

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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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.

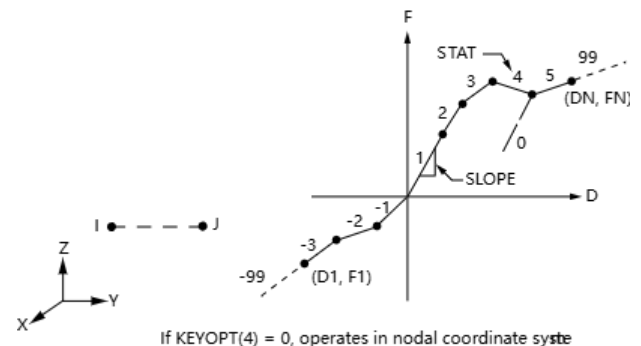
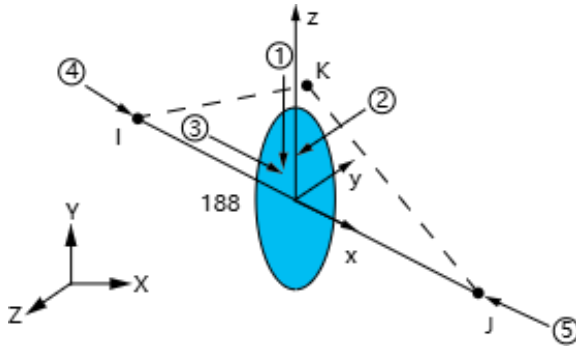
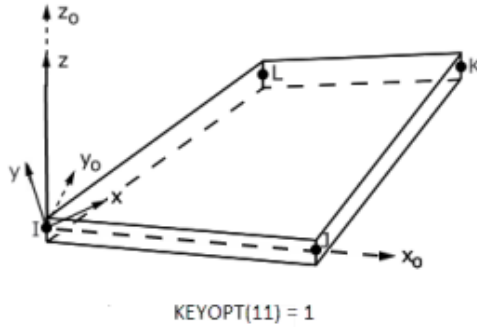
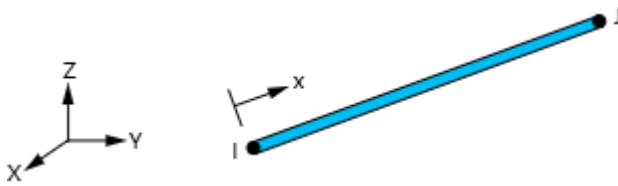


Fig. 3 COMBIN39 Geometry [26]

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.

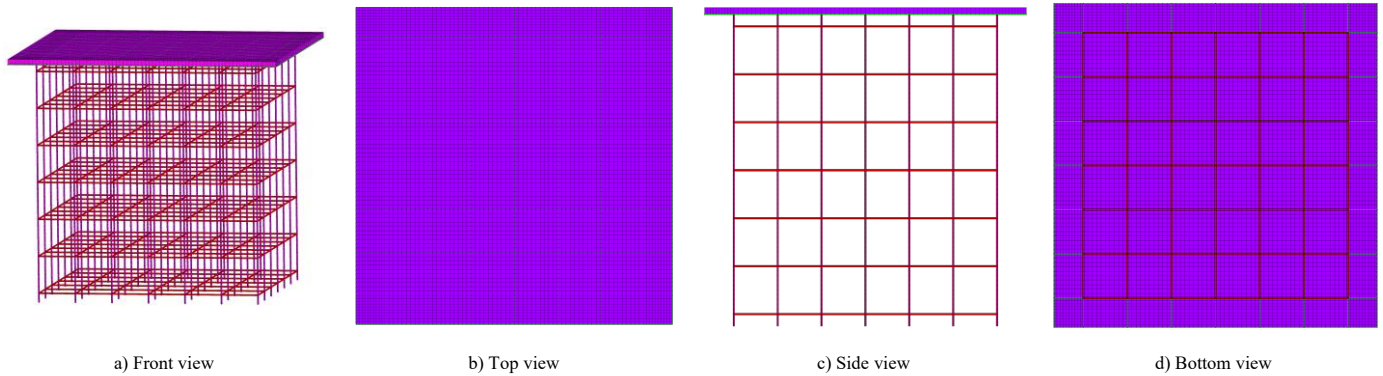


Fig. 7 Numerical model of the collaborative load-bearing system between the top plate and scaffolding

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].

