

SEISMIC PERFORMANCE AND DAMAGE EVALUATION OF IRREGULAR STEEL JOINTS IN MODERN CHINESE TRADITIONAL-STYLE BUILDINGS WITH BLIND BOLTS

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ABSTRACT

Steel modern Chinese traditional-style buildings (MCTBs) commonly adopt fully welded connections. However, the use of circular hollow section (CHS) columns at beam-column joints, coupled with strict architectural aesthetic requirements, presents significant challenges for the application of conventional bolted connections. This issue represents a critical technical bottleneck that hinders the development of prefabricated construction in such buildings. To address this challenge, a novel prefabricated joint configuration based on blind bolt-T-stub assemblies was proposed. Four 1:2.6-scale specimens, including two single-beam column joints and two double-beam column joints, were fabricated and tested under low-cycle reversed loading to evaluate their seismic performance in terms of failure modes, hysteretic behavior, ductility, energy dissipation, and stiffness degradation. Furthermore, refined finite element (FEA) models were established to reproduce the experimental deformation patterns, stress distribution, and moment-rotation hysteresis behavior, including the relative slippage of the double-beam connectors. All specimens conformed to the strong-column weak-beam design principle, with damage primarily concentrated in the T-stub region. Compared to the single-beam column joints, the double-beam column joints demonstrated superior load-bearing capacity, enhanced energy dissipation, and greater safety redundancy. Moreover, increasing the T-stub thickness further improved the overall performance. Seismic damage was quantified using a modified Park-Ang damage model, confirming that the proposed prefabricated joint exhibits favorable seismic behavior and practical engineering applicability. The findings offer an effective structural solution for the industrialized seismic design of beam-column joints in steel MCTBs.

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1. Introduction

Ancient Chinese architecture, characterized by its distinctive architectural style and sophisticated construction techniques, represents the remarkable artistic achievements of Chinese civilization and serves as an essential component of the architectural heritage of the world^[1]. Due to the inherent shortcomings of timber, such as flammability and susceptibility to decay, along with prolonged exposure to natural disasters and human-induced damage, ancient timber structures have become extremely scarce. Moreover, most of these buildings suffer from deterioration problems including structural aging, corrosion, and insect damage, posing serious challenges to their overall safety^[2]. How to preserve and inherit traditional architectural culture while satisfying the diversified demands of modern society for building functionality and aesthetics has become an urgent issue requiring resolution^[3]. In response to this challenge, MCTBs have emerged, integrating contemporary construction technologies and materials while faithfully reproducing the form and appearance of ancient Chinese architecture. Meanwhile, the structural strength and durability have been significantly improved, demonstrating excellent engineering applicability and broad prospects for development. At present, representative MCTB projects such as the Chang'an Tower in Xi'an, Dingding Gate in Luoyang, Leifeng Pagoda in Hangzhou, and Tengwang Pavilion in Nanchang have been successively constructed and have become important landmarks in major historical cities^[4,5]. Owing to their excellent seismic performance, including high-strength and good ductility, steel structures have been widely applied in MCTBs. These buildings adopt steel structural systems while retaining the traditional appearance and compositional principles of ancient Chinese architecture^[6]. The seismic behavior of the entire structural system is largely governed by the performance of beam-column joints^[7]. The roof structures of Chinese hall-style buildings are characterized by diverse forms and deep overhanging eaves, forming a "heavy roof" that occupies a significant portion of the building elevation. This prominent roof configuration is one of the most representative structural features of ancient timber architectures. To ensure the overall structural stability required by such roof systems, ancient craftsmen gradually developed a double-beam system consisting of the *Lan-e* (upper beam) and the *You-e* (lower beam)^[8], as shown in Fig. 1.

