comparisons reveal distinct thickness-dependent trends: the 90 mm slab experiences a 53.85% capacity reduction (from 1252.74 kN to 578.17 kN) under extreme curvature, while the 250 mm slab shows a 41.02% reduction (from 939.43 kN to 554.07 kN). The 10% capacity gap between these configurations highlights the thicker slab's enhanced adaptability to imperfections, attributed to its superior flexural rigidity delaying stress localization in scaffolding members. Mechanistically, thicker (250mm) slabs redistribute loads more effectively under moderate curvature but cannot compensate for scaffolding's inherent vulnerability under severe defects. These findings underscore a critical design balance: while slab thickening moderately improves defect tolerance, scaffolding quality control remains pivotal for ensuring system-wide stability under extreme geometric imperfections.

6. Load-bearing collaboration between roof slab and scaffolding

To investigate the collaborative load-bearing mechanism between the roof slab and scaffolding, the numerical model of the integrated system was modified by applying the element birth and death method. Under the condition of initial curvature imperfections in scaffolding vertical members, both the roof slab (modeled with SHELL181 elements) and its internal reinforcement (simulated by LINK180 elements) were selectively deactivated. This approach isolates the scaffolding's independent load-bearing behavior while retaining the geometric configuration of the deactivated components. The birth/death technique, defined as a state-dependent nonlinearity, induces abrupt stiffness transitions by multiplying the stiffness matrices of "killed" elements by an extreme reduction factor (e.g., 1×10^{-6}) rather than physically removing them. Although killed elements contribute zero to the load vector, their topological presence is preserved in the model. A numerical experimental matrix was designed as shown in the Table 4, and 500 randomized nonlinear buckling analyses were

performed for all working conditions.

For this analysis, the scaffolding was configured with planar dimensions of 6×6 and a height of $N_s=6$ layers, while the roof slab dimensions of $Tp_{size}=11.8m \times 11.8m$ and $Th_{size}=90mm$. Unlike previous studies, a concentrated load was applied at the top of vertical members to compare load transfer mechanisms. Computational results (Fig. 25) reveal that when only the scaffolding bears the load (Fig. 25 a and b), the capacity remains constant at approximately 200 kN regardless of initial curvature ($v_{mo, max}=l/1000$ or $50l/1000$), with nearly identical buckling modes. This indicates minimal curvature sensitivity in standalone scaffolding systems. In contrast, the collaborative system (roof slab and scaffolding) exhibits a dominant buckling mode capacity of about 800 kN higher than the scaffolding-alone configuration. The failure mode shifts from localized buckling at the scaffold top (scaffolding-alone case) to global instability in the mid-lower regions (collaborative system), demonstrating the slab's critical role in redistributing loads and transforming failure mechanisms. Mechanistically, the slab enhances system redundancy by engaging multiple load paths, mitigating stress concentrations caused by initial curvature. These findings conclusively establish the slab as an essential component in the collaborative system, elevating load-bearing capacity through synergistic interaction while fundamentally altering instability patterns.

On the other hand, the collaborative system exhibits a notable difference in computational results compared to those obtained from a standalone scaffolding configuration. The inclusion of a rigid slab significantly enhances both the stiffness and stability of the collaborative load-bearing system, with both components actively participating in load resistance. The involvement of the slab improves the overall load-bearing integrity, preventing the structural system from failing due to the localized failure of an individual member. This observation qualitatively validates the rationality and robustness of the model's behavior.

Table 4

Parameter matrix

Research Section	Variable Parameter Values	Fixed Parameter Values	Initial Imperfection Amplitude	Objective
6	roof slab and scaffold	$Tp_{size}=11.8m \times 11.8m$ $Th_{size}=90mm$	$v_{mo, max}=l/1000$ $v_{mo, max}=50l/1000$	Comparison of the Influence of Collaborative Load-Bearing
	scaffold	6×6 $N_s=6$		

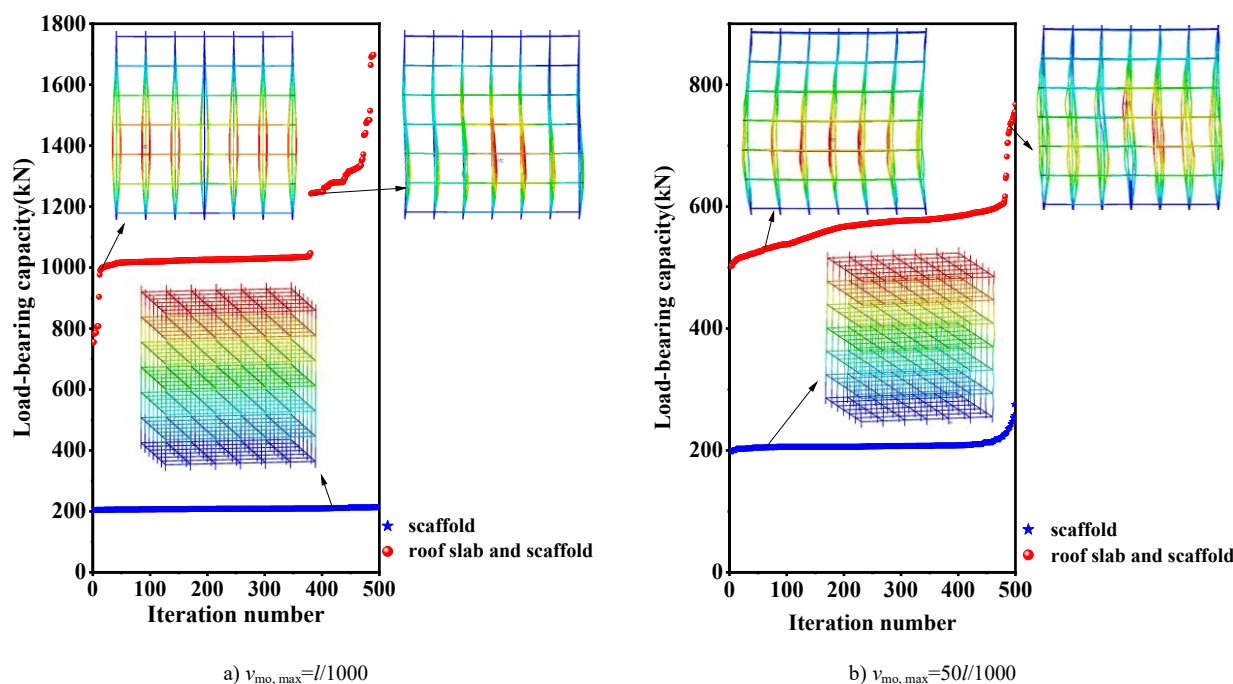


Fig. 25 Stochastic distribution of collaborative load-bearing capacity between roof slab and scaffolding

7. Conclusions

This study systematically investigated the influence of initial imperfections on the load-bearing capacity distribution of the roof slab-scaffolding collaborative system. A validated numerical model was developed using ANSYS finite element software, incorporating two critical geometric imperfections: joint positioning errors and initial curvature of vertical members.

Parametric analyses evaluated the effects of scaffolding planar dimensions, height, roof slab dimensions, and thickness on structural performance. Furthermore, the current numerical model of the collaborative load-bearing system between the roof slab and the scaffolding possesses idealized characteristics. In subsequent research, spring elements will be introduced between the slab and the scaffolding to quantify the performance difference between the assumptions of frictional connection and rigid connection. Key

findings are summarized as follows:

(1) The reliability of the numerical model was confirmed through comparative analysis between experimental moment-rotation curves of socket-template scaffolding joints and nonlinear spring element simulations. The integration of stochastic geometric initial imperfections in ANSYS ensured realistic predictions of collaborative system behavior.

(2) The system exhibits multiple buckling modes under stochastic initial imperfections, with load-bearing capacity inversely correlated to defect severity. Each capacity range corresponds to distinct instability patterns, demonstrating the critical role of imperfection randomness in failure mode transitions.

(3) Increasing scaffolding planar dimensions amplifies the detrimental impact of joint positioning errors. For 9-layer systems, capacity reductions reached 37.8% under extreme positioning errors ($\Phi_{\max}=H/100$), high heightened sensitivity in 9-layer systems.

(4) Expanding planar dimensions from 3×3 to 9×9 increased the load-bearing capacity range by 164% while reducing curvature sensitivity through enhanced structural redundancy. Taller systems (9-layer) transition from local to global buckling, necessitating prioritized global stability measures. Both undersized ($T_{\text{size}}=8.8\text{m}\times 8.8\text{m}$) and oversized ($T_{\text{size}}=14.8\text{m}\times 14.8\text{m}$) slabs showed high curvature sensitivity, with optimal performance observed at intermediate dimensions ($T_{\text{size}}=11.8\text{m}\times 11.8\text{m}$). A 250 mm slab demonstrated 41.02% lower curvature-induced capacity reduction compared to a 90 mm slab, validating thickness as a key defect mitigation parameter.

(5) Deactivating the roof slab via element birth/death techniques revealed its indispensable role, it is 800 kN higher than scaffolding-alone configurations. The slab transforms failure modes from localized member buckling to system-wide instability, redistributing 68%-72% of stresses to underutilized load paths.

(6) The study reveals that the collaborative load-bearing system under random initial imperfections exhibits multiple instability modes. It quantifies the influence of various erection parameters on the sensitivity to initial imperfections. The conclusions drawn from this research can provide a reference for the stability assessment and defect-resistant design of similar types of temporary structural components.

Acknowledgment

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STUDY ON SHEAR BEHAVIOR OF FOUR-CORNER BENT BAR CONNECTORS IN PREFABRICATED CONCRETE COMPOSITE FLOOR SLAB

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ABSTRACT

The four-corner bent bar connectors are used to connect the prefabricated concrete composite floor slab with the H-shaped steel beam, aiming to ensure working performance of the floor system. To investigate shear behavior of the four-corner bent bar connectors, a push-out test was conducted with the diameter of steel bar as a variable parameter. Additionally, a push-out specimen model was established using ABAQUS, and the accuracy of models was verified through comparison with test results. On this basis, the entire force process and key parameters of the shear behavior of the four-corner bent bar connectors were analyzed. The results indicate that the four-corner bent bar connector exhibits high initial stiffness and bearing capacity. Moreover, the characteristic displacements of four-corner bent bar connectors comply with the requirement that the lower slip limit for flexible connectors is 6mm, as stipulated in Eurocode 4. Therefore, the four-corner bent bar connector belongs to flexible connectors, demonstrating good ductility. Among the factors, the diameter of steel bars has the most significant influence on its shear bearing capacity, followed by the compressive strength of cast-in-place concrete, while the compressive strength of prefabricated concrete has the least influence. It is recommended that the diameter of steel bars for the four-corner bent bar connector should be set at 14mm, and the strength of steel bars should be HRB400. Based on the numerical simulation results, a calculation formula for the shear bearing capacity applicable to four-corner bent bar connectors was proposed using the method of multiple linear regression.

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Bearing capacity

1. Introduction

The prefabricated steel structure floor systems present advantages such as rapid construction and stable structure [1]. Among them, the prefabricated concrete composite floor system is more prominent due to its excellent integrity, crack resistance, high stiffness, and spacious interior layout. It has been widely employed in prefabricated buildings and has become one of the most commonly used forms in the prefabricated structure system [2]. Nevertheless, the bond effect at the interface between precast composite slabs and steel beams is relatively weak, failing to resist longitudinal shear force subjected to high load levels. In order to ensure the overall working performance of steel beams and precast floor slabs, the shear connectors must be installed between them.

The shear connectors primarily serve to withstand the longitudinal shear force and horizontal uplift forces between the steel beam and concrete [3]. Stud connectors, the most traditional type of shear connectors, are extensively used in practical engineering [4-7]. With the rapid development of composite structures, various types of shear connectors, including channel steel connectors, angle steel connectors, and bent bar connectors have been widely applied in all kinds of building structures. Among these, angle steel connectors offer distinct advantages over channel steel connectors, including a wider range of specifications, ease of installation, material savings, and high shear strength. By using standard welding equipment to secure steel plates to the beam flange plates via angle welds, their mechanical performance is ensured. They are now widely employed in building structures requiring high shear resistance [8].

The bent bar connector was first proposed by German scholar Leonhardt in the 1960s for optimizing the design of shear connectors in steel-concrete composite structures. Dhanush [9] analyzed truss connectors used in composite beams and segmental composite slabs, revealing that their load-bearing capacity remains stable beyond an inclination angle of 60 degrees. For given inclination angles, variations in reinforcement diameter showed no significant impact on performance. Bezerra [10] and Lima [11] investigated truss-type connectors with different angles, analyzing how geometric parameters influence shear capacity and slip behavior. Notably, Bezerra's experimental results indicated that truss-type connectors outperformed bolt-type connectors in both load-bearing capacity and slip performance [12]. Kanchanadevi [13] examined the effects of 30, 45, and 60 degrees inclination angles on the performance of truss-type connectors in floors, finding that a 45 degree angle yields optimal mechanical performance by enhancing shear capacity while effectively controlling slip behavior. Li [14] pioneered curved bar connectors in composite beams, demonstrating their shear resistance under monotonic and cyclic loading conditions. They established corresponding load-bearing formulas that laid the foundation for subsequent studies. Zhou [15] conducted the first lattice-type derivation experiment for reverse-arranged curved bar connectors, analyzing their working mechanisms and deriving load-bearing formulas under reverse arrangement. Zhang [16] investigated serpentine curved bar connectors in

beams, analyzing the effects of bar diameter, anchorage length, and concrete strength through derivation experiments. Zhang [17] first proposed the application of short-steel-bar connectors in bridge decks, using ABAQUS software to specifically examine how connector diameter and weld length influence shear capacity and slip behavior. Shao [18] analyzed short-steel-bar connectors, revealing their excellent shear transfer capability and high shear capacity, with shear capacity increasing as weld length increases. Yu [19] explored steel bar connectors in corrugated composite structures, revealing their shear performance and failure characteristics under complex conditions, and derived shear stiffness calculation formulas. In summary, previous research primarily focused on comparative analysis of connector advantages through shape modifications and the influence of different factors.

Based on the performance characteristics of existing bent bar connectors, this paper proposed a new-type shear connector, which employs the four-corner bent bar shear connectors to connect precast concrete composite floor slabs with H-shaped steel beams for ensuring the integral composite behavior of the floor system (as shown in Fig. 1). Compared with the commonly used stud connectors, the four-corner bent bar shear connectors exhibit superior shear resistance, pull-out resistance, uplift resistance and fatigue resistance. Under identical load conditions, the required number of connectors is significantly reduced, thereby enhancing construction efficiency. Furthermore, due to the structural characteristics of the four-corner bent bar shear connectors, the formed spatial configuration can effectively enhance the synergistic performance between the bent bar and concrete, while overcoming the limitation that conventional bent bar connectors can only bear shear force in a single direction. The push-out tests and finite element analysis were carried out on specimens with four-corner bent bar connectors to investigate the failure modes, entire load-carrying processes, and load-slip curves. The influences of parameters such as steel bar diameter, steel bar yield strength and concrete compressive strength on the shear behavior of connectors were analyzed. On the basis of numerical analysis results, a bearing capacity calculation formula for this type of connector was proposed, providing a reference for its design and engineering application.

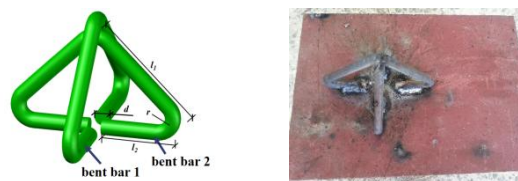


Fig. 1 Four-corner bent bar shear connectors

2. Experimental investigation

2.1. Specimen design

The specimens were designed according to Eurocode 4 [20]. The dimensions of the H-shaped steel beam were specified as 300mm×300mm×10mm×15mm, the composite slab (containing cast-in-place and precast concrete slabs) was 500mm×150mm×400mm, and the cast-in-place

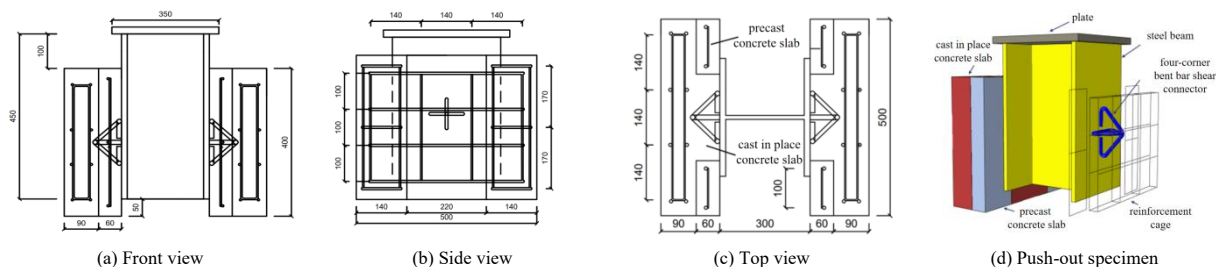


Fig. 2 Geometric dimensions of the specimen

Table 1
Parameters of four-corner bent bar connectors

(a) Bend bar 1						(b) Bend bar 2					
Specimens	d/mm	l_1/mm	l_2/mm	$\alpha/^\circ$	r/mm	Specimens	d/mm	l_1/mm	l_2/mm	$\alpha/^\circ$	r/mm
WJ1-1	12	90	54	45	12	WJ2-1	12	107	54	45	12
WJ1-2	14	105	63	45	14	WJ2-2	14	125	63	45	14
WJ1-3	16	120	72	45	16	WJ2-3	16	143	72	45	16

Note: d is the steel bar diameter; l_1 denotes the inclined length of the steel bar; l_2 denotes the straight segment length of the steel bar; α is the bent bar angle; r is the bent bar radius.