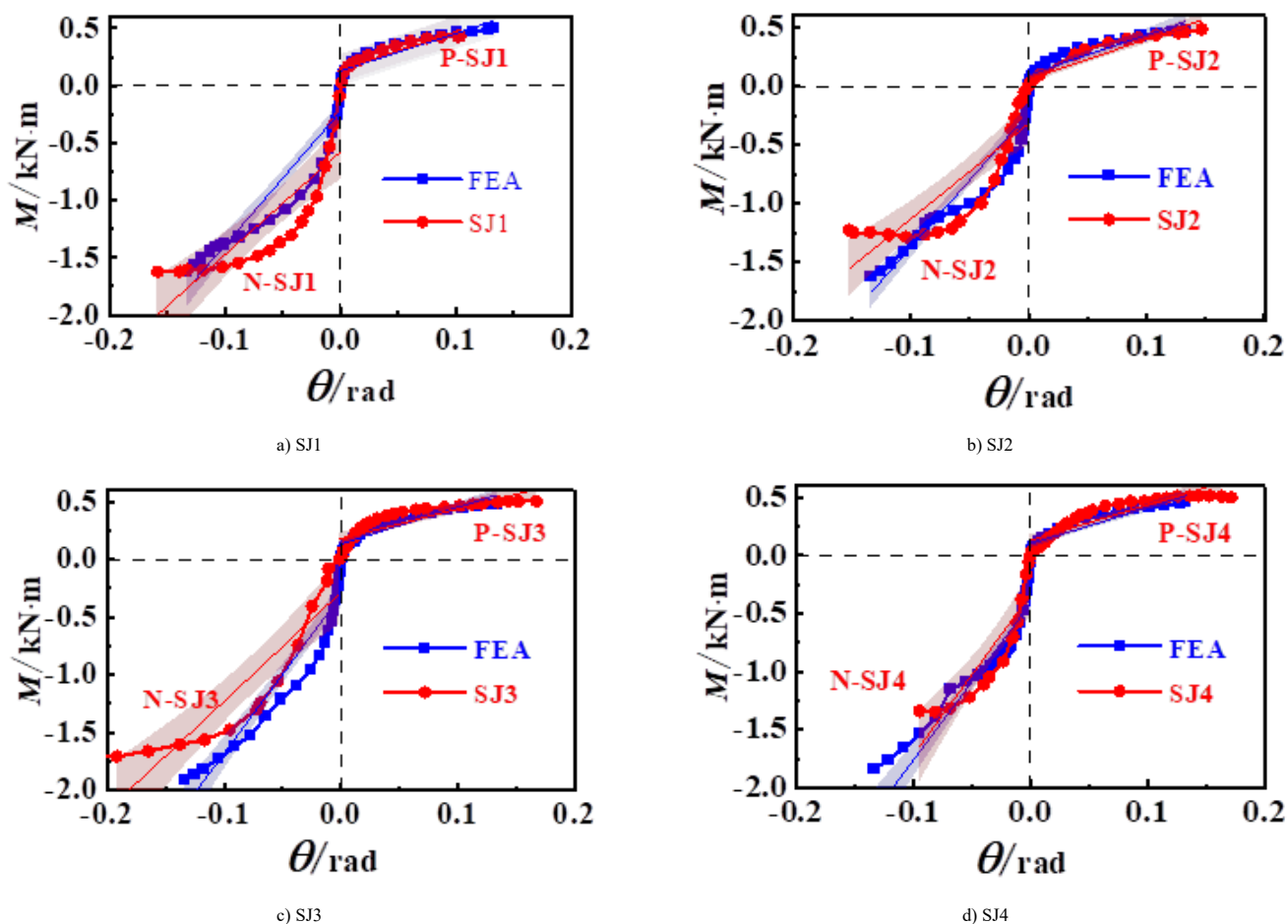


Fig. 8 Finite element simulation results and test results moment-rotation curve

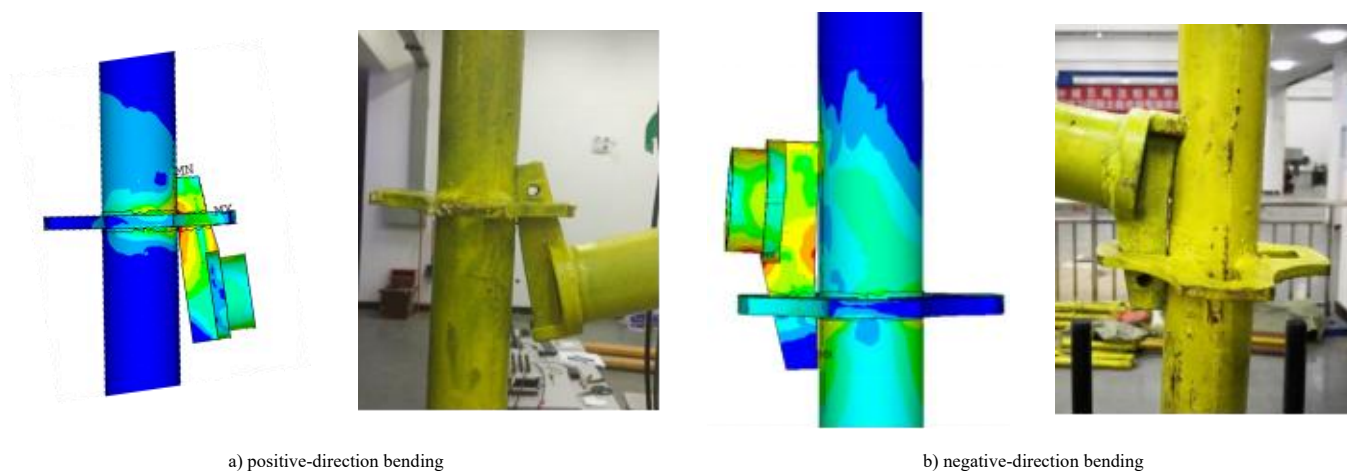


Fig. 9 Comparative analysis of failure mode

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 (R^2) 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 R^2 values ranging from 0.604 to 0.914. In particular, specimen SJ2 achieved an R^2 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 R^2 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.

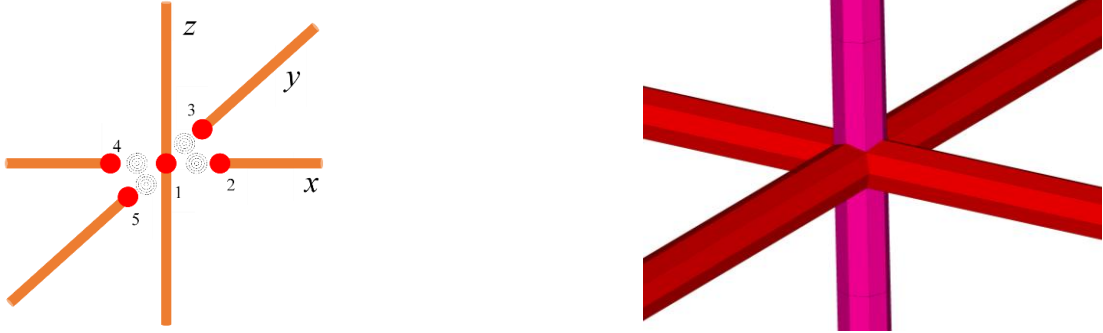


Fig. 10 Details of semi-rigid connections and physical model

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.