Although many components of ancient timber architecture are simplified in MCTBs, the main dimensions are retained to reflect the traditional architectural style. Accordingly, the columns in steel MCTBs are typically designed in two

segments. The lower column is typically composed of a circular hollow section (CHS) column with a large cross-section, whereas the upper column, which is designed to support decorative elements such as *dougong* (bracket set), generally adopts a rectangular hollow section (RHS) column with a smaller cross-section. The upper and lower columns are reliably connected through the installation of horizontal and vertical stiffening plates inside the lower column, thereby forming a "transition column". The double-beam column joint, consisting of the *Lan-e*, *You-e*, and the transition column, represents a typical and critical connection form in MCTBs. The joints in existing steel MCTBs are predominantly constructed using fully welded connections. Beams are typically made of I-sections, with the upper beam having a larger cross-sectional size than the lower beam. In smaller-scale MCTBs, only the *Lan-e* is provided, while the *You-e* is omitted, resulting in a single-beam column joint. In this study, these joints are collectively referred to as irregular steel joints in MCTBs, as the beam-column joint involves a non-standard spatial arrangement of members, including the *Lan-e* (upper beam), *You-e* (lower beam), and the transition column. The term "irregular" here describes the connection configuration and construction form, rather than the seismic irregularities (e.g., torsional or mass irregularity) defined in structural design codes. It is worth noting that non-structural components such as *dougong* (bracket sets) and *Que-Ti* (sparrow braces) primarily serve decorative purposes and do not contribute to the load-bearing system^[9].

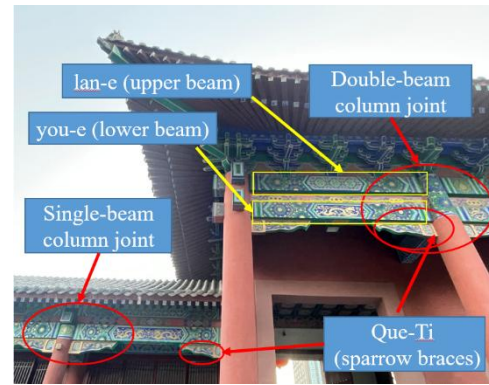
In recent years, increasing attention has been devoted to the study of MCTBs, particularly regarding the mechanical performance of irregular steel joints. These studies primarily address the hysteretic behavior, load-bearing mechanisms, and structural optimization of fully welded connections, covering a variety of joint configurations such as transition columns^[10], transition columns with bracket sets^[11], double-beam column joints^[12–14], and joints equipped with viscous^[15] or friction dampers^[16–18]. Previous studies have shown that steel MCTBs with fully welded connections exhibit favorable stiffness and load-bearing capacity. However, brittle fracture of the welds and connection failure often occur in the later stages of loading. In addition, fully welded connections impose stringent requirements on site conditions. In many MCTB projects, factors such as heritage preservation and spatial constraints are frequently involved, making it difficult to ensure welding quality during on-site construction and even posing a risk of secondary damage to surrounding historical components. Therefore, MCTBs are gradually transitioning toward bolted connection systems that offer greater efficiency and structural reliability.

However, due to the widespread use of CHS columns in MCTBs, the installation of conventional bolts is often difficult or even infeasible. Blind bolts have been widely recognized as an effective solution for connecting closed-section

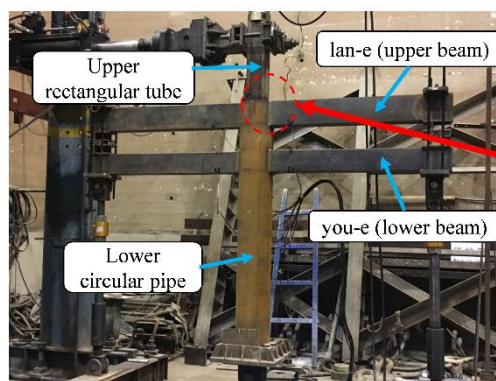
members, as they allow for single-sided installation and are particularly well-suited for closed-section columns.