2.2. Material performance

Both the H-shaped steel beam and the end plate were manufactured using Grade Q235B. Four-corner bent bars and steel reinforcement framework adopted Grade HPB300 steel, among which the stressed bars were $\phi 8$ spiral bars and the distribution bars were $\phi 6$ spiral bars. Precast concrete slabs were cast with C30 concrete, while the cast-in-place concrete slab adopted C45 concrete. Standard 150mm cubic specimens were cured consistent with the same field conditions. Test measurements showed that the average cubic compressive strength of C45 and C30 concrete were 45.3MPa and 30.0MPa, and the elastic modulus were 3.35×10^4 MPa and 3.00×10^4 MPa, respectively. The material performance parameters of the four-corner bent bar shear connectors are specified in Table 2.

Table 2
Material performance of steel

Bar diameter	Steel grade	f_y/MPa	f_u/MPa	E_s/GPa
$d=12mm$	HPB300	346.0	530.5	201
$d=14mm$	HPB300	354.2	546.3	209
$d=16mm$	HPB300	403.6	560.4	210

Note: f_y , f_u , E_s represent the yield strength, tensile strength and elastic modulus of the steel respectively.

2.3. Loading device and scheme

The loading device is depicted in Fig. 3. A 100-ton hydraulic jack was adopted for load application. The test was subjected to monotonic concentrated

load, which consisted of two stages: preloading and formal loading. The specific loading scheme was consistent with that detailed in reference [21].

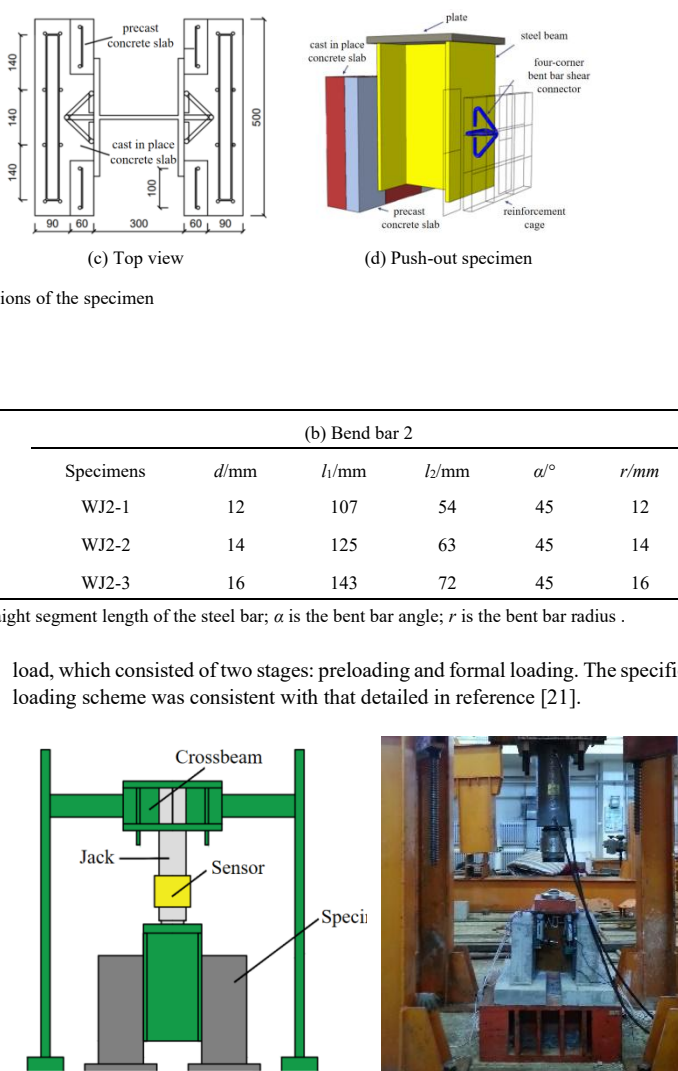


Fig. 3 Test setup

2.4. Measurement scheme

The arrangement of the strain gauges is presented in Fig. 4(a). Strain gauges 1-4 were located on the surface of the steel bars at the welding position of steel beam, while strain gauges 5-8 were located on the surface of inclined steel bars. As depicted in Fig. 4(b), two displacement transducers were set at the parallel and perpendicular positions to the welded joint between the mid-span steel beam and connectors, to measure the relative slip at the connector-steel beam interface, alongside vertical relative displacement between the H-shaped steel beam and concrete slab.

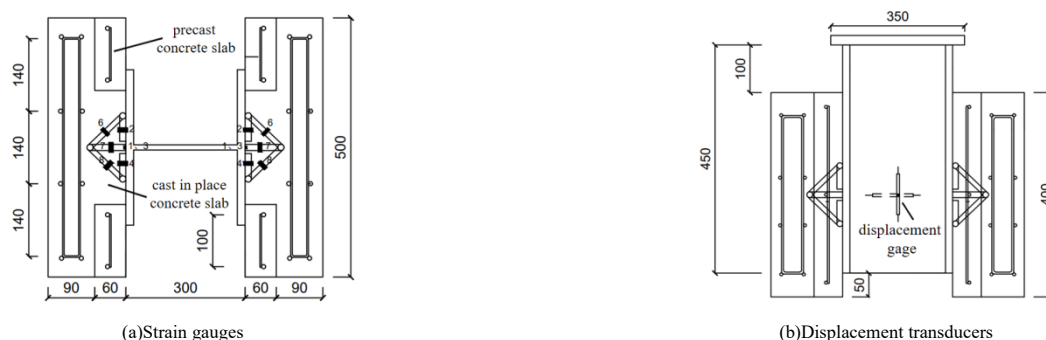


Fig. 4 Arrangement of strain gauges and displacement meters

3. Experimental analysis

3.1. Experimental phenomena and failure modes

The failure modes of specimens in three groups were similar, as shown in Fig. 5. In all cases, there was a significant sliding between the composite floor slab and H-shaped steel beam. Meanwhile, shear failure was observed at the weld joint connecting the steel beam and four-corner bent bar shear connectors, accompanied by spalling of the concrete surface.

Taking WJ-6 as the representative specimen, the experimental phenomena

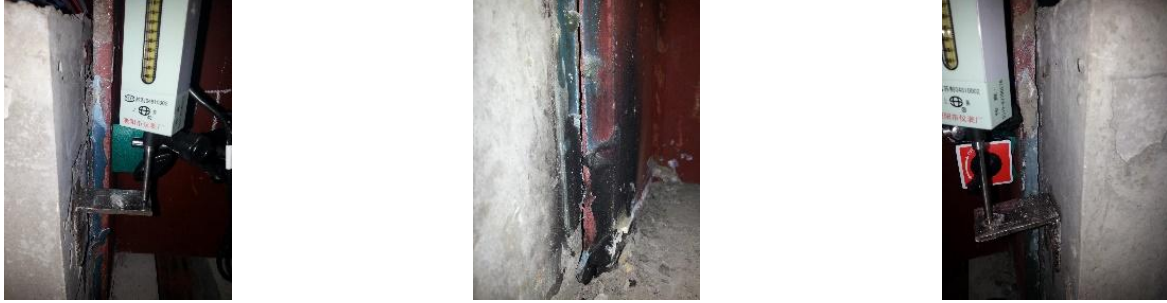


Fig. 5 Failure modes of specimens

3.2. Load versus slip curve

The load-slip curves (P - δ curves) are presented in Fig. 6. It is observable that the loading processes and failure states of all specimens are substantially consistent and can be divided into three stages. Elastic stage: For the 14-mm-diameter four-corner bent bar connector, its initial stiffness outperformed that with 12-mm-diameter connector and 16-mm-diameter connector. In this stage, P - δ curves of the three specimens displayed a linear relationship, indicating that specimens were in elastic working phase. As the loading process proceeds, the slope of the curve begins to decrease, the stiffness of the connector gradually reduces, and the specimen enters the elastoplastic stage. In this stage, the load increased slowly and no significant changes occurred in the specimen when the load reached the peak load. Descending stage: Continued loading, the load began to decrease and the slip value increased rapidly.

The characteristic slip values of all specimens range from 6.04mm to 11.48mm, which meet the slip value requirements (not less than 6mm) for flexible connectors in Eurocode 4 [20]. Therefore, the four-corner bent bar connector belongs to flexible connectors. As the steel bar diameter gradually increased, both the ultimate displacement and peak load increased significantly. The yield load increased by approximately 24.3% and 25.0% respectively, with the most remarkable improvement occurring when the

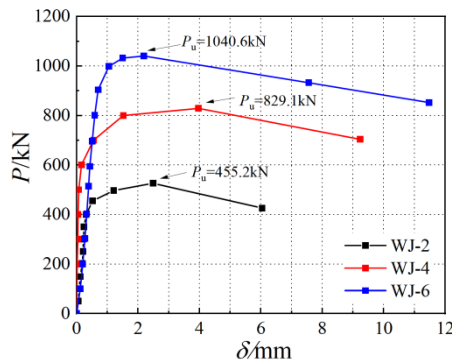
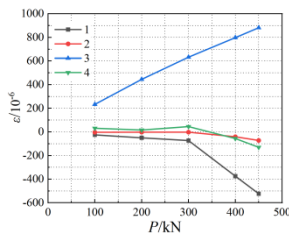
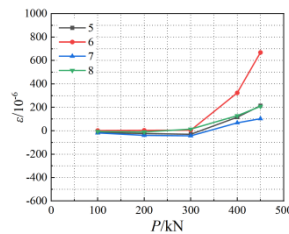


Fig. 6 P - δ curves



(a) Surface of horizontal bar of WJ-2



(b) Surface of diagonal bar of WJ-2

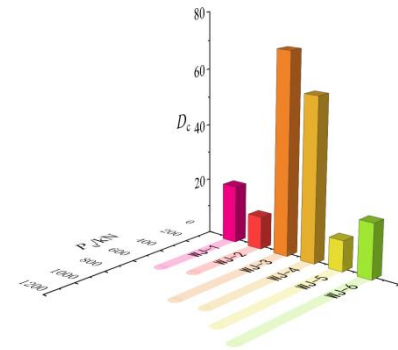
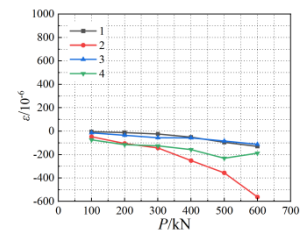


Fig. 7 P - D_c curves



(c) Surface of horizontal bar of WJ-4

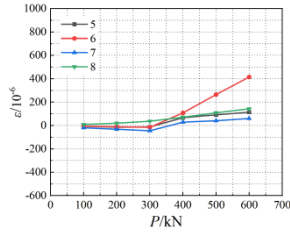
were elaborated. In the initial loading stage, there was no significant change in the specimen. When load reached 77% P_u , a slight sound was heard from interior of the composite slab, and a small amount of concrete blocks fell off at connection between the composite slab and steel beam. When load reached P_u , no obvious phenomena appeared. Continuing loading, a large amount of sliding occurred between the slab and beam, leading to severe damage of the specimen. The loading was terminated once the load reached 80% P_u . It was observed that the specimen ultimately failed via shear failure at the interface of the steel beam and composite slab. Partial concrete spalling appeared on the contact interface, while no obvious cracks were observed in the remaining concrete areas.

diameter increased from 12mm to 14mm, where the ultimate displacement and peak load increase by 34.6% and 36.5% respectively.

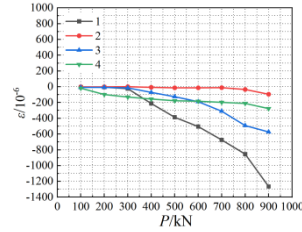
Fig. 7 depicts the bearing capacity and ductility coefficient (D_c)[22] curves. The ductility coefficients of specimens with different diameters were 11.6, 57.7, and 19.5 respectively. After reaching the peak load, all specimens exhibited excellent ductility performance during the failure stage. Notably, when the diameter increased from 12mm to 14mm, ductility improved 4.9 times.

3.3. Load versus strain curve

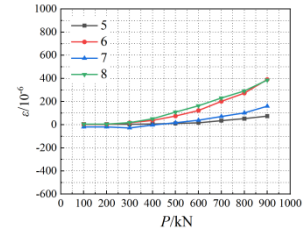
Fig. 8 presents the load-strain curves (positive values indicate tension; negative values indicate compression). An analysis of the curves indicated that the horizontal steel bars of all the specimens were in compression, while the diagonal steel bars were in tension. The strain gauge (2) on the upper surface of horizontal steel bars in WJ-2 was in tension. When compared with other strain gauges, it is speculated that the failure of the strain gauge might have caused the error. Based on the material property tests, the yield strains of 12mm, 14mm and 16mm steel bars were 1721, 1694 and 1922 respectively. By comparing the curve results, it can be seen that none of the bent bars of all the specimens yielded.



(d) Surface of diagonal bar of WJ-4



(e) Surface of horizontal bar of WJ-6



(f) Surface of diagonal bar of WJ-6

Fig. 8 Load-strain curves

4. Finite element analysis

4.1. Model establishment and validation

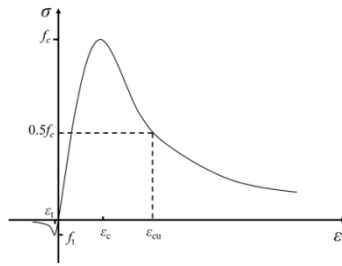
4.1.1. Material constitutive relation

A detailed numerical simulation analysis of mechanical performance of push-out specimens were conducted using ABAQUS. The constitutive model of the concrete was defined by the stress-strain curve stipulated in Appendix C of GB50010-2010 [23], as depicted in Fig. 9(a). For steel, a bi-linear elastic-plastic constitutive model [24] was utilized, as shown in Fig. 9(b).

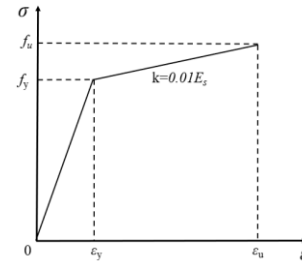
4.1.2. Element type selection and grid division

The models encompass precast concrete slab, cast-in-place concrete slab, H-shaped steel beam, four-corner bent bar shear connectors, and steel reinforcement frameworks, with the geometric dimensions of each member

corresponding to those of the test specimens. The concrete slabs and H-shaped steel beam were simulated utilizing three-dimensional solid eight-node hexahedral (C3D8R) elements, and the contact surfaces of slabs and connectors were modeled using three-dimensional solid four-node tetrahedral (C3D4) elements. The four-corner bent bar shear connectors were simulated with C3D8R, and the steel reinforcement framework was simulated via three-dimensional two-node (T3D2) truss elements [25]. In terms of grid division, mesh sensitivity analysis was performed to determine a global mesh size of 20 mm for the entire component, aiming to improve computational efficiency and facilitate model convergence. Meanwhile, to accurately simulate the mechanical behavior at the connection zone, mesh refinement was implemented for the contact interface between the concrete and the steel beam as well as the connectors. A mesh size of 2.5 mm was assigned to the concrete and the steel beam within this contact region, whereas a mesh size of 5 mm was used for four-corner bent bar connectors. The specific grid division is shown in Fig. 10.

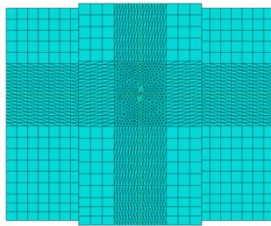


(a) Concrete

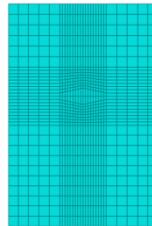


(b) Steel

Fig. 9 Material constitutive model



(a) Cast-in-place concrete slab



(b) H-beam



(c) Four-corner bent bar connector

Fig. 10 Grid division diagram

4.1.3. Contact relations and boundary conditions

Tie constraints were applied between precast concrete slabs and cast-in-place concrete slab, as well as between steel beams and connectors. Hard contact was used between precast concrete slabs and steel beam. Surface-to-surface contact was defined for interfaces of cast-in-place concrete slabs with steel beam and connectors, among which, hard contact was set in the normal direction and penalty friction coefficients in the tangential direction were 0.2 and 0.4 respectively. To guarantee the convergence of the model, contact control was executed at the interface between cast-in-place concrete slab and four-corner bent bar connectors, and embedded region constraints were imposed on the steel reinforcement frameworks. The bottoms of precast concrete slabs and cast-in-place concrete slabs were subjected to fully fixed constraints, while the upper surface of the beam was coupled and simulated using displacement loading. The specific modeling approach is illustrated in Fig. 11.