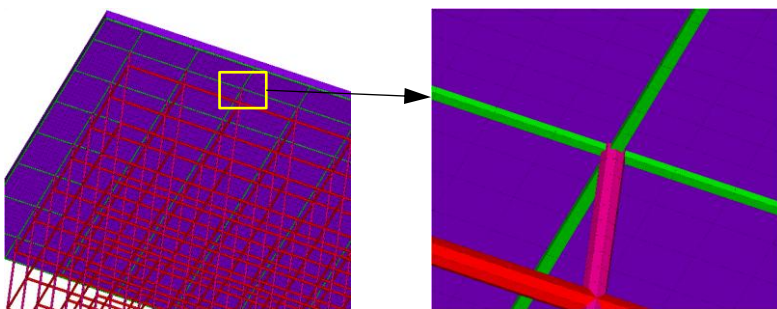


Fig. 11 Connection and details between the roof slab bottom surface and scaffolding

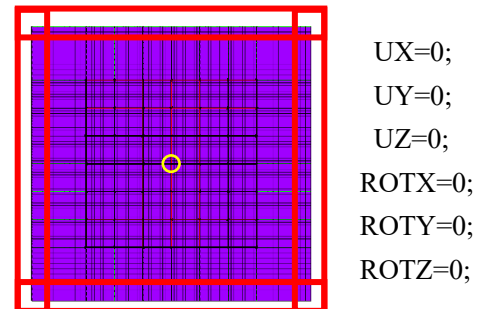


Fig. 12 Boundary conditions of the roof slab

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.

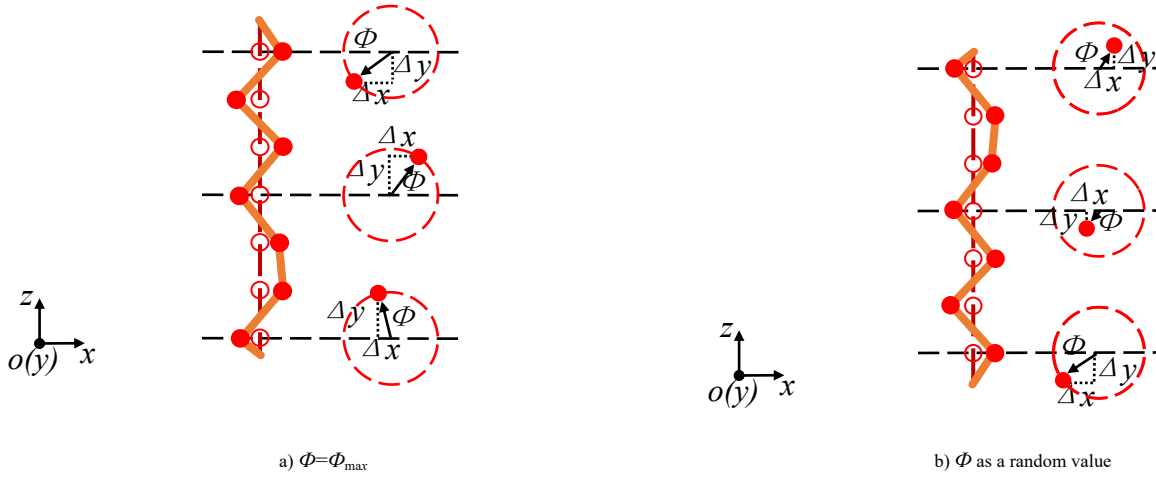


Fig. 13 Schematic of joint positioning errors

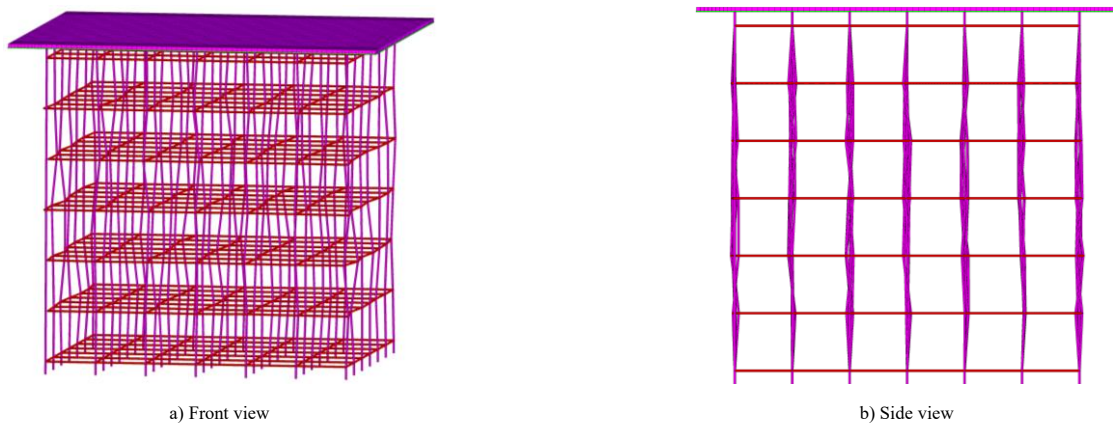


Fig. 14 Numerical model with joint positioning errors

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.

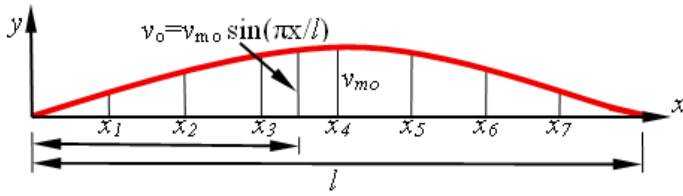


Fig. 15 Sinusoidal initial curvature

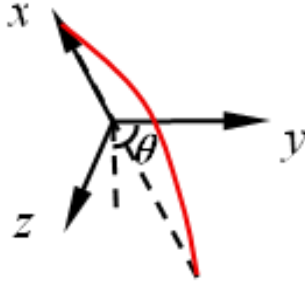


Fig. 16 Local coordinate system

The maximum bending amplitude is denoted as $v_{m0, \max}$, 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_{m0} can be set as a constant value $v_{m0, \max}$, or as a random value uniformly distributed between 0 and $v_{m0, \max}$. 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_{m0} 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.