(a) Ziyun Pavilion, Tang Paradise



(b) Irregular joints in MCTB



(a) Fully welded irregular steel joint in MCTB

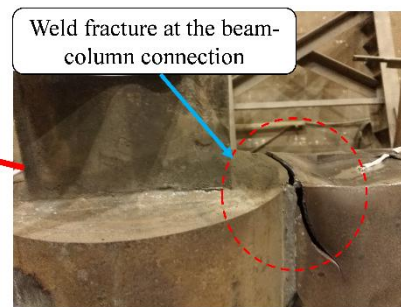


Fig. 1 Examples of MCTBs

Over the past few decades, various blind bolt products such as the Holo-Bolt, BOM, Ajax One-Side Bolt, Thread-Fixed One-Side Bolt (TFOB), and the Flowdrill series have been developed and applied in practical engineering^[19]. These blind bolts are typically used in combination with endplates for connecting steel beams to closed-section columns, significantly improving assembly efficiency and construction convenience. Tizani et al.^[20] developed the Extended Holo-Bolt (EHB), which was derived from the original Holo-Bolt.^[21] It has been demonstrated that the connection performance can be significantly enhanced when the steel tube is filled with concrete, making it comparable to that of standard bolted connections. However, in hollow columns without concrete infill, the expansion legs of the EHB may fracture suddenly once the connection reaches its peak load, resulting in abrupt connection failure^[22-25]. TFOBs^[26,27] and Flowdrill system^[28] form internal threads on the column wall, enabling bolt installation without the need for nuts. However, the Flowdrill system is only suitable for RHS tubes made of thin-walled materials. The Ajax one-side bolt features a structural configuration and load-bearing mechanism similar to those of conventional high-strength bolts, exhibiting excellent mechanical performance. Lee et al. improved the Ajax one-side bolt and developed the Headed Anchor Blind Bolt (HABB), followed by a series of studies focusing on its mechanical performance^[29,30]. The Ajax one-side bolt features a structural configuration and load-bearing mechanism similar to those of conventional high-strength bolts, exhibiting excellent mechanical performance. Based on the fastening mechanism of the Ajax one-side bolt, Wang et al.^[31] proposed a novel Slip-critical Blind Bolt (SCBB). This bolt incorporates a compressive washer and a pre-tightening sleeve structure^[32], which not only enables a preloading effect comparable to that of conventional high-strength bolts, but also achieves a pre-tensioning force that satisfies the requirements specified in the current Chinese design code^[33]. Experimental studies have demonstrated that the SCBB provides excellent connection performance under various loading conditions, including single-bolt tension, shear, and combined loading^[34]. In end-plate connections between RHS columns and I-section beams, SCBB connections exhibit superior hysteretic behavior and energy dissipation capacity under cyclic loading, with overall performance significantly exceeding that of conventional blind bolt

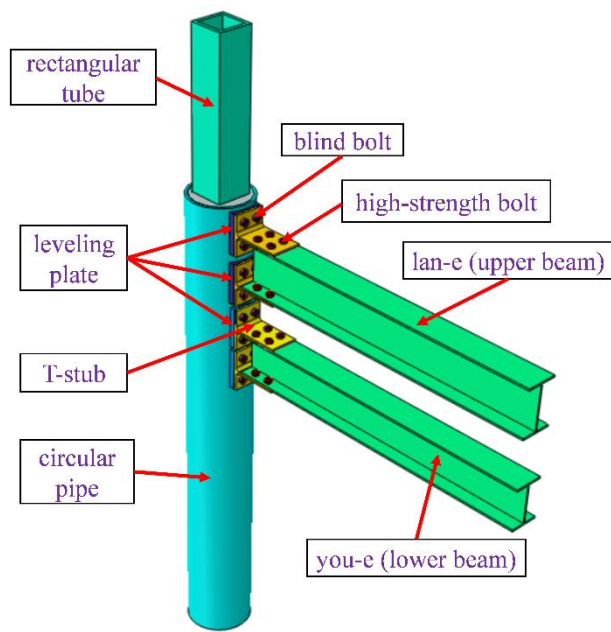
connections^[35]. At present, blind bolt connections for CHS columns are typically configured as symmetrical end-plate and blind bolt assemblies. However, due to their relatively large dimensions and the projection of bolts beyond the plane of the frame, such configurations are difficult to satisfy the decorative and structural concealment requirements of MCTBs.

In this study, SCBBs were introduced into irregular steel joints in MCTBs to achieve an efficient prefabricated connection between CHS columns and I-section steel beams. Four joint specimens were subjected to low-cycle reversed loading to evaluate their seismic behavior, including failure characteristics, hysteretic response, ductility, and energy dissipation capacity. In addition, the cumulative damage behavior of the joints was preliminarily evaluated. The results provide a novel prefabricated and industrialized connection system for MCTBs, offering substantial engineering value for the preservation and continuation of traditional Chinese architectural culture.