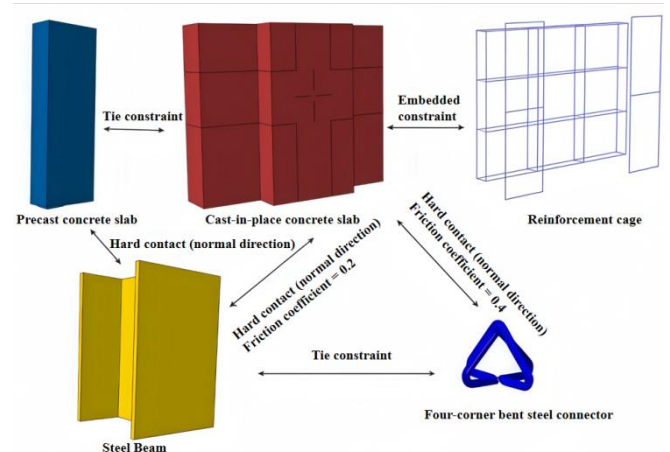


Fig. 11 Contact relations and boundary conditions diagram

4.1.4. Modelling verification

The comparison between experimental results and load-slip curves obtained from finite element simulation (Fig. 12 and Table 3) indicates that the overall trends of the simulated load-slip curves agree well with the experimental

data. Excellent consistency is observed in initial stiffness, and the relative error of peak loads is within 10%. The above comparisons verify the reliability of the proposed modeling method, laying a solid foundation for subsequent parametric investigations.

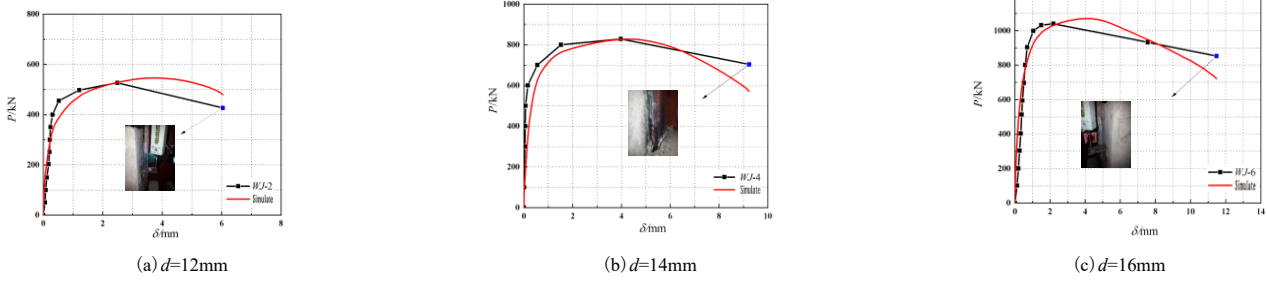


Fig. 12 Comparison of P - δ curves between test and finite element

Table 3

Table of test and FEM results

Specimen Number	Test		FEM	
	P_u /kN	Mean	$P_{u,FEM}$ /kN	$P_{u,FEM}/P_u$
WJ-1	576.3	551.5	545.7	0.99
WJ-2	526.7			
WJ-3	846.7			
WJ-4	829.1	837.9	827.9	0.99
WJ-5	950.6			
WJ-6	1040.6			

Note: P_u denotes peak load of the test; $P_{u,FEM}$ represents the peak load obtained from the finite element model.

4.2. Analysis of full mechanical response process

Taking WJ-4 as the typical specimen, corresponding numerical analysis model was established. The P - δ curve acquired via numerical simulation (Fig. 13) revealed three distinct stages demarcated by three characteristic points: the elastic working stage (OA), the elastoplastic working stage (AB), and the descending stage (BC). Point A represented the yield load at position S of the connector (where the lower bending point of the longitudinal horizontal steel bar of the connector was defined as point S). Point B represented the peak load (P_u) at position S of the connector. Point C was the termination point of loading. The stress and strain distributions of components at the three characteristic points are presented in Figs. 14 to 16.

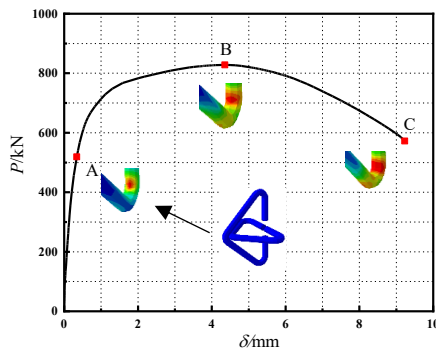


Fig. 13 P - δ curve

From the initial loading stage up to point A, the component demonstrated high initial stiffness, and the four-corner bent bar connectors remained in the elastic stage. Upon reaching point A, considerable stress was generated along the steel beam from the loaded end up to the welding surfaces with transverse bars of the four-corner bent bar connectors (Fig. 14(b)). It can be seen from Figs. 14(c) to 14(e) that extrusion contact emerged at the interfaces between the bent parts of transverse horizontal bars and diagonal bars of connectors and the cast-in-place concrete slab, where local regions reached the ultimate compressive strain. Simultaneously, extrusion contact also occurred at the upper bends of longitudinal horizontal bars and diagonal bars of connectors, as well as at the welding positions between the undersides of longitudinal horizontal bars and steel beams. However, almost no strain was generated between the cast-in-place

concrete slab and precast concrete slabs. The transverse horizontal bars near the bending point, as well as the longitudinal horizontal bars, and the longitudinal inclined bars all experienced significant stress. Among them, the inner side of the transverse horizontal bars near the bending point reached the yield load and began to exhibit local yielding phenomenon (Fig. 14(f)). At this point, the load is $0.72 P_u$, with a displacement of 0.49mm .

When loading from point A to point B, the component entered the elastoplastic working stage. Upon reaching point B, the component attained its ultimate load. Considerable compressive strains emerged in the cast-in-place concrete slab at the inner side of bending positions of the transverse horizontal bars, longitudinal horizontal bars, and longitudinal inclined bars of the connector, and some local strains even exceeded the ultimate compressive strain, indicating that the cracks had already occurred in the concrete slab at this point (Figs. 15(c)-15(e)). Large stresses were generated at the welded joints of steel beams and connectors (Fig. 15(b)), and local regions of the welded joints of the transverse horizontal bars reached the yield load (235MPa). Meanwhile, the bent regions of transverse horizontal bars, longitudinal horizontal bars and inclined bars all reached the yield load. Notably, the lower inner side of the bending section of the longitudinal horizontal bars locally reached the ultimate tensile stress (546.3MPa), indicating that local failure and damage had occurred in the steel bars here (Fig. 15(f)).

When loading from point B to point C, the component entered the descending stage. Upon reaching point C, large strains were observed at the welding joints between the steel beams and the horizontal bars, as well as at the bending positions where the longitudinal horizontal bars met the inclined bars (Fig. 16(b)). At the same time, the concrete in the immediate vicinity of the horizontal bars of the connectors and the bending points of these bars, as well as the longitudinal horizontal bars and their bending points, all reached the ultimate compressive strain (Figs. 16(c)-16(e)). Compared with the test results, no obvious cracks appeared in the concrete slabs, and the phenomenon remained consistent. The upper portions of transverse horizontal bars, longitudinal horizontal bars and inclined bars, as well as the lower segments of inclined (from the bending point of the horizontal bars to 1/3 of the length of the inclined bars) all generated large stresses. The stress at the upper bent part of the longitudinal horizontal bars was greater than that at the lower bent part, and it failed earlier than the transverse horizontal reinforcement (Fig. 16(f)). At this time, high weld stress existed at the welding interface between the connectors and the steel beam. The connector lost its shear capacity, which further induced shear failure of the component.

4.3. Extended parameter analysis

The bending angle of the four-corner bent bar connector designed in this paper was kept constant at 45° . By changing other variables to analyze the influence of different factors on performance of the four-corner bent bar connector, where the lengths of the horizontal bars ($1-l_1$, $2-l_1$), the longitudinal bars ($1-l_2$, $2-l_2$), and the height of the connector (H_1 , H_2), due to their special structural form, changed with the diameter of the bars. According to GB50010-

2010[23], the length requirements were as follows. Fig. 17 presents the schematic diagram of the four-corner bent bar connector. This study established 21 numerical models, and the selected parameters included: steel bar diameter d (ranging from 8 mm to 18 mm), precast concrete compressive strength f_{cu} (30MPa-70MPa), f_{cu} (30MPa-70MPa), and f_{yk} (300MPa-500MPa). The specific parameters are detailed in Table 4.

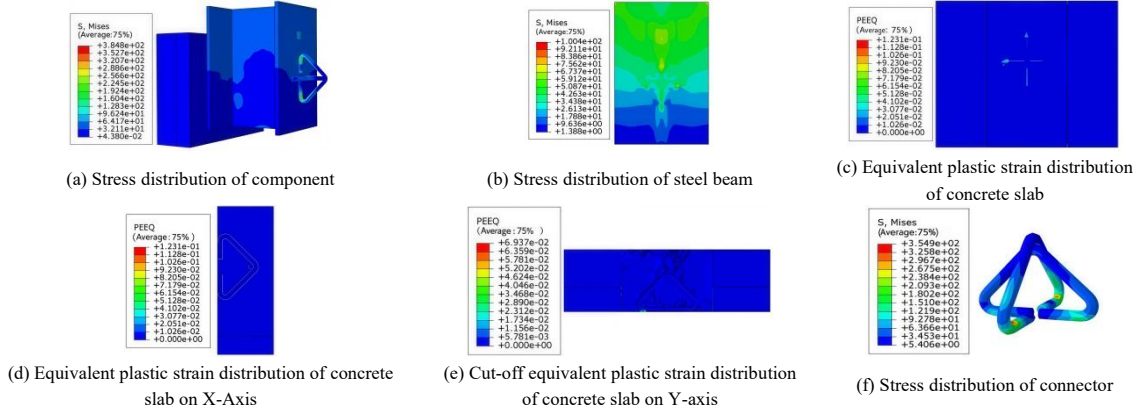


Fig. 14 Stress and strain distributions of WJ-4 (Point A)

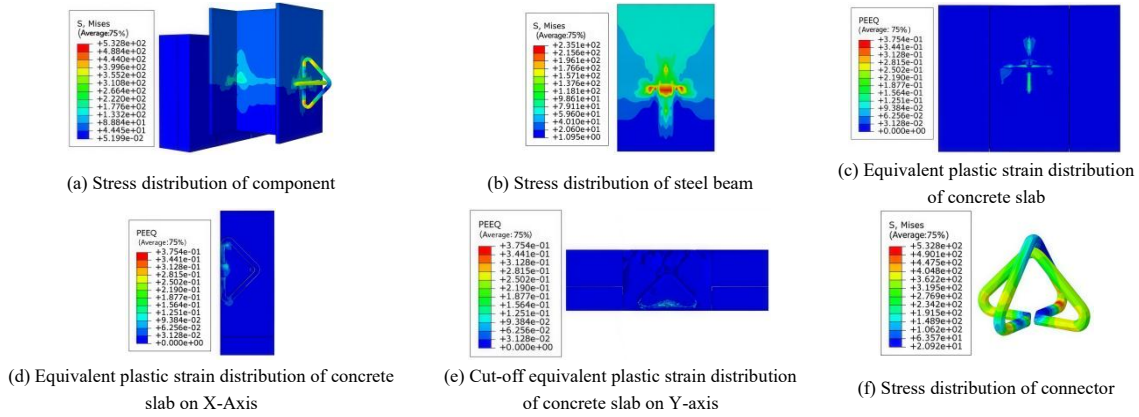


Fig. 15 Stress and strain distributions of WJ-4 (Point B)

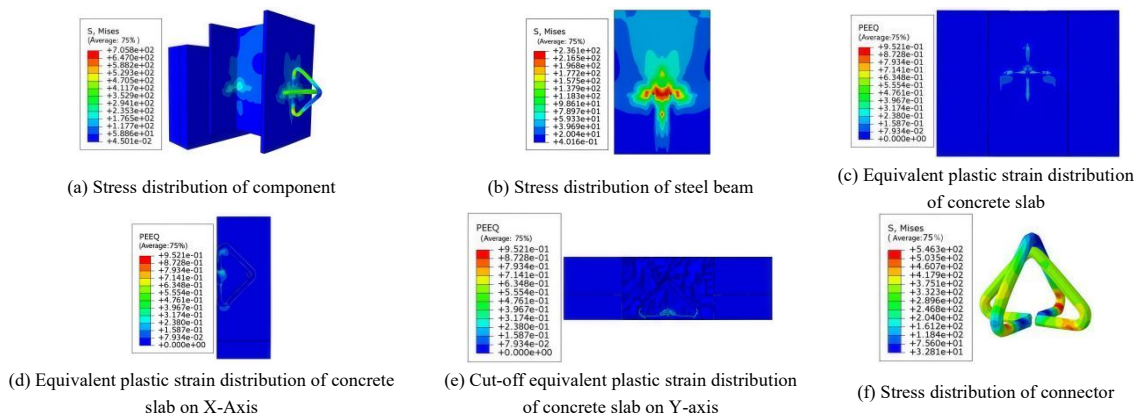


Fig. 16 Stress and strain distributions of WJ-4 (Point C)

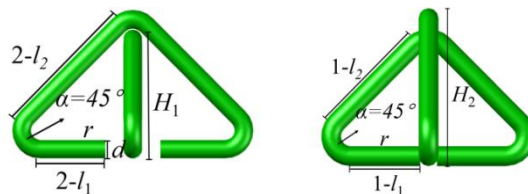


Fig. 17 The four-corner bent bar connector

Table 4

Details of components

Component number	d/mm	r/mm	$1-l_1/mm$	$1-l_2/mm$	$2-l_1/mm$	$2-l_2/mm$	f_{cu}/MPa	f_{cu}'/MPa	Grade	$P_{ub,FEM}/kN$
WJS-1-1	8	8	28	44	28	55.5	30	40	HPB300	222.1
WJS-1-2	10	10	35	55	35	69.5	30	40	HPB300	235.5
WJS-1-3	12	12	42	66	42	83	30	40	HPB300	248.7
WJS-1-4	14	14	49	77	49	97	30	40	HPB300	276.9
WJS-1-5	16	16	56	88	56	111	30	40	HPB300	322.5
WJS-1-6	18	18	63	99	63	126	30	40	HPB300	355.7
WJS-2-1	14	14	49	77	49	97	30	40	HPB300	276.9
WJS-2-2	14	14	49	77	49	97	40	40	HPB300	278.5
WJS-2-3	14	14	49	77	49	97	50	40	HPB300	279.5
WJS-2-4	14	14	49	77	49	97	60	40	HPB300	280.2
WJS-2-5	14	14	49	77	49	97	70	40	HPB300	280.7
WJS-3-1	14	14	49	77	49	97	30	30	HPB300	256.8
WJS-3-2	14	14	49	77	49	97	30	40	HPB300	276.9
WJS-3-3	14	14	49	77	49	97	30	50	HPB300	292.9
WJS-3-4	14	14	49	77	49	97	30	60	HPB300	304.8
WJS-3-5	14	14	49	77	49	97	30	70	HPB300	314.2
WJS-4-1	14	14	49	77	49	97	30	40	HPB300	276.9
WJS-4-2	14	14	49	77	49	97	30	40	HRB335	282.2
WJS-4-3	14	14	49	77	49	97	30	40	HRB400	293.35
WJS-4-4	14	14	49	77	49	97	30	40	HRB500	307.75
WJS-4-5	14	14	49	77	49	97	30	40	HRB600	318.9

Note: $1-l_1$ represents the diagonal length of the transverse bar; $1-l_2$ indicates the straight length of the transverse bar; $2-l_1$ denotes the diagonal length of the longitudinal bar; $2-l_2$ indicates the straight length of the longitudinal bar; $P_{ub,FEM}$ is the bearing capacity of the four-corner bent bar connector obtained from the finite element model.

(1) Steel bar diameter (d)

As depicted in Fig. 18(a), when the steel bar diameter was incrementally increased from 8mm to 18mm, the shear capacity of components increased by 6%, 12%, 24.7%, 45.2%, and 60.2% respectively. Among them, the most substantial improvement occurred when d increased from 12mm to 14mm, increasing by 16.5%. This indicates that d is primary factor affecting shear capacity of the four-corner bent bar shear connectors. This is mainly attributed to two aspects: the excellent tensile performance of the steel bars themselves, and the unique spatial configuration of the four-corner bent bars, which provides effective confinement to the surrounding concrete and the concrete inside connectors, thereby significantly improving the shear capacity. Additionally, the ductility of components gradually increases as d increases. These results indicate that increasing d can effectively improve the shear capacity of components within a certain range. It is recommended that the steel bar diameter should not be less than 12mm.

(2) Precast concrete compressive strength (f_{cu})

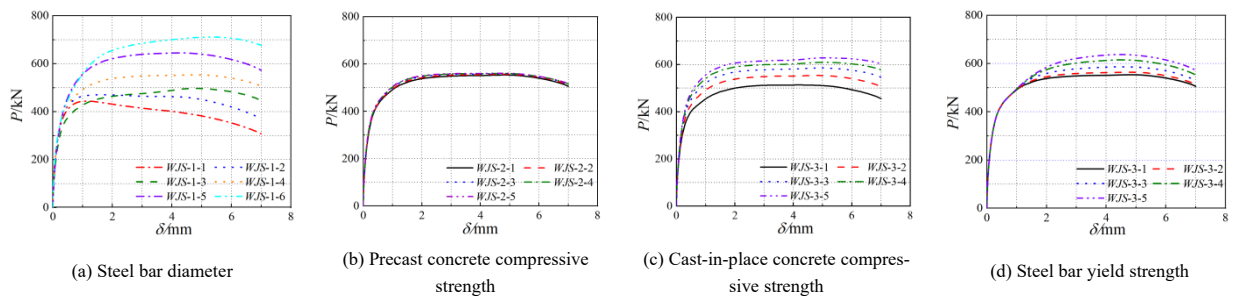
As shown in Fig. 18(b), when the precast concrete compressive strength was increased from 30MPa to 70MPa, the bearing capacity improved by 0.6%, 0.9%, 1.2%, and 1.4% respectively. The increase rate was less than 2%, which was considered negligible. Only when f_{cu}' was enhanced from 30MPa to 40MPa, does the bearing capacity increased relatively significantly. Therefore, considering both bearing capacity and economic benefits, it is recommended to select precast concrete with a compressive strength of 40MPa.