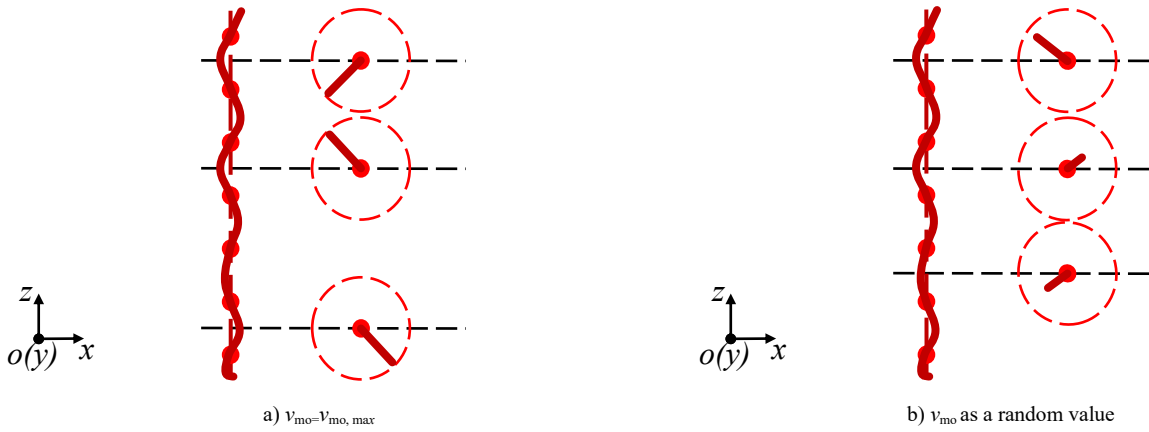


Fig. 17 Schematic of initial curvature in vertical members

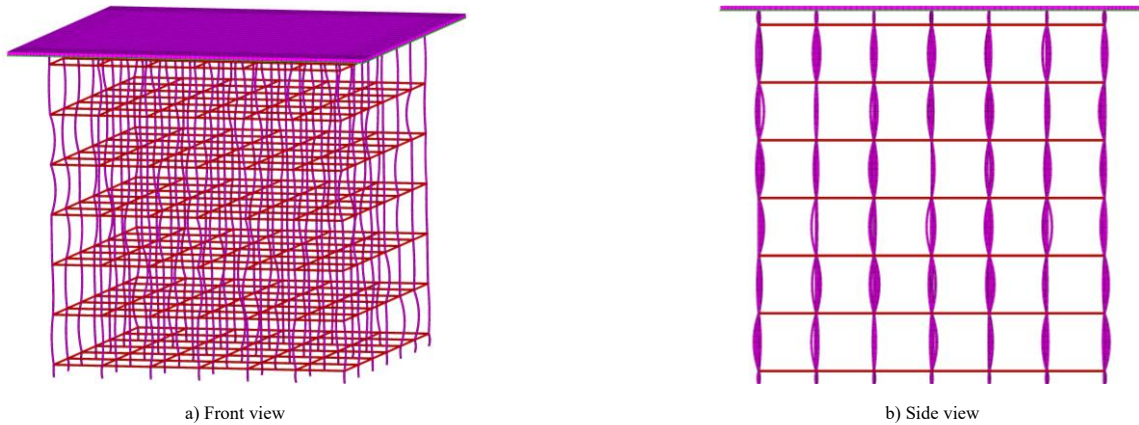


Fig. 18 Numerical model with random initial curvature

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.

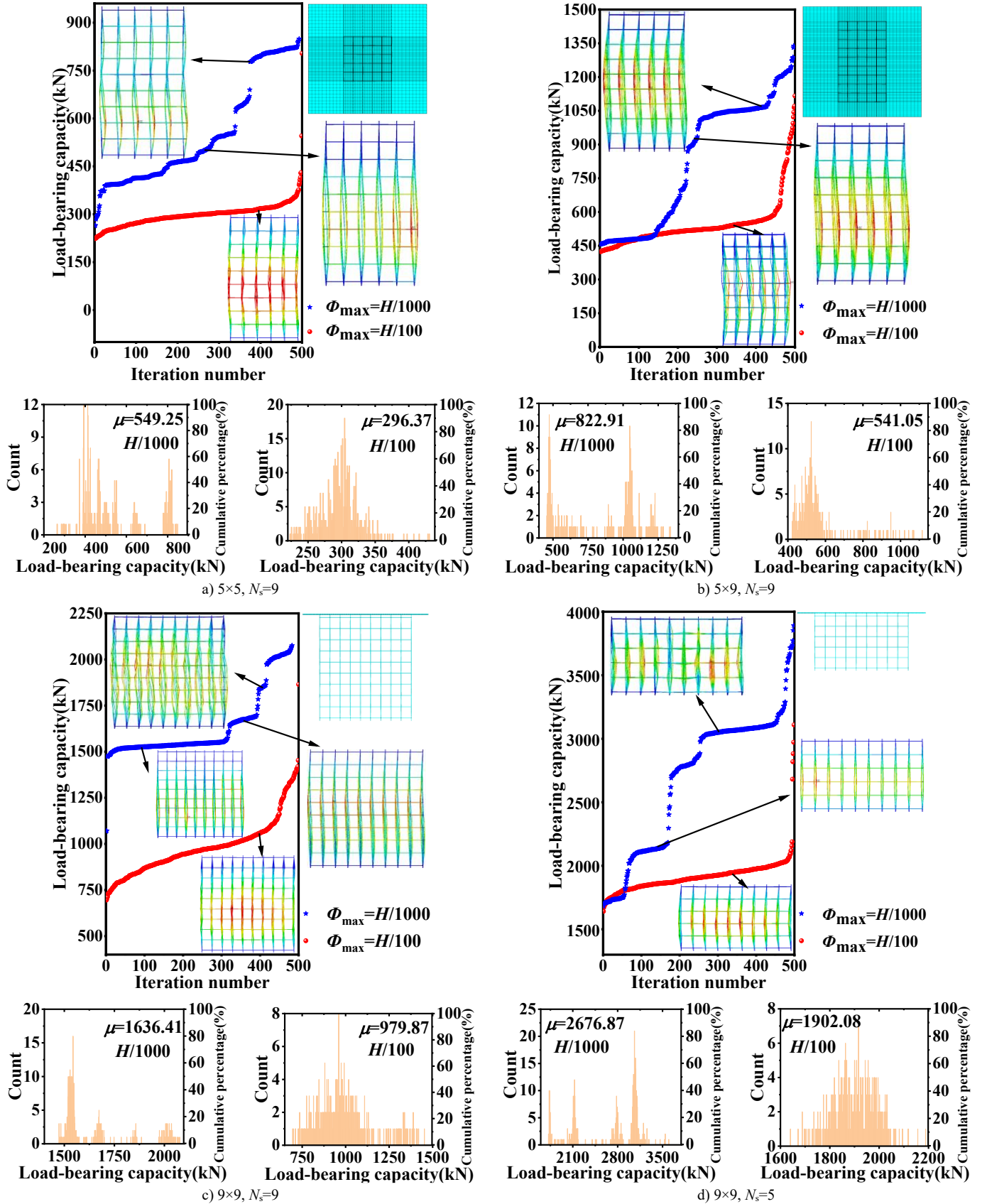


Fig. 19 Joint positioning errors on the stochastic distribution of load-bearing capacity