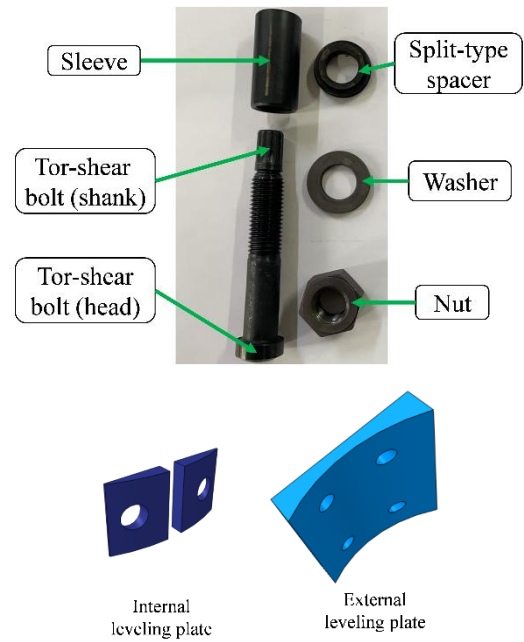
2. Experimental program

2.1. Specimen design

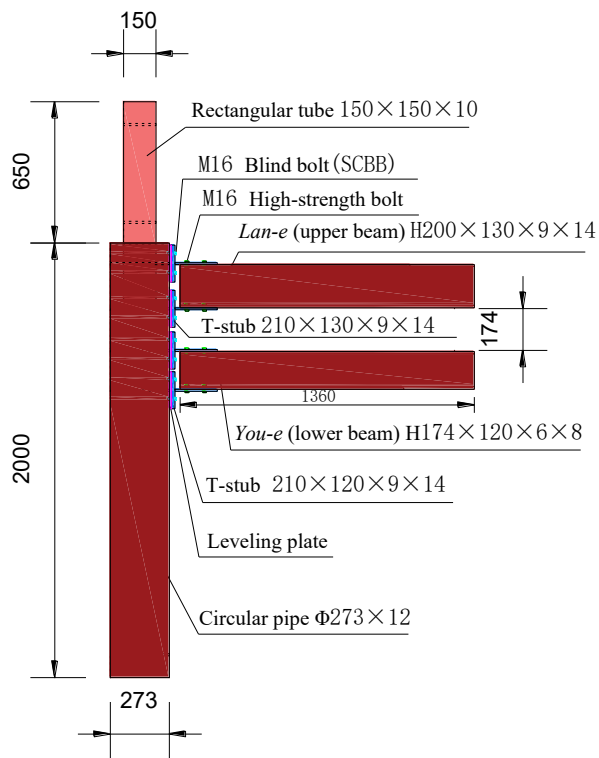
The research group previously carried out experimental and finite element studies on the seismic performance of fully welded steel MCTBs. Building upon this foundation, the present study optimizes the beam-column connection by proposing a novel prefabricated joint configuration that incorporates a blind bolt and T-stub assembly. The proposed joint is composed of a steel transition column, an I-section steel beam, and several connection components. The connection components include T-stubs, internal and external leveling plates, high-strength bolts, and blind bolts. The detailed configuration is shown in Fig. 2. The steel transition column is composed of an upper rectangular tube, a lower circular pipe, as well as horizontal and vertical stiffeners, which are welded together^[36]. T-stubs serve as the connecting elements between the lower circular pipe and the I-section steel beam. Internal and external leveling plates are placed between the lower circular pipe and the web of the T-stub and are connected using SCBB blind bolts. The flange of the T-stub is connected to the I-section beam by high-strength bolts.



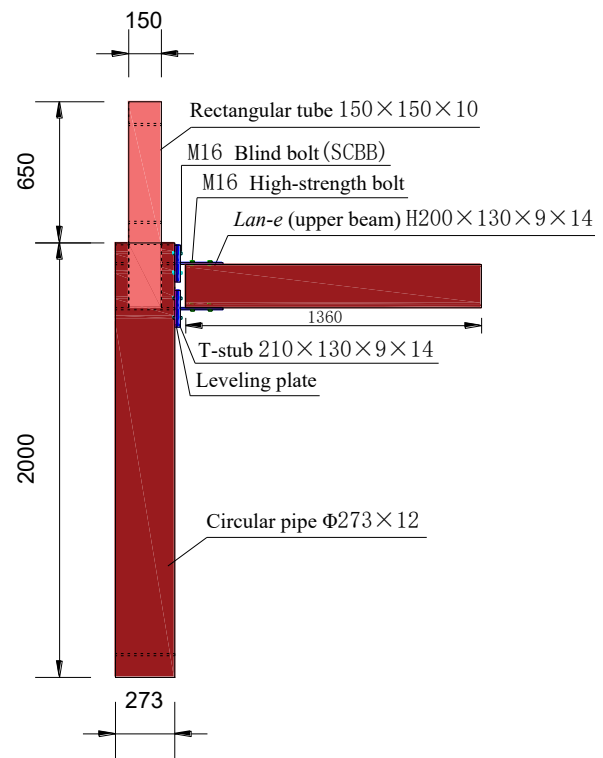
(a) Configuration of Double-beam column joint