(3) Cast-in-place concrete compressive strength (f_{cu})

As presented in Fig. 18(c), when the cast-in-place concrete compressive strength was raised from 30MPa to 70MPa, the shear capacity of components increased by 7.8%, 14.1%, 18.7%, and 22.4% respectively. This demonstrates that f_{cu} is a crucial factor in enhancing bearing capacity of connectors. When the connector bears shear load, the load is primarily resisted by the bearing action of longitudinal bars, the concrete beneath longitudinal bars, and the concrete enclosed within the four-corner bent bars. As f_{cu} increases, the bearing capacity improves accordingly, which effectively enhances the shear behavior of components. However, the improvement effect of bearing capacity of four-corner bent bar connectors gradually decreases as f_{cu} increases (Fig. 3.19(b)). Therefore, it is recommended to select the cast-in-place concrete compressive strength according to construction requirements.

(4) Steel bar yield strength (f_{yk})

As illustrated in Fig. 18(d), the shear capacity of four-corner bent bar connectors increases non-linearly with the increase in the steel bar yield strength. When the strength grade of the steel bar was gradually upgraded from HRB300 to HRB335, HRB400, HRB500, and HRB600, the bearing capacity increased by 1.9%, 6%, 11.2%, and 15.2% respectively. This shows that connector with HRB400 steel bars exhibits a more significant improvement in shear capacity and better economic efficiency.

**Fig. 18** The influence of different parameters on shear capacity

4.4. Design suggestions of four-corner bent bar connector

The parameter analysis indicates that the shear performance of four-corner bent bar connectors is significantly affected by the steel bar diameter, the cast-in-place concrete compressive strength, and the steel bar yield strength, whereas the precast concrete compressive strength has a negligible impact. Therefore, the recommended design parameters are as follows: the steel bar diameter should be greater than or equal to 12 mm, and the grade of steel bar should be less than or equal to HRB400. Taking economic efficiency and structural requirements into account, when the steel bar diameter is 14 mm and the strength grade is HRB400, the four-corner bent bar connector exhibits better shear behaviour in connecting the prefabricated concrete composite floor slab to H-shaped steel beam. The strengths of other materials should be selected in accordance with design requirements.

5. Shear bearing capacity calculation

5.1. Calculation formula for shear capacity of bent bar connector

The European Code (EC 4) [20] takes into account factors such as the steel bar strength, geometric properties of connectors, and bonding characteristics between materials. The calculation considers not only the shear capacity but also the bond strength between the steel bar and concrete. The proposed formula focuses on analyzing the internal forces of truss web members and the local crushing concrete, as presented in the following equation:

$$P_x = A_u f_y \sqrt{\cos^2 \alpha + \frac{\sin^2 \alpha}{\gamma}} \quad (1)$$

Where: A_{st} denotes the cross-sectional area of the steel bar and f_y refers to its

yield strength; α is the angle between the steel bar and steel beam; γ is the resistance coefficient, typically ranging from 1.00 to 1.25.

When $\alpha \leq 45^\circ$, the expression is as follows:

$$P_r = \frac{A_{st} f_y}{1.2\gamma} \quad (2)$$

The American Concrete Institute (ACI) [26] has considered the frictional interaction between the steel beam and concrete, and concluded that shear force is transmitted through shear reinforcement, the following formula is provided:

$$P_x = A_x f_y (\cos \alpha + \mu \sin \alpha) \quad (3)$$

where μ is the coefficient of the resistance component, and $\mu = 0.7$.

Li [27] had analyzed the effect of flexible connectors in composite beams, indicating that these connectors not only transmit shear force but also control the deformation at the beam end. It is of vital importance to the overall stability and bearing capacity. Further, local pressure of the concrete at welded end of the bent bar was considered, and a formula was provided:

$$V_u = A_f \sqrt{(\cos \alpha + \mu \sin \alpha)^3 + \frac{1}{2} (\sin \alpha - \mu \cos \alpha)^3 + P} \quad (4)$$

Where: $p = \beta A_{st} f_y$, $\beta = 5.3 \sim 12$.

The Chinese Code (GB50017-2003) [28] only considers the strength effect of the steel bar itself and gives the following formula for the bearing capacity calculation:

$$N_{\pi}^k = A_{st} f_{st} \quad (5)$$

Table 5

Comparison of calculated values from various national standards and simulation results

Component number	$P_{ub,FEM}/kN$	μ_1	μ_2	μ_3	μ_4
WJS-1-1	222.1	0.07	0.08	0.46	0.07
WJS-1-2	235.5	0.10	0.12	0.68	0.10
WJS-1-3	248.7	0.14	0.16	0.92	0.14
WJS-1-4	276.9	0.17	0.20	1.13	0.17
WJS-1-5	322.5	0.19	0.22	1.27	0.19
WJS-1-6	355.7	0.22	0.26	1.45	0.21
WJS-2-1	276.9	0.17	0.20	1.13	0.17
WJS-2-2	278.5	0.17	0.20	1.12	0.17
WJS-2-3	279.5	0.17	0.20	1.12	0.17
WJS-2-4	280.2	0.17	0.20	1.12	0.16
WJS-2-5	280.7	0.17	0.20	1.11	0.16
WJS-3-1	256.8	0.19	0.22	1.22	0.18
WJS-3-2	276.9	0.17	0.20	1.13	0.17
WJS-3-3	292.9	0.16	0.19	1.07	0.16
WJS-3-4	304.8	0.16	0.18	1.03	0.15
WJS-3-5	314.2	0.15	0.18	0.99	0.15
WJS-4-1	276.9	0.17	0.20	1.13	0.17
WJS-4-2	282.2	0.19	0.22	1.24	0.18
WJS-4-3	293.4	0.22	0.25	1.42	0.21
WJS-4-4	307.8	0.26	0.30	1.69	0.25
WJS-4-5	318.9	0.30	0.35	1.96	0.29
AVE		0.18	0.21	1.16	0.17
STE		0.08	0.52	0.08	0.18

Note: μ_1 , μ_2 , μ_3 , and μ_4 represent the ratios of the calculated values P_c from the Equation 1, 3, 4, 5 to $P_{ub,FEM}$, respectively. AVE indicates the data average, while STE denotes the standard deviation.

As shown in Table 5, a comparison between the calculated values from various national standards and the simulation results. It can be seen that when the steel bar diameter is less than 10 mm, the results of four formulas for the bearing capacity are all relatively small. The main reason for this is that the four

formulas only consider the influence of a single steel bar on the bearing capacity. Specifically, Formula 2, Formula 3, and Formula 5 are overly conservative in their calculations of the shear bearing capacity for four-corner bent bar shear connectors, mainly due to the special structural form of concrete and the four-

corner bent bar was not considered for the influence on the bearing capacity. The calculation results of Formula 4 are relatively close, but the mean error is still large and the dispersion is high. The primary reasons for this phenomenon are twofold: the contribution of concrete to bearing capacity was neglected, and the effect of bent bar on shear capacity was considered twice.

5.2. Calculation formula for shear capacity of four-corner bent bar connector

Based on an investigation of the parametric influence laws of four-corner bent bar shear connectors and a summary of shear capacity formulas for steel bars presented in various national design codes, multivariate nonlinear regression analysis was carried out on the numerical results of 21 finite element models, and the following shear capacity calculation formula was obtained:

$$P_u' = (2.4A_{s2}f_r - 2.8A_{s1}f_r)(\cos \alpha + \phi \sin \alpha) + 0.27A_c f_c \quad (6)$$

Where: A_{s1} , A_{s2} represent the cross-sectional area of transverse steel bar and longitudinal steel bar, respectively; α is the bending angle (a value of 45° is recommended); A_c is the cross-sectional area of cast-in-place concrete slab; ϕ is the coefficient of the resistance component, and $\phi = 0.7$; 2.4, 2.8, 0.27 represent the weighted contributions of the transverse steel bar, longitudinal steel bar, and cast-in-place concrete slab, respectively.

Table 6
Comparison of calculated values and test results

Component number	P_{ub}/kN	Mean value	μ_1	μ_2	μ_3	μ_4	δ_1
WJ-1	288.2	275.8	0.15	0.17	0.96	0.14	1.10
WJ-2	263.4						
WJ-3	423.4	419	0.21	0.24	1.34	0.20	0.80
WJ-4	414.6						
WJ-5	475.3	497.8	0.31	0.35	1.99	0.29	0.78
WJ-6	520.3						
AVE			0.22	0.25	1.43	0.21	0.90
STE			0.08	0.09	0.52	0.08	0.18

Note: P_{ub} represents the bearing capacity of the four-corner bent bar connector obtained from the test. δ_1 represents the ratio of the calculated value P_u' in Equation 6 to P_{ub} .

Table 6 shows a comparison between the theoretical values calculated by the proposed equations and the measured experimental data. The results indicate that the shear capacity calculated using Equation 6 is in good agreement with experimental data, with an average error of 10% and a standard deviation of 0.18. Therefore, the proposed calculation formula can be effectively utilized to predict the shear capacity of the new-type connector. The applicable range is as follows: $d = 8 - 18$ mm, $f_{cu}' = 30 - 70$ MPa, $f_{cu} = 30 - 70$ MPa, $f_{yk} = 300 - 500$ MPa and bending angle of 45° .

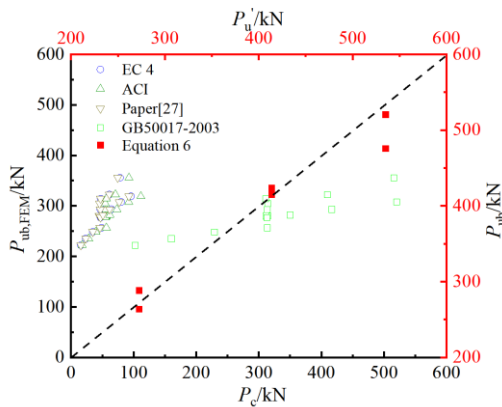


Fig. 19 Comparison of the simulated value of shear capacity of connector with the calculated value of national codes

6. Conclusions

1) The four-corner bent bar connector possesses relatively high initial stiffness and favourable ductility within the serviceability limit state. The characteristic slip values of all specimens vary from 6.04 to 11.48 mm, which meet the requirement of Eurocode 4 for flexible connectors (not less than 6 mm). Consequently, the four-corner bent bar shear connectors are categorized as flexible connectors.

2) The failure mode of every specimen was shear failure at the connection position between the four-corner bent bar shear connectors and the H-shape steel beam. Local concrete block detachment occurred at the welded interface of the composite slab and the steel beam.

3) Modifying the cross-sectional dimensions of the four-corner bent bar and material strength exerts varying degrees of influence on the shear bearing capacity of components. Among these factors, the steel bar diameter imposes the most prominent influence on the shear capacity, followed by the cast-in-place concrete compressive strength. In contrast, the precast concrete compressive strength has almost no effect. Recommendations for the optimal cross-sectional dimensions of the four-corner bent bar connectors are proposed with due consideration of shear performance, cost-effectiveness and structural detailing requirements.

4) Based on extensive numerical analysis results, the shear capacity calculation formula of four-corner bent bar shear connector is proposed through the method of multivariate linear regression. The theoretical predictions exhibit satisfactory agreement with measured test results.

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EFFECT OF POROSITY DEFECTS ON THE FATIGUE BEHAVIOR OF WELDED JOINTS IN STEEL BRIDGE

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ABSTRACT

Welding technology offers high efficiency and flexibility in the bridge construction. However, weld-included defects are inevitable and significantly reduce the fatigue performance of welded joints. Porosity is one of the most common welding defects and has a considerable influence on fatigue performance. In this study, both fatigue tests and numerical simulation are used to investigate the effect of porosity defects. Test specimens of butt-welded and cruciform-welded joints were designed, with 48 specimens divided into two groups, including 24 specimens with porosity defects and 24 control specimens without pores. The results indicate that the fatigue strength of butt-welded joints and cruciform-welded joints without porosity defects with 2 million cycles was 142.0 MPa and 92.3 MPa, respectively. When the pore depth and diameter were both 2.0 mm, the fatigue strengths of butt-welded joints and cruciform-welded joints with porosity defects decreased by 27.4% and 7.3%, respectively. Numerical simulation results showed that porosity significantly increased the stress concentration factor (SCF) of the welded joints. The diameter, position, and depth of porosity had clear effects on SCF. An increase in the pore diameter significantly elevated SCF, although the rate of increase diminished when the diameter exceeded 1.6 mm; the SCF decreased when the pore was located farther from the weld toe, with cruciform joints being more sensitive to pore position; surface pores caused significantly higher SCF than internal pores, and the SCF was higher when pore depth was closer to the weld surface.

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1. Introduction

Steel bridges have been extensively utilized in both railway and highway infrastructures, owing to their notable advantages such as high load-bearing capacity, relatively lightweight properties, and constructability [1-6]. Welding, which is an efficient and flexible joining method used in steel bridge construction, has significantly promoted the widespread use of steel bridges. But welded joints often encounter fatigue issues under cyclic stresses induced by traffic loads, primarily because of welding defects [7-13]. Investigations into several incidents of steel-bridge collapse have revealed that welding defects result in local stress concentrations [14-17]. Under repeated traffic loads, weld defects are more likely to be the main initiation sites for fatigue cracks in welded joints, significantly reducing their fatigue resistance and, in some cases, becoming a major cause of steel bridge failure [14,15].

Given the large number of welds in steel bridges, only a certain proportion are randomly inspected in accordance with the relevant standards, making it difficult to completely identify and eliminate weld defects in welded joints [16]. A typical photograph of porosity defects is shown in Fig. 1 in a welded joint of a steel bridge. Surveys of welded joints have identified a variety of welding defects, including undercutting at the weld toe, porosity, slag inclusion,

inadequate penetration and welding cracks [16-19]. Among these, porosity defects account for a relatively high proportion, according to survey results [16]. Porosity is one of the most common issues encountered during welding. It not only reduces the strength of welded joints, but also serves as a primary initiation site for fatigue cracks under cyclic loading, significantly reducing the fatigue life of welded joints.

Porosity defects are prevalent in the welded joints of steel bridges and exert a detrimental effect on their fatigue performance. Nevertheless, research dedicated to assessing the degradation of fatigue life induced by porosity remains limited, particularly in the realm of high-cycle fatigue behavior. Key characteristics of porosity, such as morphology, dimensions, and spatial distribution, govern the stress state and fatigue response of the welded joints. Consequently, a thorough investigation into the mechanisms through which porosity influences fatigue performance is imperative for enhancing the service life and structural safety of steel bridges. This study aims to elucidate the impact of porosity defects and their underlying mechanisms on the fatigue behavior of welded joints in steel bridges. By integrating experimental approaches with theoretical analysis, the effects of porosity characteristics, including morphology, size, and location, on the fatigue performance of welded joints were systematically examined.

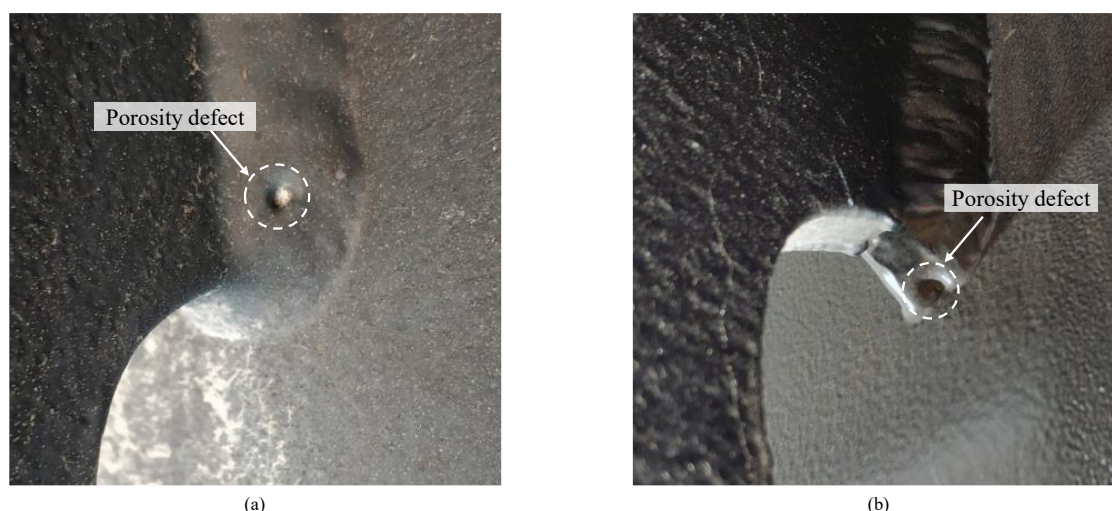


Fig. 1 The porosity in the welded joints of a steel bridge: (a) welded joint 1; and (b) welded joint 2

In recent years, the influence of porosity defects on the fatigue behavior of welded joints has been investigated by several researchers. For instance, Chen et al. [20] systematically examined the effect of porosity defects on the ultra-long fatigue life of aluminum alloy welded joints and demonstrated that such defects lead to a significant reduction in fatigue life. By combining experimental methods with finite element analysis, they further explored how porosity affects the stress concentration factor (SCF) of welded joints, revealing that the SCF at the pore location increases pronouncedly with pore diameter. Xu et al. [22] used synchrotron radiation X-ray micro-computed tomography to statistically analyze porosity defects in laser-hybrid welded aluminum alloy joints and found that the pore size and location significantly influenced the crack initiation and propagation paths. However, most of these studies focused on aluminum alloy welded joints, whereas research on welded joints of steel bridges is relatively scarce. The effect of porosity on the stress distribution in welded joints was revealed by the numerical analysis of Wu et al. [23], finding that SCF at the pore location increases significantly with increasing pore diameter. Luo et al. [24] experimentally studied the effect of porosity defects on the fatigue behavior of welded joints and observed that porosity considerably reduced the fatigue life. Nonetheless, comprehensive studies considering the combined influence of pore shape, size, and position on the fatigue performance remain limited.