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_{mo} , $\text{max}=l/1000$ and v_{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

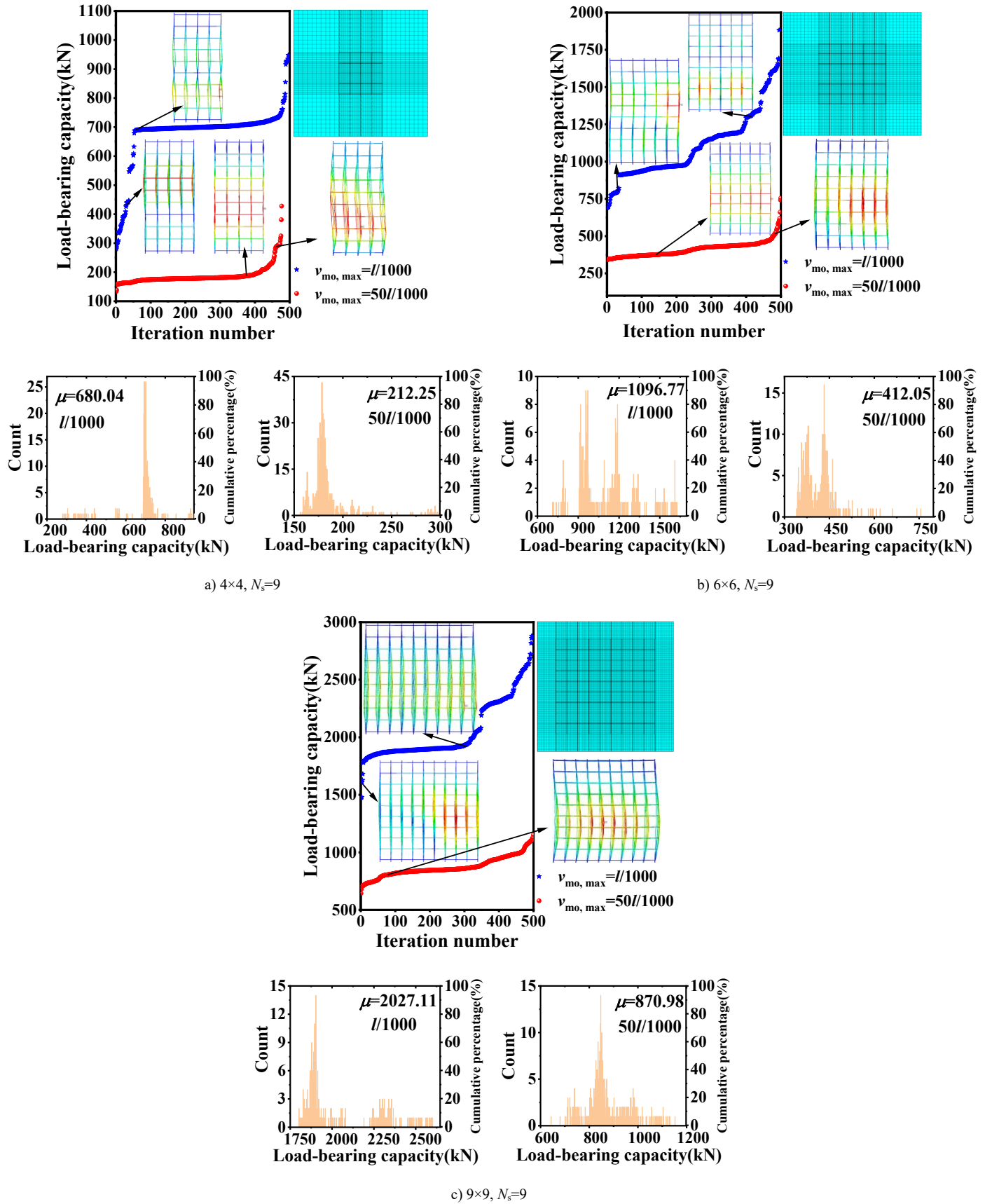


Fig. 20 Stochastic distribution of load-bearing capacity under different planar dimensions of scaffolding

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 ($v_{mo, max} \leq l/1000$).