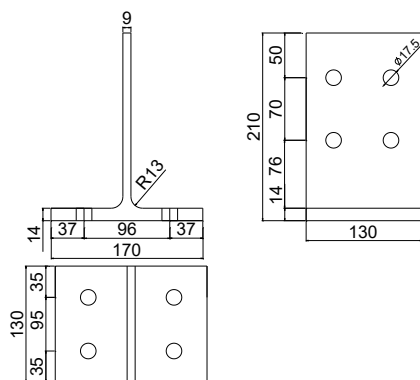
(b) Blind bolt and leveling plate



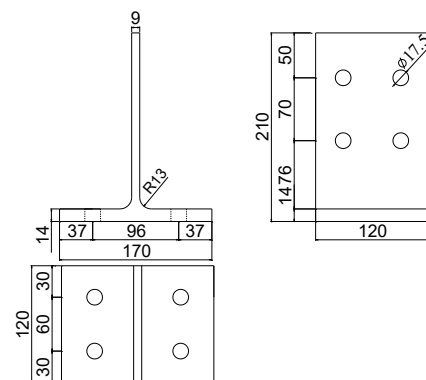
(c) Double-beam column joint (JD1, JD2)



(d) Single-beam column joint (JD3, JD4)



(e) Upper T-stub



(f) Lower T-stub

Fig. 2 Test specimens (Unit: mm)

Yingzao Fashi, issued during the Song Dynasty, is the first officially promulgated building standard in the history of ancient Chinese architecture^[37]. It provides explicit specifications for architectural hierarchy, height-to-width ratios, bay spacing, as well as the structural forms and dimensional standards of key components such as beams and columns. The building standard established a modular system based on the standardized units of *ts'ai*, *chi'i*, and *fen*, which is referred to as the “material portion grading system”. The dimensions of all building components were controlled according to fixed proportional relationships, thereby ensuring consistency in design and reliability in structural performance.

Once the architectural hierarchy (i.e., the rank of the building as classified in traditional Chinese standards) was determined, the height-to-width ratio and column spacing were subsequently defined, based on which the lengths of beams and columns were calculated. The width of the beam was determined by the column diameter, and the beam cross-section maintained a fixed height-to-width ratio of 3:2. For other components such as bracket sets, corresponding specifications were also explicitly provided in the *Yingzao Fashi*. This standardized design methodology reflects the advanced understanding of structural mechanics embodied in ancient Chinese architecture. For steel MCTBs constructed using modern steel materials, other parameters such as the wall thickness of circular steel tube columns and the thicknesses of beam flanges and webs need to be determined through structural calculations.

A total of four irregular steel joints in MCTBs with blind bolts were designed in this study, including two double-beam column joints and two single-beam column joints. The specimen dimensions were determined with reference to the previous experimental study on fully welded irregular steel joints in MCTBs conducted by the research group, and were further adjusted based on practical engineering requirements and the material portion grading system described in the *Yingzao Fashi*. The prototype of the specimen was based on a second-class hall-style building from the Tang Dynasty, and its structural dimensions were determined in accordance with the material portion grading system specified in the *Yingzao Fashi*. In this modular system, the smallest unit was *fen*. The diameter of the eave column was 42 *fen*, and the beam

cross-section followed a fixed height to width ratio of 3:2. Specifically, the *lan-e* (upper beam) had a height and width of 30 *fen* and 20 *fen*, respectively, while the *you-e* (lower beam) measured 27 *fen* in height and 18 *fen* in width. According to the modern metric system, 1 *fen* is equivalent to 17.16 millimeters, and the eave column diameter was therefore approximately 720 millimeters.