To address this research gap, this study designed fatigue test models for butt- and cruciform-welded joints, including 48 specimens divided into four groups. Among these, 24 specimens contained porosity defects, whereas the remaining 24 which were without pores served as control specimens. Fatigue tests were performed under cyclic loading conditions to evaluate the impact of porosity on the fatigue life of welded joints. Furthermore, finite element analysis was employed to examine how porosity characteristics affect the stress distribution within the joints. By integrating experimental and numerical methods, this study provides a systematic evaluation of the influence of porosity defects on the fatigue performance of welded joints, thereby elucidating the underlying mechanisms through which these defects alter stress distribution.

2. Experiment

2.1. Specimen design

To delineate and quantify the detrimental influence of porosity defects, two representative welded joint types, butt joints and cruciform joints, were selected as the research objects. Four distinct categories of fatigue specimens were fabricated, each comprising 12 specimens, as detailed in Table 1. Group BW1 consisted of conventional butt-welded specimens, whereas Group BW2 consisted of butt-welded specimens with porosity defects. Group CW1 consisted of conventional cruciform-welded specimens, and Group CW2 consisted of cruciform-welded specimens having porosity defects. The butt-welded specimens were fabricated with dimensions of 300 mm (length) $\times$ 50 mm (width) $\times$ 8 mm (thickness). The cruciform-welded specimens measured 420 mm in length, 80 mm in height, with flange and web plates both 16 mm thick. Schematic diagrams and additional structural details of both specimen types are shown in Figs. 2 and 3. Specially, the width and surplus height of butt-welded seam are respectively 8 mm and 1.5 mm. The leg size of cruciform-welded seam equal to 8 mm. The tolerance of the weld size complies with the specification requirements [25].

Table 1
Specimen categories

Group number	Group type	Porosity depth $h(\text{mm})$	Porosity radius $\rho(\text{mm})$	Number
BW1	Without porosity defects	/	/	12
BW2	With porosity defects	2.0	2.0	12
CW1	Without porosity defects	/	/	12
CW2	With porosity defects	2.0	2.0	12

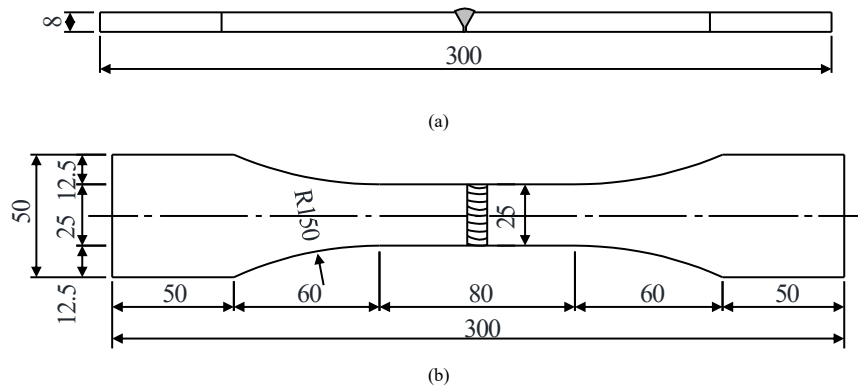


Fig. 2 Sketch of butt-welded joint specimens (unit: mm): (a) side view; (b) top view

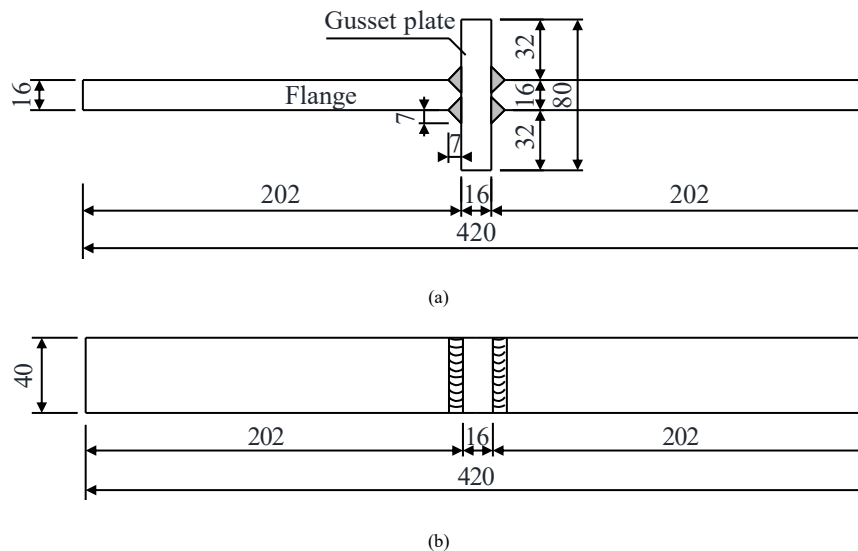


Fig. 3 Sketch of cruciform-welded joint specimens (unit: mm): (a) side view; (b) top view

Welding defects in steel bridges demonstrate marked stochasticity in their type, location, and geometry. Research on the fatigue performance of welded joints typically employs a dichotomy of defect types: natural and artificial [26–29]. A significant methodological challenge is posed by the fact that while natural defects offer superior representativeness of in-service structures, their deliberate generation under controlled laboratory conditions remains problematic. Moreover, the size and location of natural weld defects tend to be highly variable and random, complicating quantitative research. In contrast, artificial defects can be deliberately introduced and precisely positioned in critical areas of the welded joints, facilitating quantification of the effects of fatigue degradation caused by welding defects. Given the validated effectiveness of artificial defects in simulating realistic conditions [27–29], controlled artificial porosity was introduced in this study to evaluate its impact on the fatigue behavior of welded joints.

The specific requirements for the surface porosity defects according to the EN ISO 5817 standard [25] are listed in Table 2. For weld-quality level B, porosity defects are not permitted. For weld quality level C, when the plate thickness t is more than 3 mm, the pore diameter d is less than $0.2s$ ($d \leq 2\text{mm}$). For weld quality level D, the pore diameter d is less than $0.3s$ ($d \leq 3\text{mm}$) when t is more than 3 mm. When designing defect-bearing specimens, the defect dimensions were determined by combining the quality standards for weld defects from EN ISO 5817 and the findings from field investigations of steel bridge weld defects. The diameter and depth of the artificial porosity defects were set to 2.0 mm. All specimens were fabricated using welding procedures consistent with actual bridge construction. A set of 12 butt-welded specimens and 12 cruciform-welded specimens were engineered to contain artificial porosity defects within their welds. A schematic and a photograph of the porosity defects in the specimens are shown in Fig. 4, where the porosity defects were made by using the electric drill. Compared with laser or welding process manipulation, the size of artificial defects is more easily controlled. The porosities are located at the middle weld toe near to the weld zone.

2.2. Materials and fabrication

The steel materials and welding procedures used to fabricate the specimens are consistent with those used for actual bridge structures. All the specimens were made of Q345qE bridge steel with a nominal yield strength of 345 MPa. The chemical composition and mechanical properties of Q345qE steel are listed in Tables 3, respectively. CO₂ gas-shielded arc welding was employed with parameters set at 220–280 A (current), 27–33 V (voltage), and a 15–25 L/min shielding gas flow rate.

For butt-welded joints, a “V-shaped” double-sided groove was used, and a residual weld reinforcement was retained after welding. For cruciform-welded joint specimens, full-penetration welding was performed with a weld leg of size 7 mm. All the operations, including plate cutting and specimen welding, were performed indoors. Before welding, all shop primers and contaminants in the weld zones were removed. The specimens were fabricated as integrated welded plates and subsequently cut into individual specimens using wire electrical discharge machining (EDM), as shown in Fig. 5. The weld toes of the specimens were ground with sandpaper to reduce the stress concentration effect caused by surface unevenness.

Table 3
Chemical compositions of Q345qE bridge steel (%)

Thickness (mm)	C ($\times 10^{-2}$)	Si ($\times 10^{-2}$)	Mn ($\times 10^{-2}$)	P ($\times 10^{-3}$)	S ($\times 10^{-3}$)	Als ($\times 10^{-3}$)	Ni ($\times 10^{-3}$)	Ti ($\times 10^{-3}$)
8	11	24	142	10	3	38	17	2
16	10	28	146	13	2	34	10	14

Table 2
Limits for porosity defects [29]

Plate thickness	Limits for imperfections for quality levels		
	D	C	B
$t > 3.0$	$d \leq 0.3s$ and up to 3.0mm	$d \leq 0.3s$ and up to 2.0mm	Not allowed

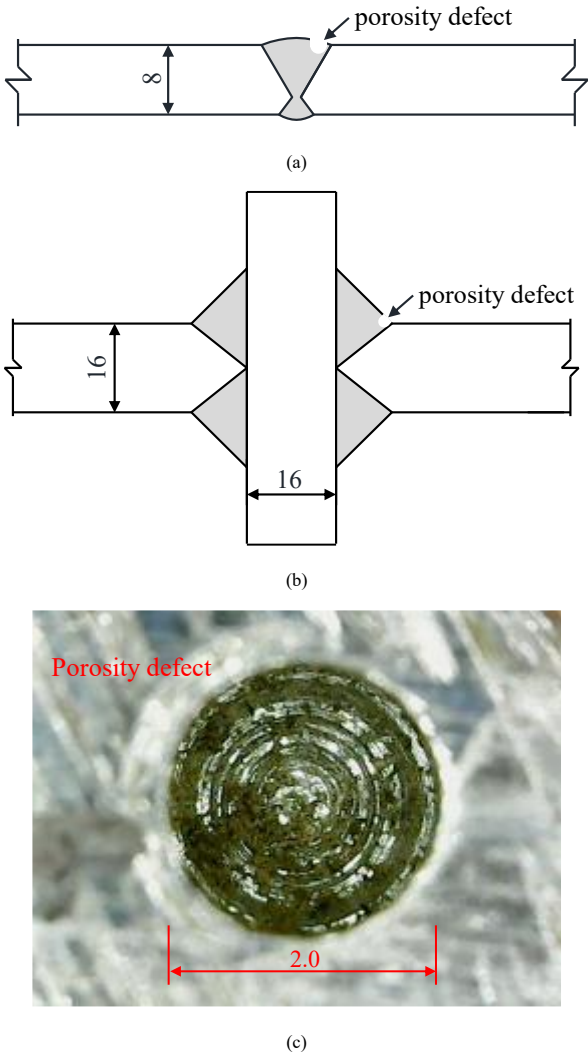


Fig. 4 Sketch of porosity (unit: mm): (a) butt-welded specimens; (b) cruciform-welded specimens; and (c) photograph of porosity defects

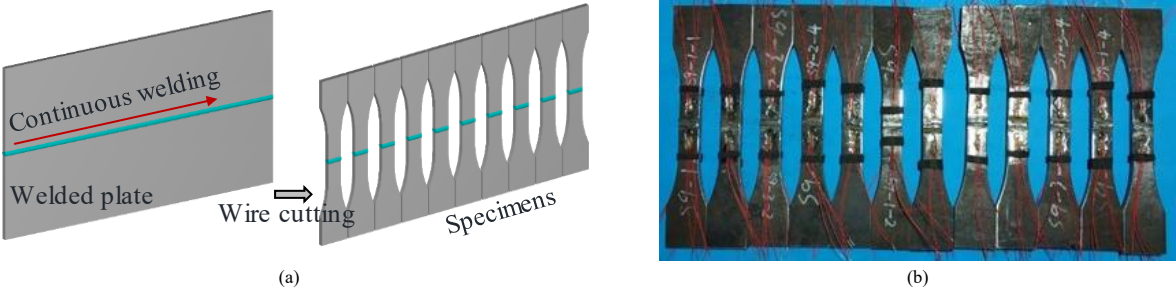


Fig. 5 Sketch of specimens manufacturing: (a) specimens manufacturing; and (b) photograph of the specimens

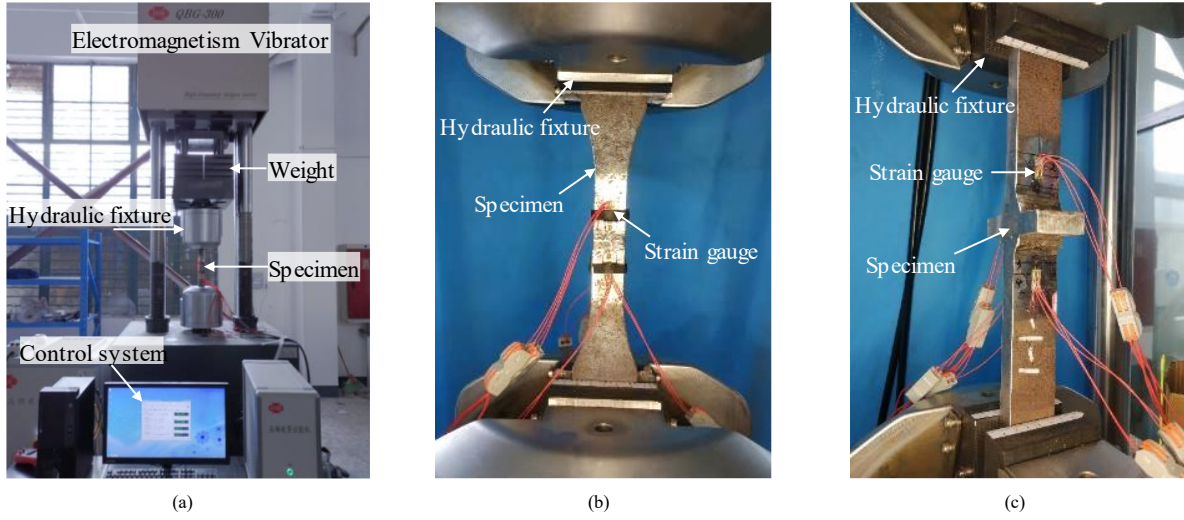


Fig. 6 Test setup: (a) Layout; (b) butt-welded joint; and (c) cruciform-welded joint

2.3. Test setup, loading scheme, and instruments

A QBG-300 electromagnetic resonance fatigue testing machine, operating under force control with a sinusoidal waveform (68–120 Hz, Fig. 6), was used to apply constant amplitude cyclic loading. The specific stress range and loading frequency for each specimen are provided in Section 3.2. The maximum loading cycle limit for each specimen was set to 5×10^6 cycles. If a specimen did not fracture after exceeding 5×10^6 cycles, a new specimen was tested with increased stress to continue the experiment. A Vernier caliper and an electronic microscope were used to measure the width, plate thickness, weld dimensions, and pore sizes of the specimens.

The experimental loading scheme is presented in Fig. 7. Prior to fatigue testing, static load tests were conducted to verify the functionality of all equipment and to determine the appropriate stress range for each specimen. Strain gauges were strategically positioned at locations susceptible to crack initiation, with eight gauges in total affixed to the top and bottom surfaces to measure hot-spot stresses at distances of $0.4t$ and $1.0t$ from the weld toe (where t denotes the plate thickness). The specific locations and designations (B1–B4) for these four measurement zones are illustrated in Fig. 8. The strain gauges located at $0.4t$ and $1.0t$ from the weld toe were denoted by “04” and “10,” respectively. For example, “B1-04” refers to the strain gauge positioned at $0.4t$ from the weld toe in the measurement zone B1.