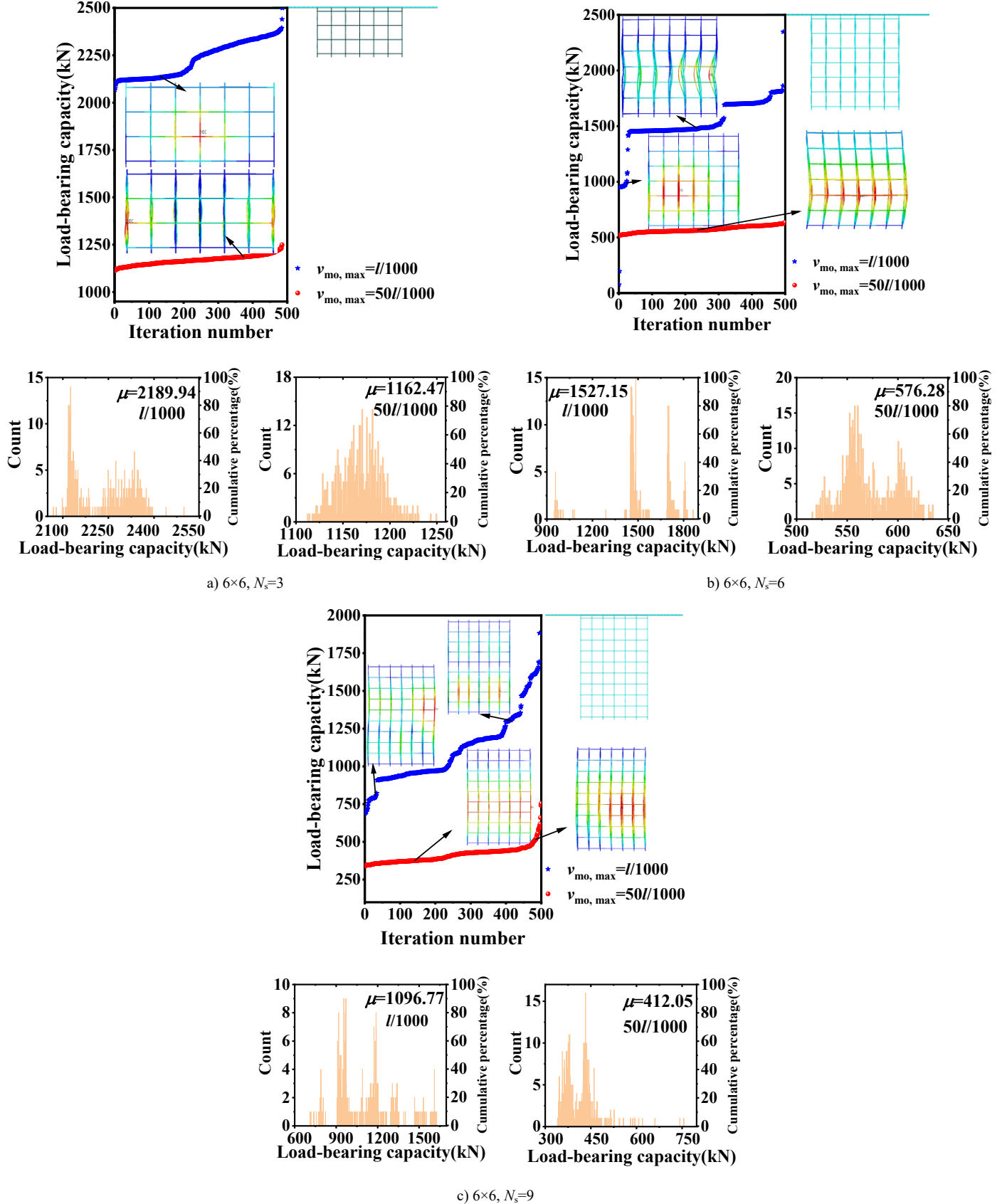


Fig. 21 Stochastic distribution of load-bearing capacity under different scaffolding heights

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.

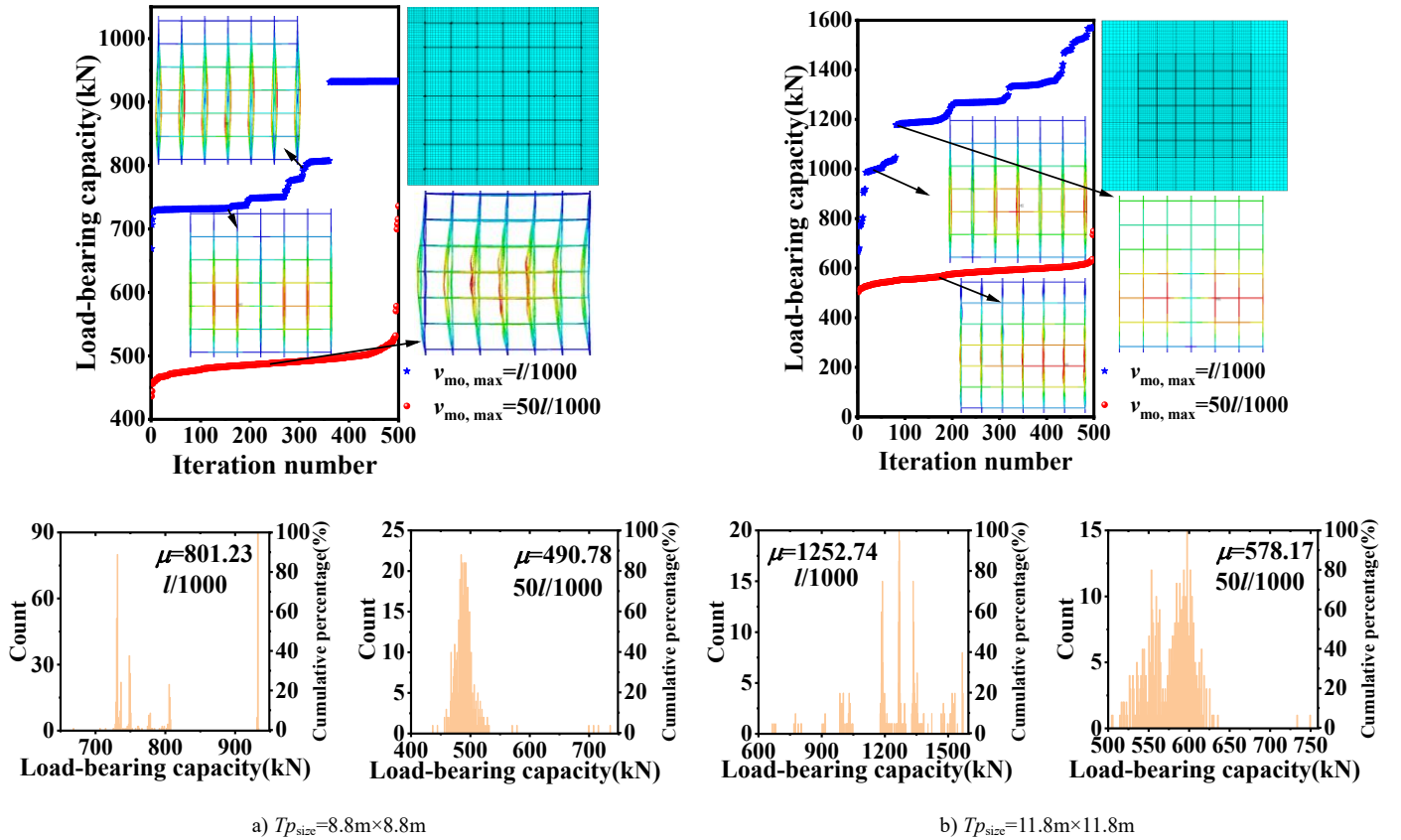
Comparative analysis of 3-layer and 9-layer systems reveals a clear inverse relationship between scaffolding height and stability load-bearing capacity. Under severe initial curvature ($v_{mo, \max} = 50/1000$), all configurations exhibit single-curve capacity distributions, indicating mono-modal buckling 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.