To accommodate the capacity of the laboratory loading equipment and to comply with the current national standards for seamless circular steel tubes^[38], the final scale ratio of the specimen model was determined as 1:2.6, and a seamless circular steel tube with an outer diameter of 273 mm was selected for the eave column. An axial compression ratio of 0.2 corresponds to an axial force of 275 kN. The remaining structural dimensions were determined using the same conversion method. The main parameters are summarized in Table 1, and the specimen dimensions are illustrated in Fig. 2. The total length of the steel column was 2650 mm, consisting of two segments with different cross-sections. The lower segment was a circular pipe with dimensions of $\Phi 273 \text{ mm} \times 12 \text{ mm}$, and the upper segment was a rectangular tube measuring $150 \text{ mm} \times 150 \text{ mm} \times 10 \text{ mm}$. The beam adopted an I-section profile, with the upper beam and lower beam having cross-sectional dimensions of $200 \text{ mm} \times 130 \text{ mm} \times 6 \text{ mm} \times 8 \text{ mm}$ and $174 \text{ mm} \times 120 \text{ mm} \times 6 \text{ mm} \times 8 \text{ mm}$, respectively. Openings were made in the connection region between the circular pipe column and the steel beam, and internal leveling plates were welded to the inner wall of the column^[39]. In addition, a circular horizontal stiffening plate with a thickness of 10 mm was welded inside the circular pipe column at the height corresponding to the beam flange. The upper rectangular tube was embedded into the lower circular pipe and extended downward to the level of the lower flange of the upper beam. Its bottom perimeter was welded to the inner wall of the circular pipe column using four vertical stiffeners. During the assembly of the beam and column, internal and external leveling plates were positioned on both sides of the circular pipe column wall and precisely aligned with the predrilled holes. The circular pipe column and the T-stub were connected using M16 Grade 10.9 nested blind bolts, while the web of the T-stub was connected to the beam flange using M16 Grade 10.9 high-strength bolts. The fabrication process of the specimen is illustrated in Fig. 3.

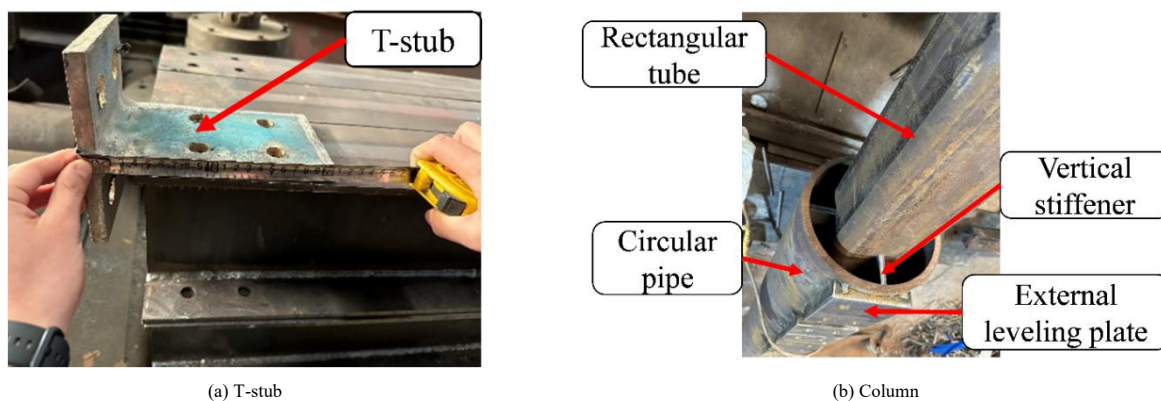


Fig. 3 Photos at manufacturing site

Table 1
Specimen details

Specimen	Circular pipe (mm)	Rectangular tube (mm)	Upper T-stub (mm)	Lower T-stub (mm)	Upper beam (mm)	Lower beam (mm)
JD1	273×12	150×10	T210×130×9×14	T210×120×9×14	HN200×130×6×8	HN174×120×6×8
JD2	273×12	150×10	T210×130×8×12	T210×120×8×12	HN200×130×6×8	HN174×120×6×8
JD3	273×12	150×10	T210×130×9×14	/	HN200×130×6×8	/
JD4	273×12	150×10	T210×130×8×12	/	HN200×130×6×8	/