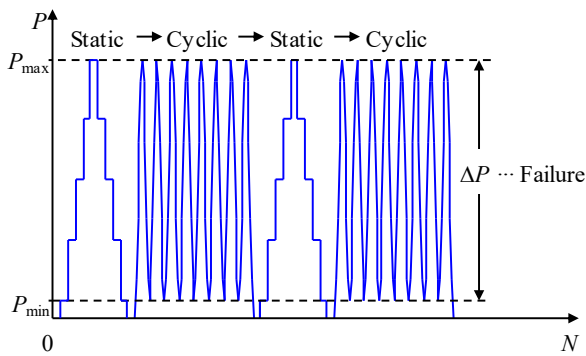


Fig. 7 Fatigue loading procedure

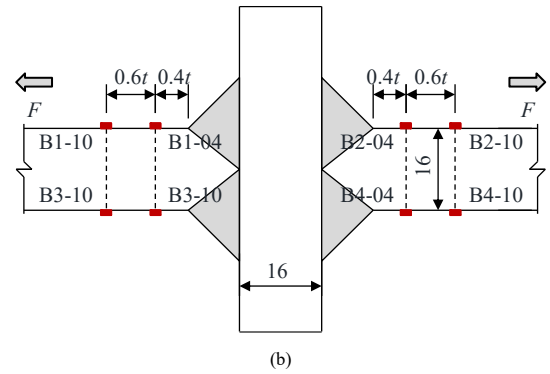
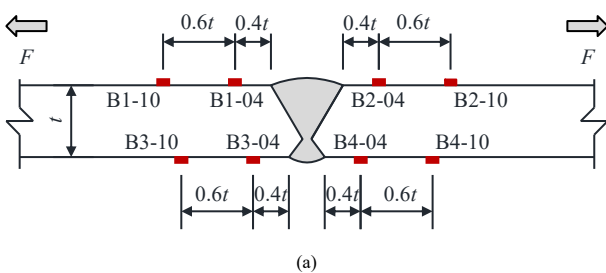


Fig. 8 Location of strain gauges (unit: mm): (a) BW groups; and (b) CW groups

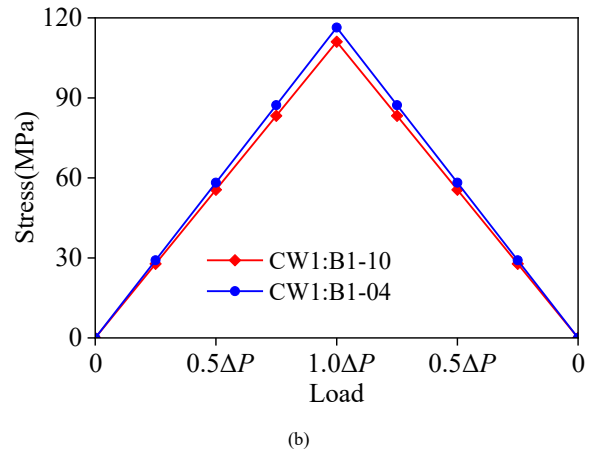
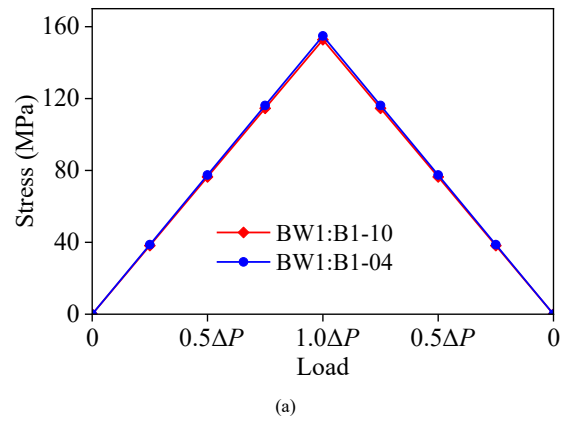


Fig. 9 Stresses of the specimens: (a) butt-welded specimen; and (b) cruciform-welded specimen

3. Experimental results and discussion

3.1. Initial static test results

The static tests validated the experimental system's reliability, as evidenced by the linear and symmetric stress-load responses shown in Fig. 9 (tension positive). Moreover, the measured stresses at points B1-04 and B1-10 were nearly identical for both joint configurations, implying a minimal stress gradient in the critical weld toe area.

3.2. Fatigue test results

3.2.1. *S-N curve fitting method*

A standard methodology for evaluating fatigue life involves the construction of an *S-N* curve. This study derived the *S-N* curve for the butt-welded joints by subjecting the experimental data to linear regression analysis on a log-log plot, consistent with the typical power-function representation for welded details:

$$\Delta\sigma^m N = C \quad (1)$$

where N represents the number of cycles to failure, $\Delta\sigma$ is the applied stress range, C is a material constant related to fatigue strength, and m is the slope of the *S-N* curve plotted on a log-log scale.

Taking the logarithm of both sides of Eq. (1) yields the logarithmic form expressed in Eq. (2).

$$\lg N = \lg C - m \lg \Delta\sigma \quad (2)$$

The *S-N* curves were evaluated through regression analysis. The slope m was set to 3, as stipulated by the governing standards [29-32]. It is imperative to clarify that the failure cycle represents the critical point of structural failure under cyclic loading. Conversely, the total failure life accounts for the entire

fatigue process, comprising both the initiation and subsequent propagation of a crack to a critical size.

3.2.2. *Test results*

Summary of the fatigue test data from 19 specimens (17 valid) is presented in Table 4. Despite a common stress ratio, individual specimens were subjected to different stress ranges and loading frequencies. The *S-N* relationship was established by applying a linear regression to the log-log plot of nominal stress range $\Delta\sigma$ against failure cycles (N), as shown in Fig. 10. The corresponding *S-N* curves for the BW1 and BW2 groups are defined by Eqs. (3) and (4).

$$\lg N = 12.758 - 3.000 \lg \Delta\sigma \quad (3)$$

$$\lg N = 12.341 - 3.000 \lg \Delta\sigma \quad (4)$$

Fatigue assessment according to Eurocode 3 assigns a detail category of 90 to the tested butt-welded joints, indicating a reference stress range of 90 MPa at 2 million cycles. The experimental *S-N* curves for groups BW1 and BW2, determined from the data in Table 4 using the nominal stress method (Fig. 10), provide a basis for comparison with this standard. The run-out values are conservatively regarded as the failure cycle. The fatigue test results for the BW1 group showed a fatigue resistance of 142.0 MPa after 2 million cycles, which was 57.8% higher than the standard value of 90.0 MPa specified in Eurocode 3. This indicates that the standard value is relatively conservative when estimating the fatigue resistance of butt-welded joints.

Evaluation using the fixed-slope fitting equation yielded an equivalent nominal stress fatigue strength of 103.1 MPa at 2 million cycles for the BW2 group. The primary failure mode initiated at porosity defects, with crack propagation through the plate thickness leading to fracture. This resulted in a 27.4% decrease in fatigue strength relative to defect-free specimens, indicating that porosity substantially compromises the fatigue strengths of butt-welded joints.

Table 4
Fatigue test parameter and results of specimens

Specimens	Weld size	Stress range (MPa)	Failure cycle ($\times 10^4$ cycles)	Defect/ Location	Fracture mode
BW1-1	Butt	120.0	>500.00	No	Run-out
BW1-2	Butt	130.0	416.91	No	Weld toe
BW1-3	Butt	140.0	278.00	No	Weld toe
BW1-4	Butt	150.0	211.17	No	Weld toe
BW1-5	Butt	160.0	166.11	No	Weld toe
BW1-6	Butt	170.0	64.78	No	Weld toe
BW1-7	Butt	180.0	89.10	No	Weld toe
BW1-8	Butt	190.0	117.14	No	Weld toe
BW1-9	Butt	200.0	32.14	No	Weld toe
BW2-1	Butt	120.0	102.91	Porosity	Porosity
BW2-2	Butt	130.0	124.52	Porosity	Porosity
BW2-3	Butt	140.0	115.39	Porosity	Porosity
BW2-4	Butt	150.0	58.18	Porosity	Porosity
BW2-5	Butt	160.0	37.12	Porosity	Porosity
BW2-6	Butt	170.0	43.83	Porosity	Porosity
BW2-7	Butt	180.0	69.19	Porosity	Weld toe
BW2-8	Butt	180.0	37.95	Porosity	Porosity
BW2-9	Butt	190.0	31.87	Porosity	Porosity
BW2-10	Butt	180.0	30.65	Porosity	Porosity
CW1-1	Cruciform	90.0	368.52	No	Weld toe
CW1-2	Cruciform	100.0	202.84	No	Weld toe
CW1-3	Cruciform	110.0	90.65	No	Weld toe
CW1-4	Cruciform	120.0	119.06	No	Weld toe
CW1-5	Cruciform	130.0	53.06	No	Weld toe
CW1-6	Cruciform	140.0	52.96	No	Weld toe

CW1-7	Cruciform	150.0	35.63	No	Weld toe
CW1-8	Cruciform	190.0	19.89	No	Weld toe
CW2-1	Cruciform	100.0	160.52	Porosity	Porosity
CW2-2	Cruciform	110.0	181.28	Porosity	Weld toe
CW2-3	Cruciform	120.0	97.97	Porosity	Weld toe
CW2-4	Cruciform	130.0	83.91	Porosity	Weld toe
CW2-5	Cruciform	140.0	41.55	Porosity	Weld toe
CW2-6	Cruciform	150.0	36.60	Porosity	Weld toe
CW2-7	Cruciform	160.0	31.97	Porosity	Porosity
CW2-8	Cruciform	160.0	21.07	Porosity	Weld toe
CW2-9	Cruciform	170.0	18.49	Porosity	Porosity

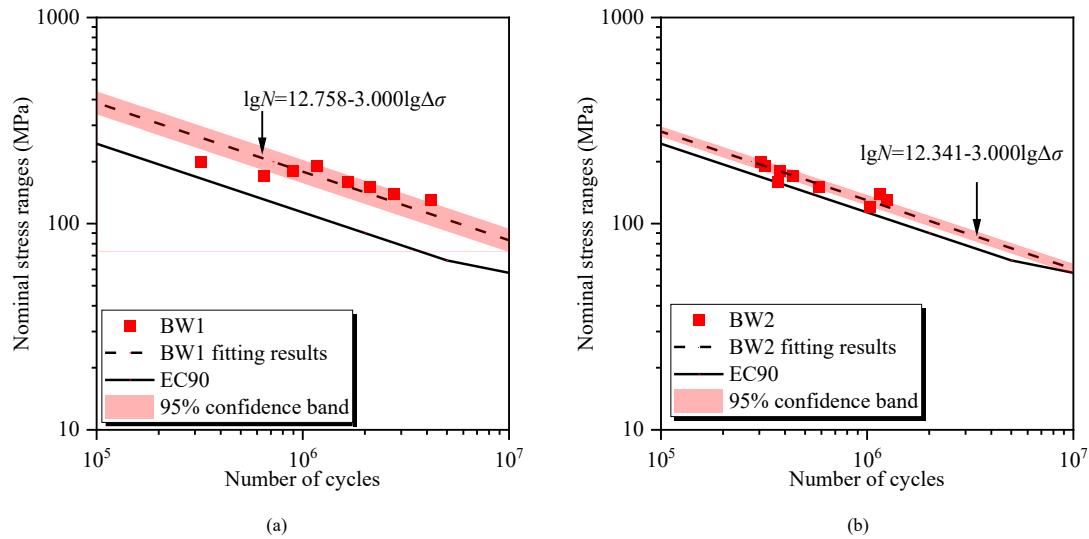


Fig. 10 Fatigue test data and the S - N curves: (a) BW1 group; (b) BW2 group

Through fatigue testing of cruciform-welded joint specimens, 11 valid fatigue test data points were obtained a total of 9 fatigue tests were conducted on cruciform joints with porosity defects, among which 6 specimens exhibited crack initiation and propagation from the weld toe. Only 3 specimens showed crack initiation at the porosity defect and propagation through the plate thickness. A summary of failure cycles and fracture locations is provided in Table 4, and the S - N data are plotted in Fig. 11. To obtain the S - N curves, a linear fit was performed on the nominal stress range $\Delta\sigma$ and the number of failure cycles N for each specimen. The nominal stress S - N curves for the CW1 group, CW2 group initiating at the weld toe and initiating at the porosity are

expressed by Eqs (5), (6) and (7), respectively.

$$\lg N = 12.197 - 3.000 \lg \Delta\sigma \quad (5)$$

$$\lg N = 12.161 - 3.000 \lg \Delta\sigma \quad (6)$$

$$\lg N = 12.094 - 3.000 \lg \Delta\sigma \quad (7)$$

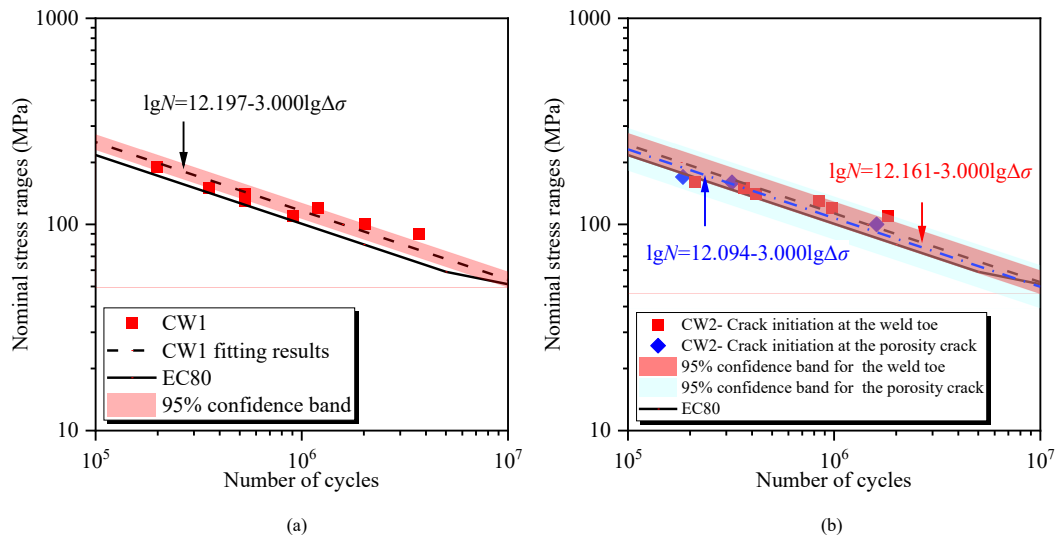


Fig. 11 Fatigue test data and S - N curves: (a) CW1 group; (b) CW2 group

On the basis of the fatigue test results (Table 4), the S-N relationships for the CW1 and CW2 specimen groups were constructed via the nominal stress method, with the results depicted in Fig. 11. According to the standard, EC 80 corresponds to a nominal stress range of 80 MPa for 2 million cycles. The fatigue test results for the CW1 group were all higher than the standard value of 80 MPa, with a fatigue resistance of 92.3 MPa at 2 million cycles, which was 15.4% higher than the value specified in Eurocode 3.

For the CW2 group specimens, the average nominal stress fatigue strength at 2 million cycles for specimens where cracks initiated at the porosity defect was 85.5 MPa, representing a 7.3% reduction compared to the CW1 group comprising conventional cruciform joint specimens. For specimens where

cracks were initiated at the weld toe, the average nominal stress fatigue strength at 2 million cycles was 90.3 MPa, which is close to the average value of 92.3 MPa observed in the CW1 group.

The fatigue failure of BW1 specimens, characterized by a loss of load-bearing capacity, is attributed to crack initiation and growth. As shown in Fig. 12, cracks predominantly initiate at the weld toe. The resulting fracture surface morphology correlates with the initiation location: semi-elliptical cracks correspond to mid-section initiation (Fig. 12b, d), while quarter-elliptical cracks correspond to weld-end initiation (Fig. 12c). The observation of multiple initiation zones (Fig. 12d) and varied initiation sites among specimens underscores the inherent randomness in the crack nucleation phase.