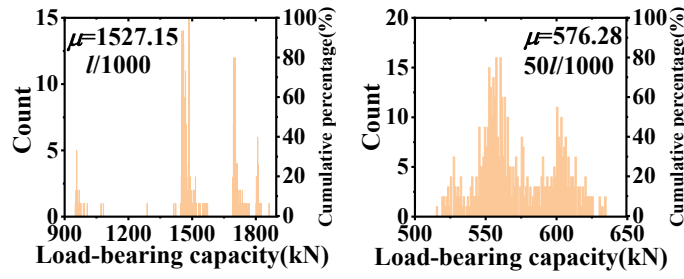
c) $Tp_{size}=14.8m \times 14.8m$

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_{mo} , $\max=1/1000$ exhibits a load-bearing capacity of 801.23 kN, which decreases by 38.75% to 490.78 kN when initial curvature increases to 501/1000. 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_{mo} , $\max=1/1000$), 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_{mo} , $\max=501/1000$), 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 1/1000 to 501/1000. 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.

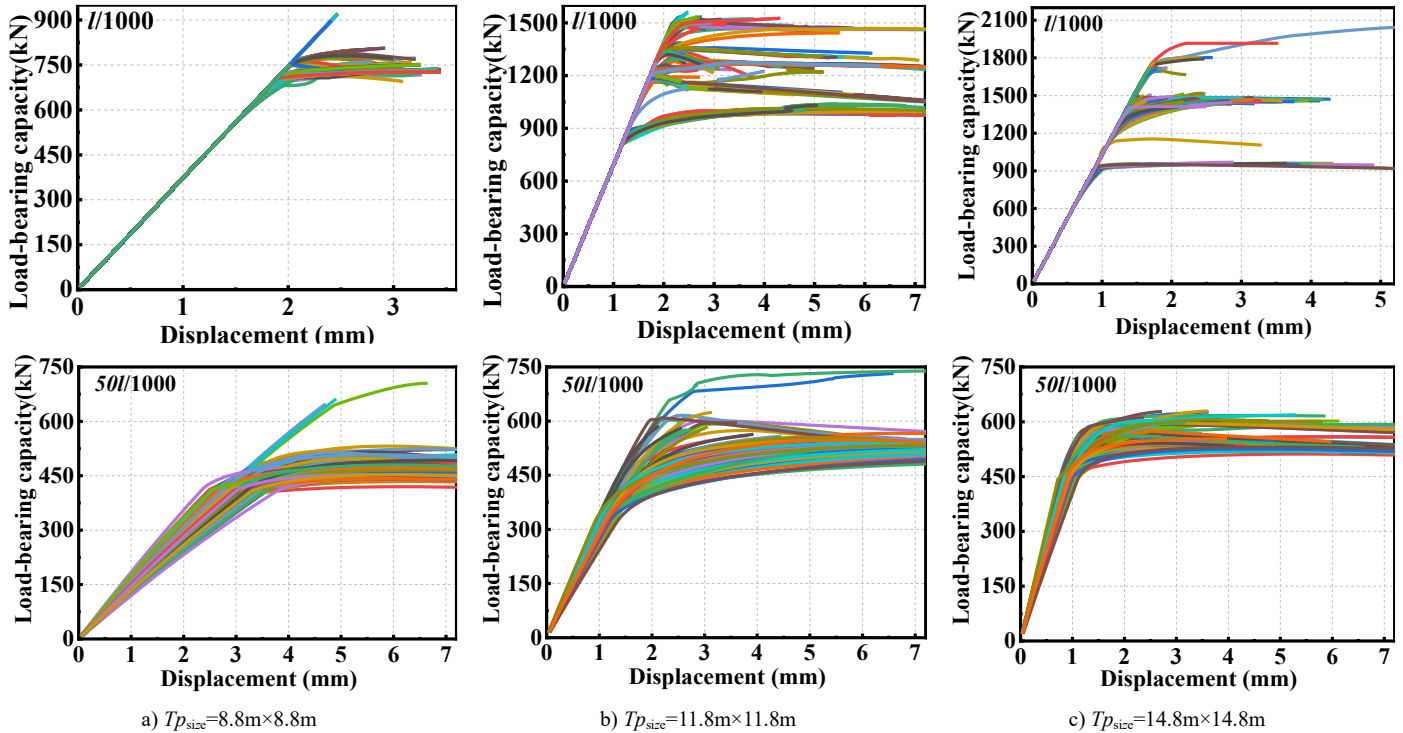


Fig. 23 Load-displacement curves

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.