Table 2
Mechanic performance indexes

Type	Thickness (mm)	f_y (MPa)	f_u (MPa)	E_s ($\times 10^5$ MPa)	δ (%)
Tube	12	288.3	388.1	1.92	/
Beam Flange	8	252.5	440.6	1.99	32.2
Beam Web	6	280.6	442.7	1.96	35.1
T-stub Flange	14	268.3	447.6	1.96	34.5
T-stub Web	9	271.1	445.2	2.16	37.5
T-stub Flange	12	283.3	483.3	2.21	39.5
T-stub Web	8	330.4	450.4	2.15	36.3

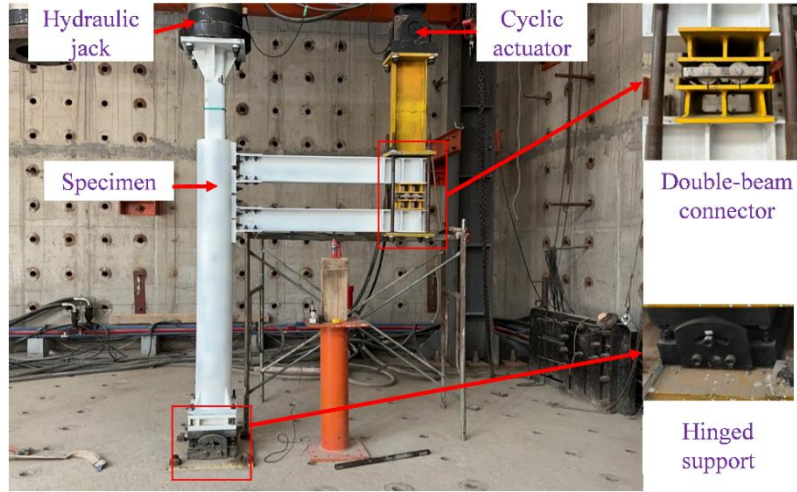
2.2. Mechanical properties of the material

All specimens were fabricated using Q235B steel. In accordance with the relevant Chinese standards^[40], tensile tests were conducted on samples taken from the upper rectangular tube, lower circular pipe, I-section steel beams, and T-stub. The measured mechanical properties of the steel materials are presented in Table 2.

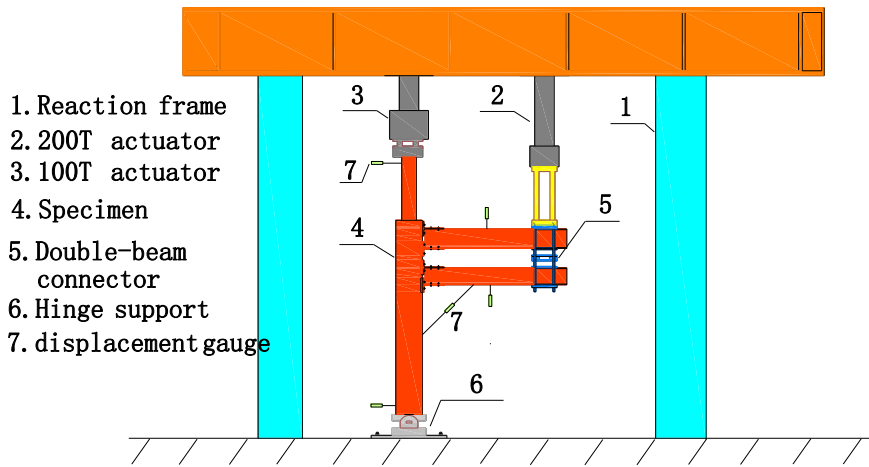
2.3. Loading scheme

The test was conducted using a beam end loading method, and the on-site loading setup is illustrated in Fig. 4. A vertical axial load of 275 kN was applied to the top of the column using a hydraulic jack, corresponding to an axial compression ratio of 0.2, and was maintained at a constant level throughout the entire test to ensure axial load stability during cyclic loading. An