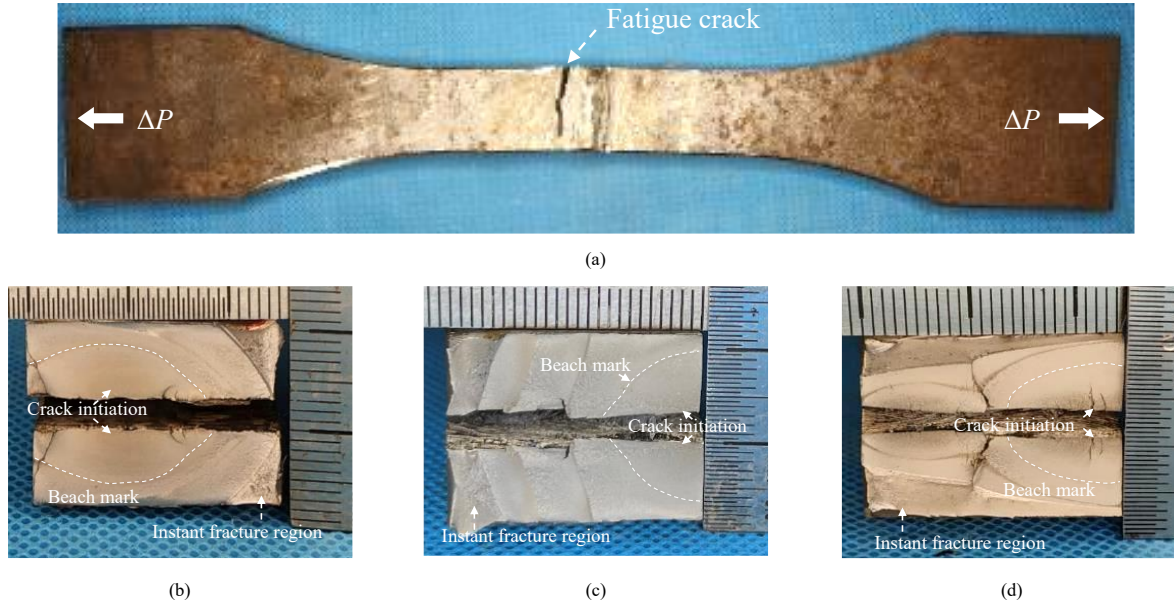
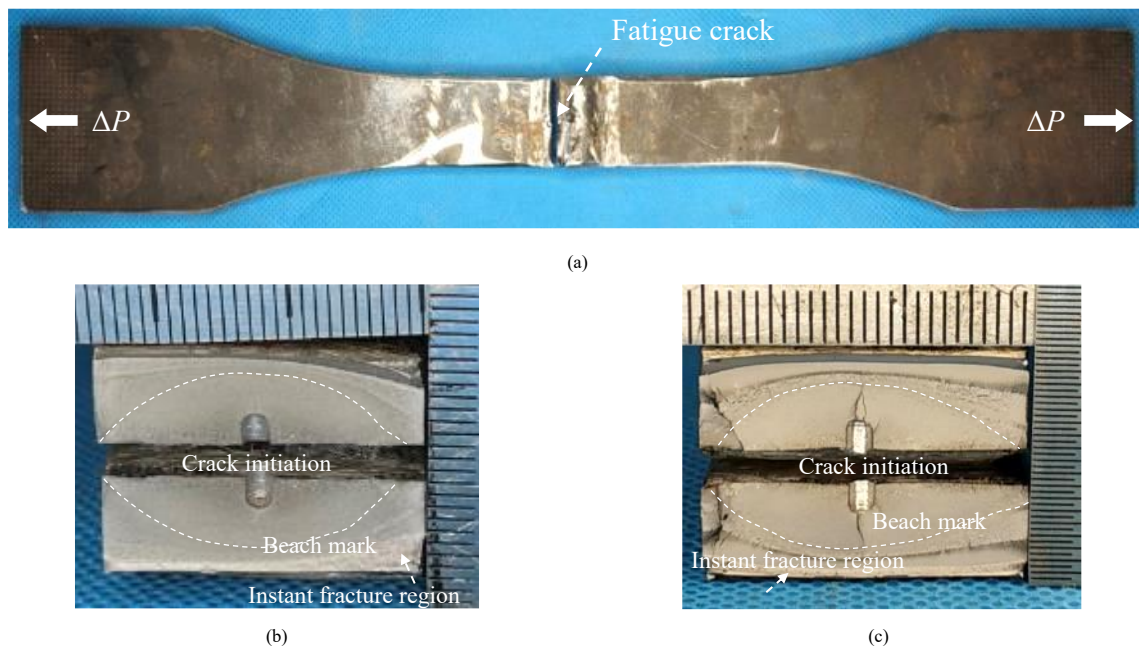


Fig. 12 Fatigue crack of the BW1 group: (a) failed welded joint; (b) failure section 1; (c) failure section 2; (d) failure section 3

A key distinction observed in the fracture morphology of the BW2 specimens (Fig. 13) is that fatigue crack initiation was primarily associated with porosity defects, a contrast to the behavior observed in the BW1 group. The fracture surfaces predominantly exhibited elliptical cracks, all showing typical fatigue fracture features. The fatigue crack propagation zones were smooth and flat, while the final fracture zones were rough and exhibited tearing patterns. Fatigue cracks initiated at the fusion line between the defect and weld metal and propagated through the plate thickness, with radial striations observed on the fracture surfaces. The macroscopic fracture sections of the specimens are shown in Fig. 13(b), (c) and (d). The fatigue crack propagation zones appeared darker,

with clear striations on the fracture surface, and obvious tearing secondary cracks were present. The fatigue fracture morphology of specimen observed under a Scanning Electron Microscope (SEM) was shown in Fig. 13(e). Fatigue cracks originate at the fusion line between the porosity defect and the weldment in the specimen, and extend towards the plate thickness direction. There are outward radiating streaks on the fracture surface. The crack extension area is darker in color, and the bands on the fracture surface are more distinct, with obvious tear-like secondary cracks present. Overall, the failure process is similar to that of actual pores causing crack initiation and propagation. The results are therefore representative to a certain extent.



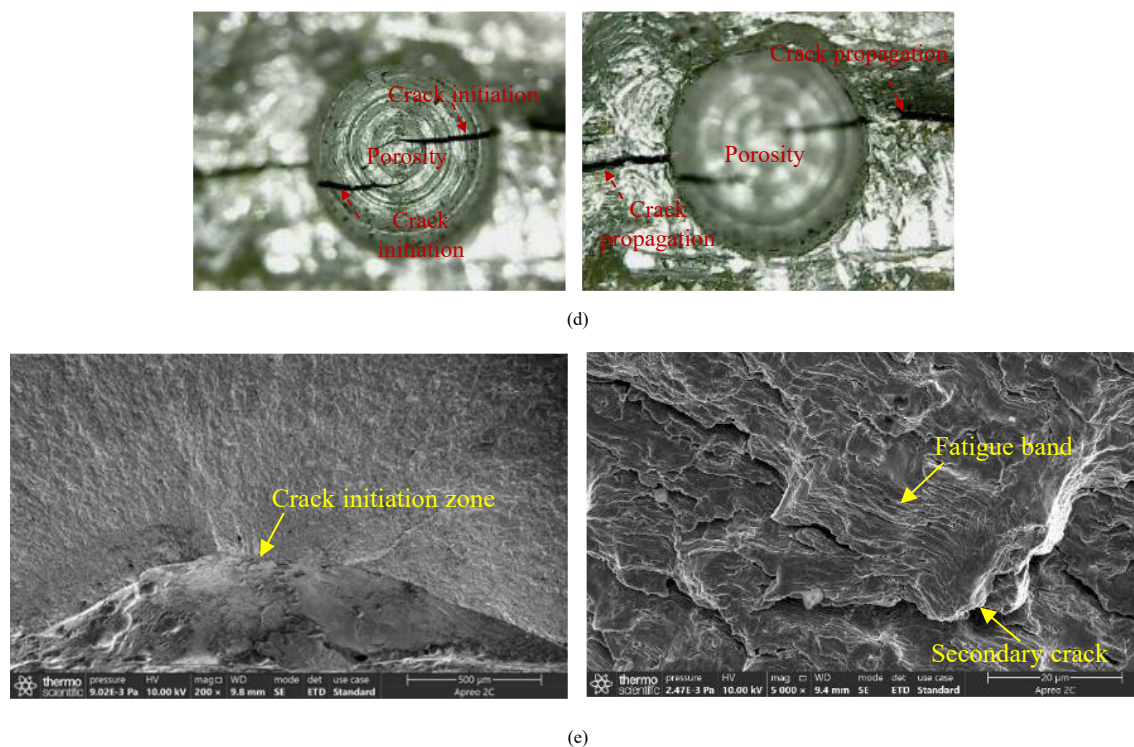


Fig. 13 Fatigue crack of the BW2 group: (a) failed welded joint; (b) failure section 1; (c) failure section 2; (d) location of crack initiation; (e) SEM morphology of crack initiation and propagation zone

Figs. 14 and 15 show the typical morphology of the fatigue failure in the CW1 and CW2 groups, respectively. Fatigue cracks were generally initiated at the weld toe where the stress was concentrated. The fatigue tests on CW1 specimens indicated crack initiation at the weld toe and subsequent through-thickness propagation, leading to elliptical fracture surfaces and, notably, multiple initiation zones as shown in Fig. 14(c). For the CW2 group specimens, when there was no overlap between the porosity defect and the weld toe line,

fatigue cracks initiated from the weld toe. However, when the porosity defect intersected the weld toe line, cracks initiated at the location of the porosity defect. After initiation, the cracks propagated primarily through the plate thickness with predominantly elliptical crack patterns. All fracture surfaces displayed typical fatigue fracture features, where the fatigue crack propagation zones were smooth and flat, whereas the final fracture zones were rough with tearing characteristics, as shown in Fig. 15.

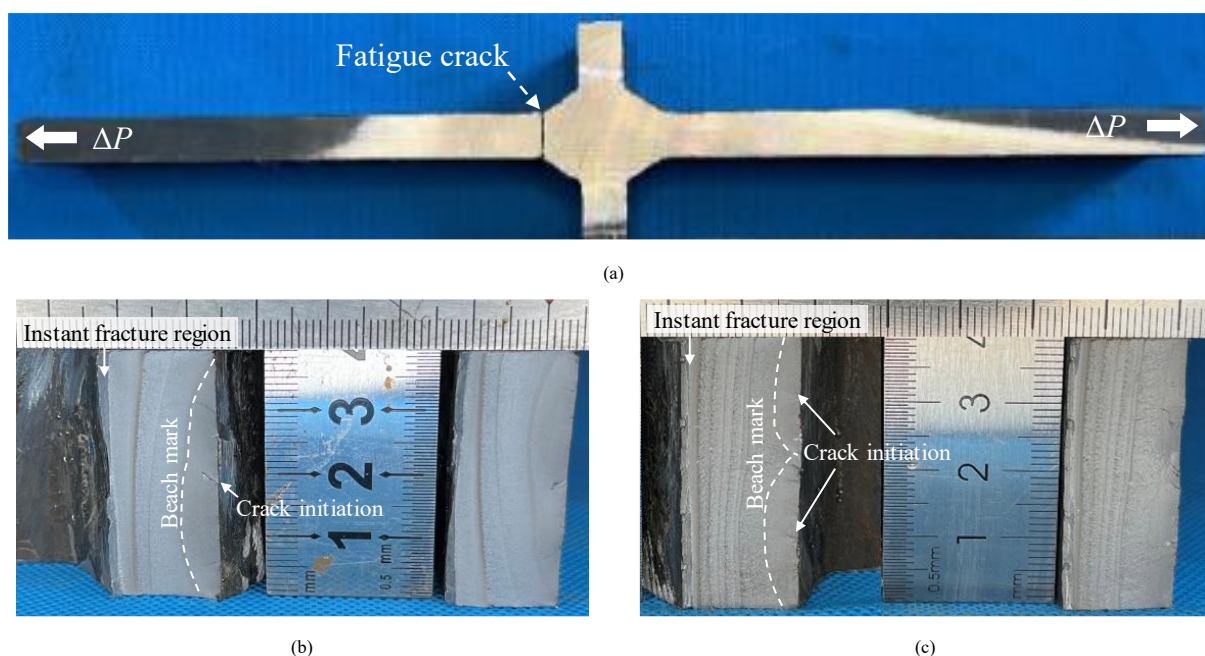


Fig. 14 Fatigue failure of the CW1 group: (a) failed welded joint; (b) failure section 1; (c) failure section 2

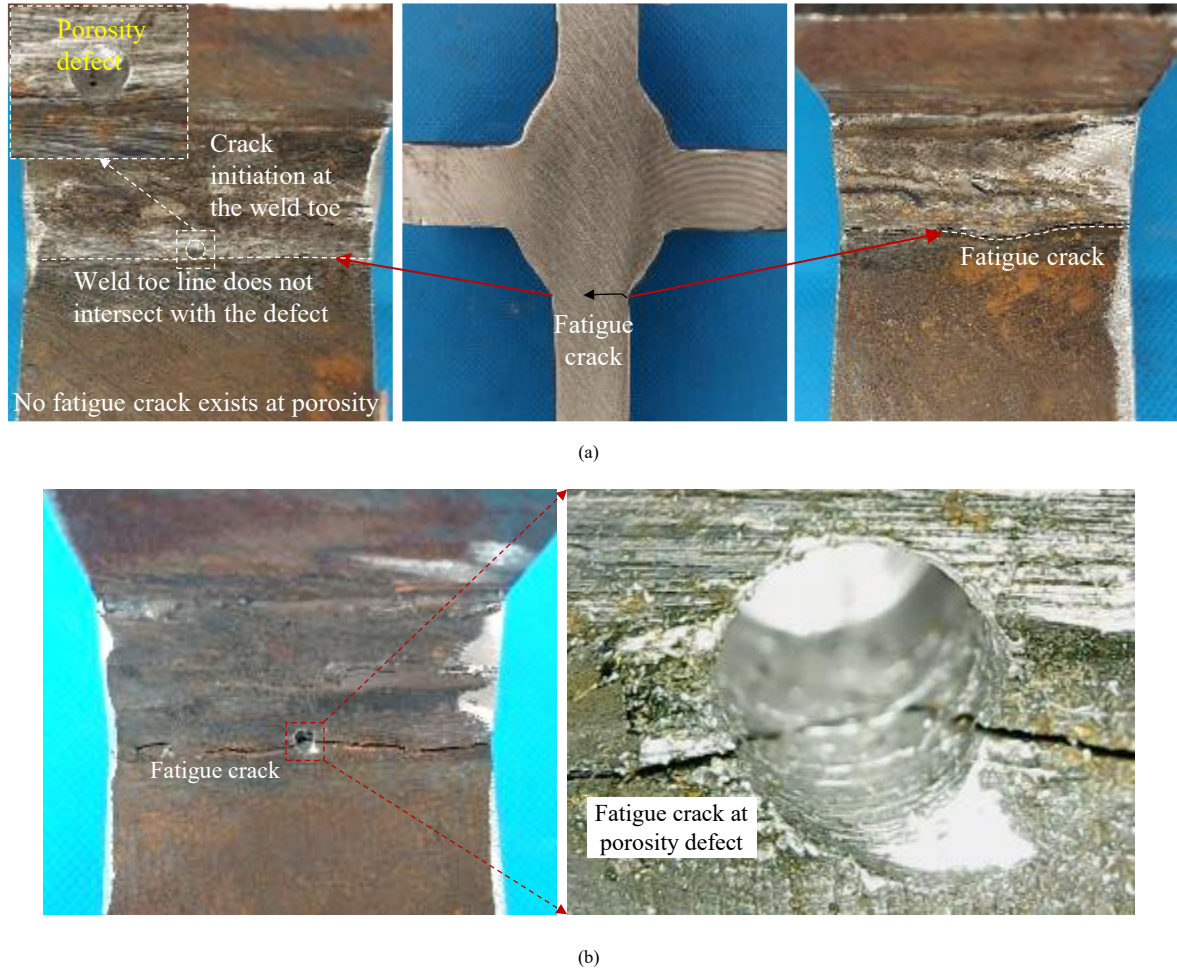


Fig. 15 Fatigue failure of the CW2 group: (a) initiation at the weld toe; (b) initiation at the porosity defect

3.2.3. Fatigue strength analysis

Butt-welded joints demonstrated superior fatigue strength under nominal stress compared to cruciform-welded joints in the absence of defects (Table 5). Nevertheless, under the condition of 2 mm pore defects, their fatigue resistance decreased significantly by 27.4%, a reduction substantially greater than the 7.3% observed for cruciform-welded joints, highlighting the critical influence of joint geometry on defect sensitivity. These findings indicate that porosity defects significantly affect the fatigue performance of welded joints.

Table 5
Comparison of fatigue strength

Group number	Group types	Fatigue resistance at 2 million cycles (MPa)	Failure locations
BW1	without porosity defects	142.0	Weld toe
BW2	with porosity defects	103.1	Porosity
CW1	without porosity defects	92.3	Weld toe
CW2	with porosity defects	90.3	Weld toe
CW2	with porosity defects	85.5	Porosity

4. Numerical study

4.1. Finite element model

A series of finite element models were developed within the ABAQUS environment to analyze stress distributions in the welded joints (Fig. 16). The geometry was meshed with C3D8 solid elements. Subsequently, a high level of mesh refinement, achieving an element size of 0.2 mm near the weld toe, was refined to enhance the accuracy of the finite element analysis. The butt-welded model consisted of approximately 65,700 elements, whereas the cruciform-welded model contained approximately 235,500 elements. The boundary conditions were consistent with those used in the fatigue tests. The translational degrees of freedom at one clamped end of the specimen were fully restrained, and cyclic loads were applied to the clamped region at the opposite end,

following the loading values defined in the experimental scheme, to simulate the fatigue loading.

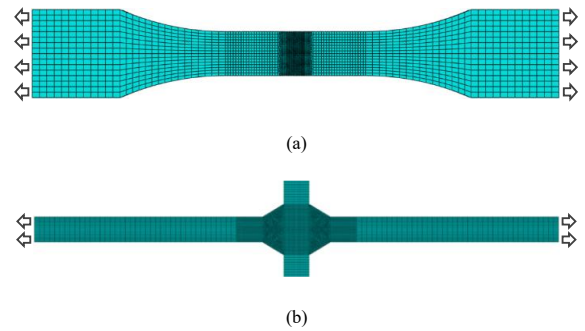


Fig. 16 The finite element model of the welded joints: (a) butt-welded joints; and (b) cruciform-welded joints

4.2. Model validation

The accuracy of the finite element model was validated through a comparison with static load test results. Two representative measurement points (B1-04 and B1-10) within the B1 strain measurement zone were selected, and their experimental data were compared with the results of the finite element analysis, as shown in Fig. 17. The results showed that for the butt-welded joint, the average deviation between the experimentally measured stress amplitudes and the finite element results at the two representative points was 1.4%. For the cruciform-welded joint, the average deviation in the strain amplitude between the experimental measurements and results of the finite element analysis at the two points was 3.1%. The close correlation observed between the finite element predictions and the experimental measurements serves to affirm the reliability and accuracy of the computational model.