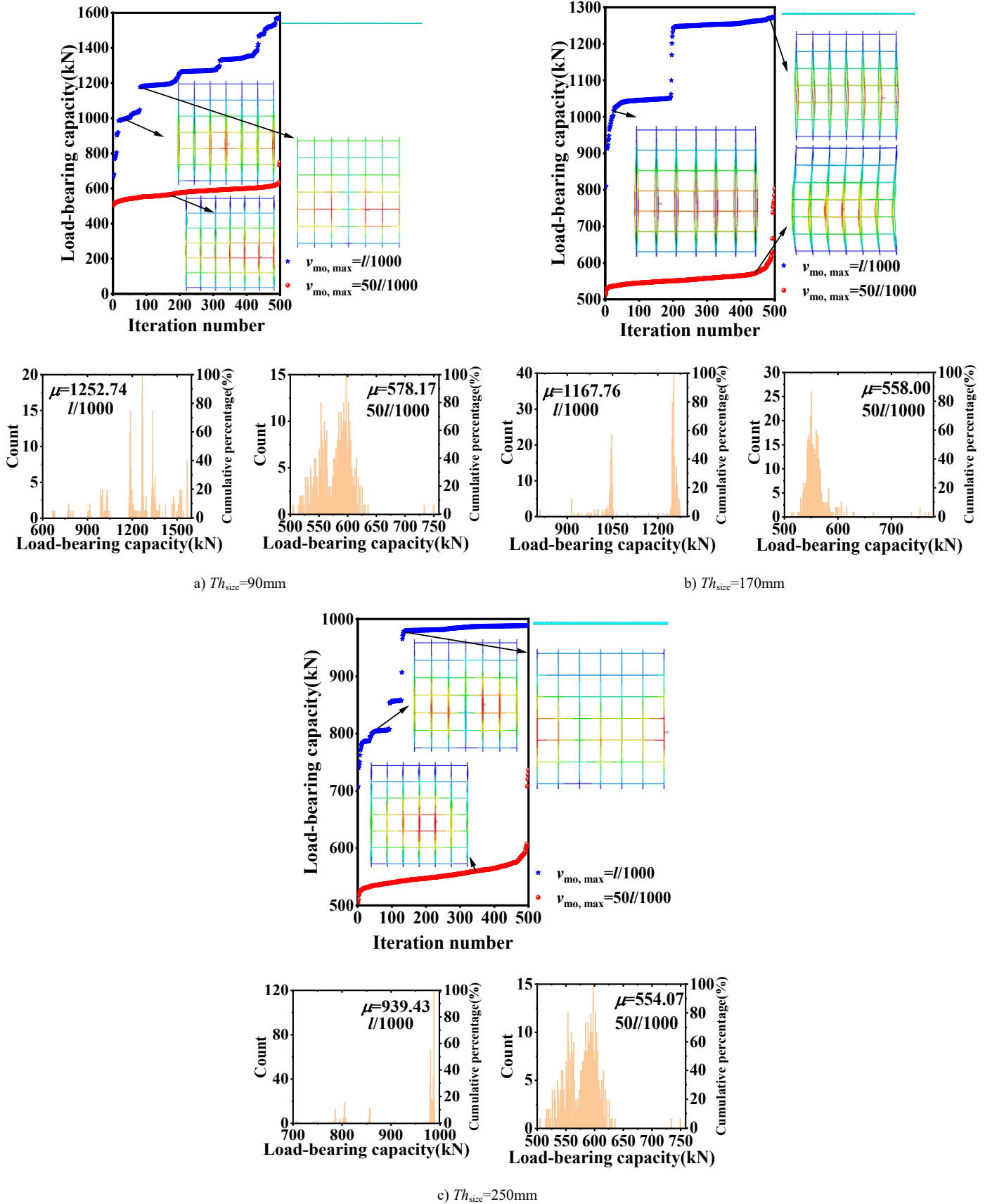


Fig. 24 Stochastic distribution of load-bearing capacity under different roof slab thicknesses

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 $N_s=6$		

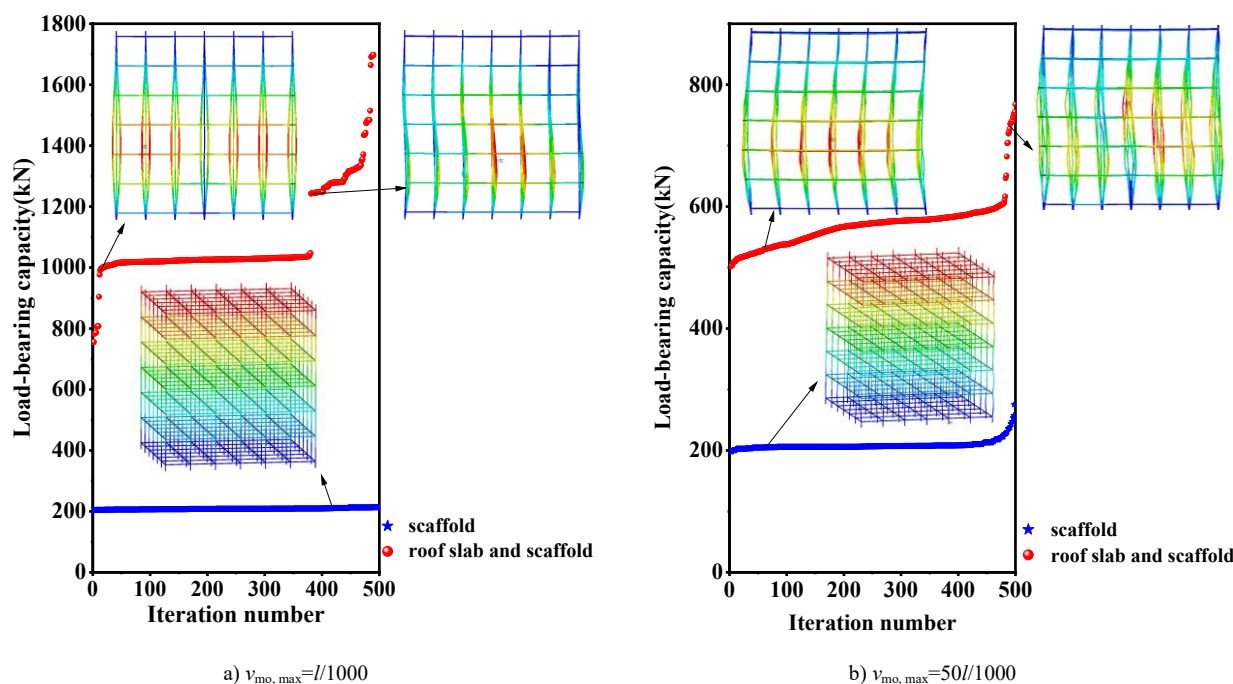


Fig. 25 Stochastic distribution of collaborative load-bearing capacity between roof slab and scaffolding

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.

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