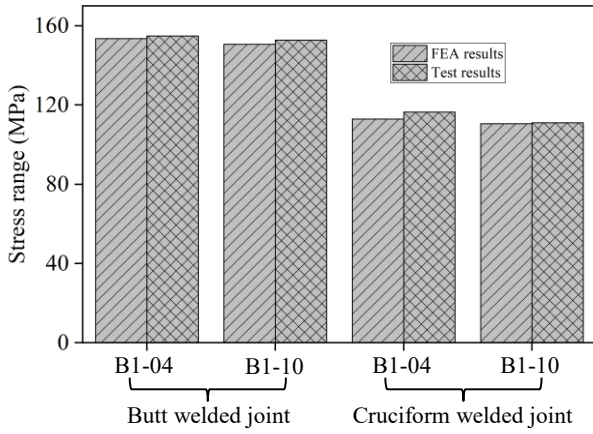


Fig. 17 Comparison of test results and numerical results

4.3. Parametric study and results

The influence of porosity defects on the stress state of welded joints was investigated through a parametric study using finite element modeling, focusing on key defect characteristics, such as pore diameter, pore depth, and pore position, on the stress state of the welded details. First, refined finite-element models were established based on the design drawings of the two types of welded joints used in the fatigue tests. During modeling, the welded joints were assumed to be homogeneous and continuous, whereas the porosity defect was modeled as a hollow sphere. The dimensions of the finite element model were consistent with those of the actual specimens. The modeling procedure and parameter settings were the same as those described in Section 4.1 and are therefore not repeated here. A far-field load was applied with nominal stress $\sigma_{nom}=100$ MPa as the control parameter. The effect of porosity defects on stress concentration was evaluated using the stress concentration factor, k_t , which is the ratio of the local maximum tensile stress at the pore to the nominal stress.

According to EN ISO 5817, the diameter d of the porosity defects should be limited to 3 mm. Therefore, the pore diameter parameter was selected accordingly, with a minimum pore diameter of 0.4 mm and a maximum of 3.0 mm. The center of the pore was aligned with the weld surface in the depth direction, and the pores were located on the weld near the weld toe. For consistency, the same pore diameter parameters were applied to both the butt-welded and cruciform-welded joints. To eliminate interference from other factors, only the pore diameter was varied during the study, whereas the other parameters remained unchanged. Plotted in Fig. 18 are the stress concentration factors corresponding to various porosity diameters for the butt- and cruciform-welded joints.

The results indicate that both joint types exhibit similar trends in the variation of k_t with the pore diameter. As the pore diameter increased, the stress concentration factor initially increased significantly and then gradually leveled off. Once the pore diameter reached 1.6 mm, the rate of increase in k_t began to decrease and eventually stabilized. The stress concentration factors for the butt- and cruciform-welded joints were 1.13 and 1.40 at a 0.4 mm pore diameter, with corresponding increases of 39.7% and 24.6% observed when the diameter was raised to 3.0 mm.

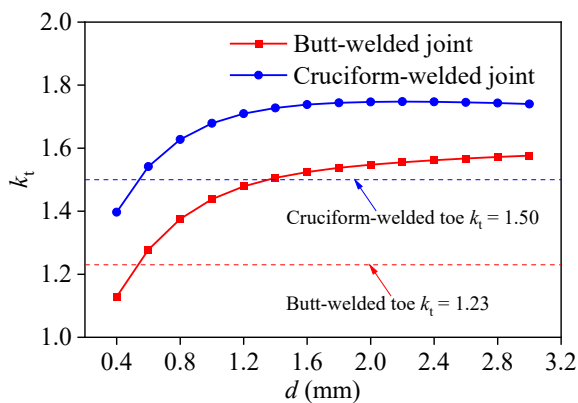


Fig. 18 Curve of stress concentration factor of welding details vs. pore diameter parameter

To accurately describe the location of the porosity defect in the finite element model, the pore center was assumed to be located on the weld surface, with the distance from the pore center to the weld toe defined as L . This distance parameter L was used to characterize the pore location, as illustrated in Fig. 19. The pore diameter d was set to 2 mm and all other model parameters were the same as those previously described.

The influence of porosity location L on the stress concentration factor is shown in Fig. 20. For both joint types, the stress concentration factor decreased significantly with increasing L , implying a weaker effect with greater distance from the weld toe. The cruciform-welded joint was considerably more sensitive to this parameter. As L increased from 1.75 mm to 8.50 mm, the stress concentration factor decreased by 19.9% (1.64 to 1.31) for the butt-welded joint, in contrast to a 51.3% decrease (1.96 to 0.96) for the cruciform-welded joint.

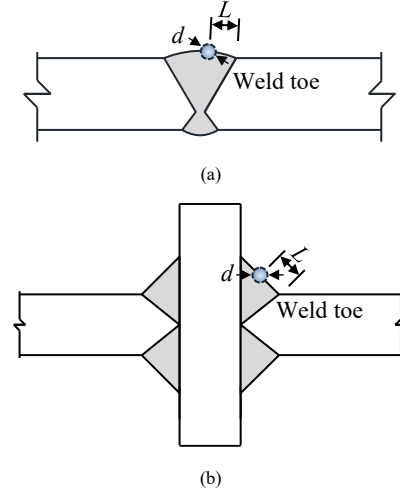


Fig. 19 Positional relationship between pores and weld toes

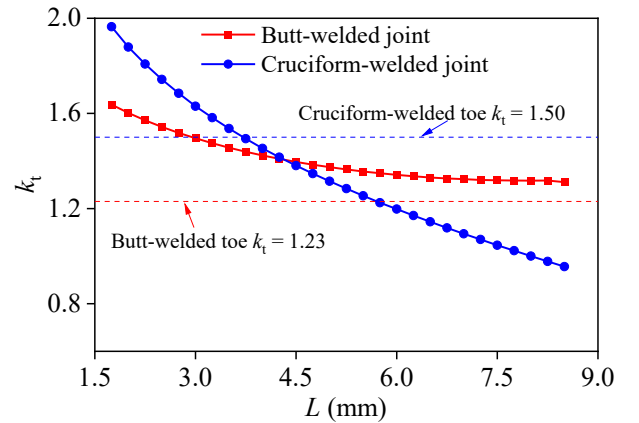


Fig. 20 Curve of stress concentration factor of welding details vs. pore location

To investigate the stress distribution characteristics of the welded details at different pore depths, a butt-welded joint was selected as an example, and the stress distributions at various pore depths were analyzed. The distance between the pore center and weld surface is defined as s , and the pore diameter is defined as d . The ratio s/d was used to describe the pore depth. A schematic diagram of the pore depth within the weld is shown in Fig. 21.

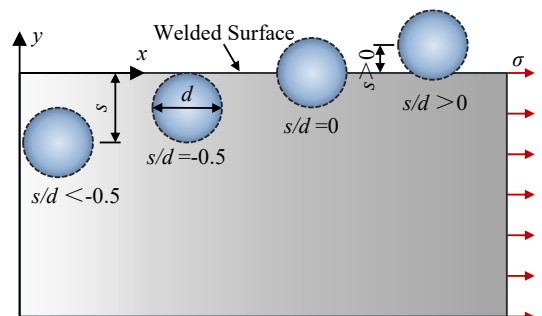


Fig. 21 Positional relationship between pores and the weld surface

In this context, $s/d = -0.5$ indicates that the pore is tangential to the weld surface; when $s/d < -0.5$, the pore is considered an internal pore; and when $s/d > -0.5$ it is defined as a surface pore. In the finite element model, the pore diameter d was set to 2 mm, and all other modeling parameters were kept consistent with those previously described.

Fig. 22 shows the contour plots of the first principal stress σ_1 at the pore defect location in the butt-welded joint specimen under tensile loading conditions. Figs. 22(a) to 22(d) correspond to the stress fields on a plane

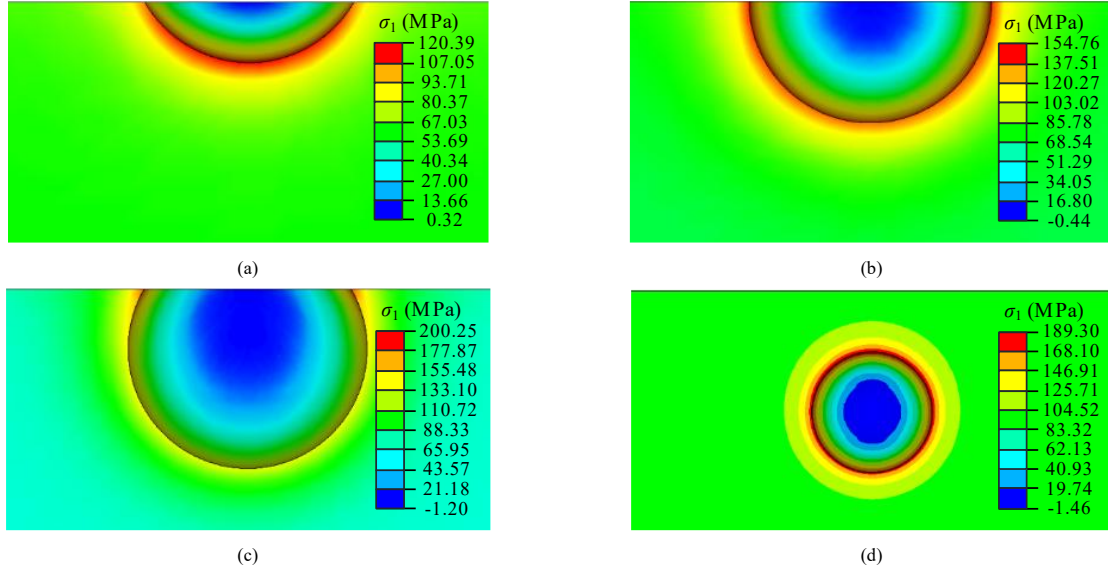


Fig. 22 Stress nephogram of the position of porosity defects in butt joints: (a) $s/d = 0.25$; (b) $s/d = 0.00$; (c) $s/d = -0.25$; and (d) $s/d = -1.00$

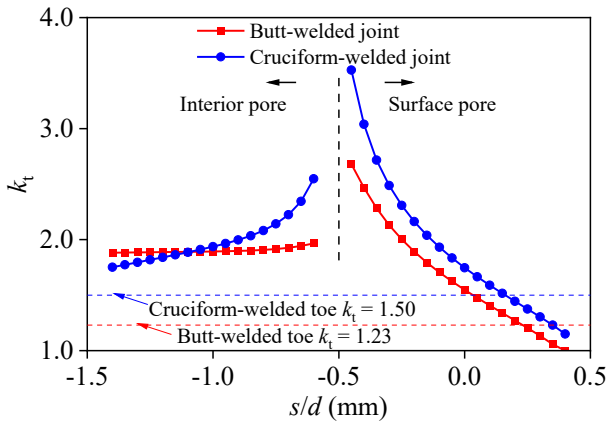


Fig. 23 Curve of stress concentration factor of welding details vs. pore depth

To further apply k_t to the structural design, a quantitative study on the parameters of porosity and the stress concentration effect was carried out. According to the standard EN ISO 5817[29], the diameter d of porosity defects is limited to within 3mm. Therefore, when determining the influence of porosity diameter on the stress state of welding details, the porosity diameter parameters were selected in accordance with this regulation, with the minimum pore diameter set at 0.2 mm and the maximum at 3.0 mm. Additionally, the location of porosity has a significant impact on their stress concentration coefficient. However, when the porosity does not overlap with the weld toe line, although their stress concentration coefficient is relatively large, the fatigue test results show that it does not affect their fatigue performance. Therefore, only the condition where the location coincides with the weld toe line is considered in the following analysis. The previous research results indicate that the porosity diameter has a varying degree of deteriorating effect on the fatigue resistance of welding details. When the porosity diameter is greater than 0.4 mm, the relationship between the porosity diameter parameter and the defect deterioration coefficient conforms to the polynomial function rule. Therefore,

perpendicular to the tensile load and passing through the pore center when s/d equals 0.25, 0.00, -0.25, and -1.00, respectively. The relationship between the stress concentration factor and porosity depth s/d is presented in Fig. 23. A critical observation is the pronounced decrease in the stress concentration factor for the cruciform joint when $s/d < -0.5$, exemplified by a value of 1.75 at $s/d = -1.4$. In contrast, the butt-welded joint showed minimal impact from pore depth changes, with the stress concentration factor remaining high at the $s/d = -0.50$ threshold.

the evaluation equation for the deteriorating effect of porosity defects on fatigue resistance can be determined through fitting. The least square method is used to fit the relationship between the porosity diameter and the defect deterioration coefficient according to the cubic function shown in Eq. (7):

$$k_t = a_1 + a_2 d + a_3 d^2 + a_4 d^3 \quad (8)$$

where d is the diameter of the porosity defect. Coefficient a_i ($i=1, 2, 3, 4$) of the equation can be determined by fitting the results of parameter analysis, as shown in Table 6.

Table 6
The fitted coefficient of k_t

Coefficient	Butt-welded joint	Cruciform-welded joint
a_1	1.1810	1.0745
a_2	-0.5787	-0.3000
a_3	0.2244	0.0954
a_4	-0.0312	-0.0110

Fig. 24 shows the correlation coefficient between the calculated values of the stress concentration equation for porosity defects and the calculated values from the finite element model. The correlation coefficients for butt- and cruciform-welded joint are above 0.9991, indicating that the proposed effect evaluation equation can well reflect the stress concentration effect.

It should be noted that the parameters of the equation are determined through data fitting based on a range of 0.4 to 3.0 mm for the porosity diameter. Therefore, this equation is mainly applicable to situations where the porosity diameter is within the range of 0.4 to 3.0 mm. For cases where the porosity diameter is less than 0.4 mm, a more conservative approach can be taken by assuming a porosity diameter of 0.4 mm and determining the defect-induced stress concentration coefficient accordingly.

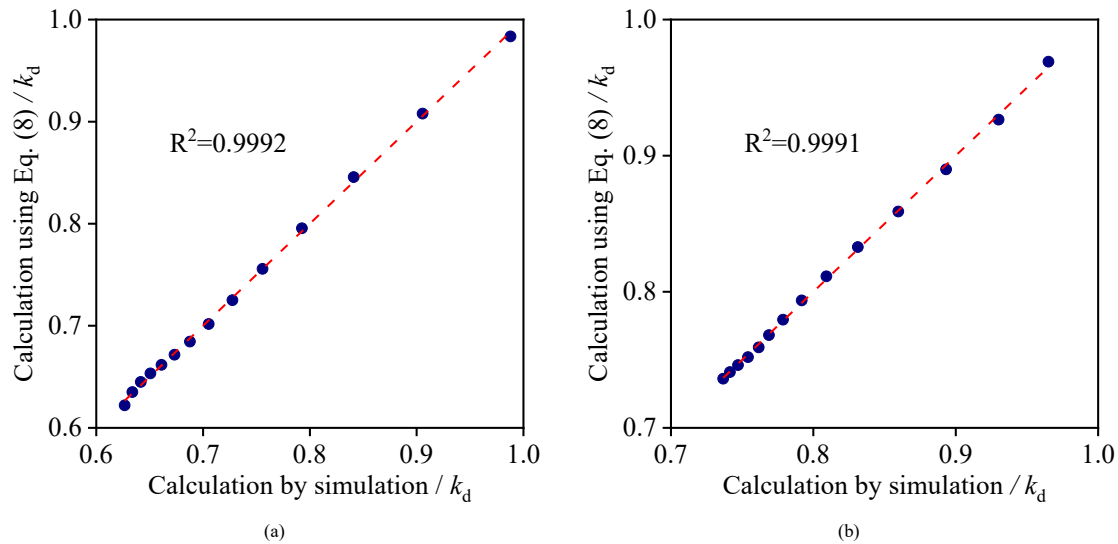


Fig. 24 Correlation coefficients of fitted equation: (a) butt-welded joint; cruciform-welded joint

5. Conclusions

This investigation centers on two prevalent types of welded joints found in steel bridge structures. Utilizing an integrated methodology of fatigue tests and numerical simulations, the study analyzes how porosity defects influence the fatigue performance and stress behavior of these joints. The main conclusions are as follows:

(1) The fatigue strengths at 2 million cycles for the conventional butt-welded and cruciform-welded joints without porosity were 142.0 MPa and 92.3 MPa, respectively. This failure mode was associated with cracks initiating at the weld toe, which then propagated through the plate thickness. The fracture surfaces were primarily elliptical, and the presence of multiple initiation zones in some specimens suggested stochastic crack nucleation behavior.

(2) For joints with porosity defects (2.0 mm depth/diameter), fatigue cracking originated at the porosity-weld toe intersection. This led to a fatigue strength of 103.1 MPa for butt-welded and 85.5 MPa for cruciform-welded joints, representing reductions of 27.4% and 7.4% compared to defect-free specimens, respectively, indicating a markedly higher defect sensitivity for the butt-welded geometry.

(3) Porosity defects induced a localized stress concentration in the weld region, significantly increasing the stress concentration factor of the welded joints. As the pore diameter increased, the stress concentration factor for both the butt-welded and cruciform-welded joints increased significantly, although the rate of increase slowed when the pore diameter exceeded 1.6 mm. The farther the pore was from the weld toe, the lower was the stress concentration factor, with cruciform joints being more sensitive to pore location. In addition, the surface pores exhibited significantly higher stress concentration factors than internal pores. When the pore was close to the weld surface, the stress concentration factor was high. As the pore depth increased, the stress concentration factor in cruciform joints decreased significantly, whereas the butt-welded joints were relatively less affected by the pore depth.

Acknowledgments

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