electrohydraulic servo loading system was applied at the column top, with a maximum load capacity of 2000 kN and a displacement range of ± 100 mm. The column base was connected to the ground through a unidirectional hinged support, which was anchored to the rigid floor using ground anchor bolts. Cyclic loading was applied at the beam ends using a 500 kN electrohydraulic servo actuator. The beam end reached a maximum load of 1000 kN, with a displacement range of ± 250 mm. The entire testing procedure was controlled by an MTS973 electrohydraulic servo structural testing system, as illustrated in Fig. 4. To ensure that the beam end force conditions of the double beam column joints (JD1 and JD2) in the test are consistent with those in engineering practice, it is important to prevent moment transfer between the upper and lower beam ends during loading while allowing relative horizontal movement. A double-beam connector was designed and fabricated, as shown in Fig. 4(b). The extension of the hydraulic servo actuator was defined as the positive direction, whereas retraction was defined as the negative direction.



(a) On-site photograph



(b) Schematic diagram

Fig. 4 Test setup

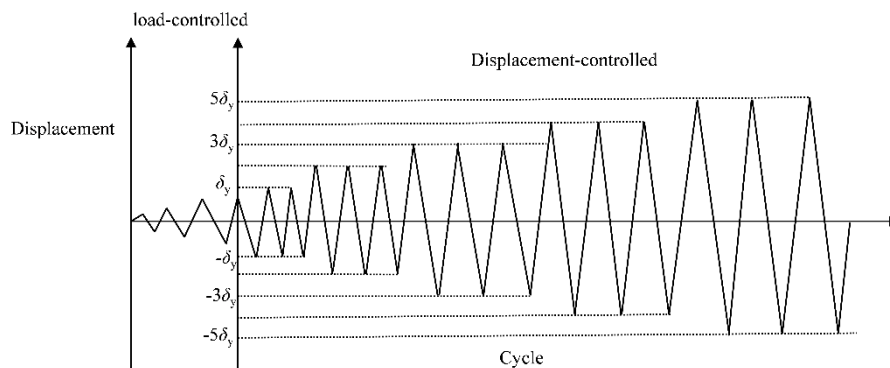


Fig. 5 Loading protocol

According to relevant standards^[41], the loading protocol is shown in Fig. 5. The yield displacement (δ_y) was determined based on strain measurements obtained from strain gauges installed at critical locations of the T-stub component. Yielding of the specimen was identified when the measured strain reached the yield strain of the steel material (ϵ_y), and the corresponding lateral displacement was defined as the yield displacement (δ_y). The cyclic loading history was applied in a symmetric displacement-controlled pattern in both positive and negative directions.

In the initial stage, displacement controlled loading was applied. After the specimen yielded, the displacement increment was progressively increased based on multiples of the yield displacement, with each loading level repeated three times. The test was terminated when the lateral load capacity dropped to 85% of the peak load, indicating significant strength degradation, or when severe failure of the specimen occurred.

2.4. Data measurement

The DH3816N static stress and strain analysis system was used to collect data on displacement, strain, and load during the test. Displacement gauges were primarily arranged at the top and bottom of the column. A cable displacement transducer was installed at the beam end to measure the horizontal displacement at the loading point. The measurement layout is illustrated in Fig. 4(b).

The joint rotation angle was calculated based on the measured beam-end displacement assuming that the beam segment between the column axis and the loading point behaves approximately as a rigid body. The rotation angle θ was calculated as $\theta = \Delta / L$, where Δ is the beam-end displacement recorded by the displacement transducer, and L is the distance from the column axis to the loading point. In the present test setup, $L = 1480$ mm. The bending moment was calculated as $M = P \times L$, where P is the applied load.

Strain gauges were primarily arranged on the flange and web of the T-stub, as well as on the web and flange at the beam ends. In addition, a small number of strain gauges were installed at the top, bottom, and mid height of the column. To monitor the strain development of the blind bolts during the test, a flat surface was ground at the root of the bolt shank, and strain gauges were affixed along the axial direction of the bolt. The measurement points are illustrated in Fig. 6.

For ease of identification, each strain gauge was labeled with a concise code comprising the component type, measurement location, and sequence number. The coding system is defined as follows. C represents the column, T represents the T-stub, UB represents the upper beam, and LB represents the lower beam. For example, the code T-UB-1-1 indicates the first strain gauge installed on the upper T-stub, whereas T-LB-2-1 refers to the first strain gauge installed on the lower T-stub.

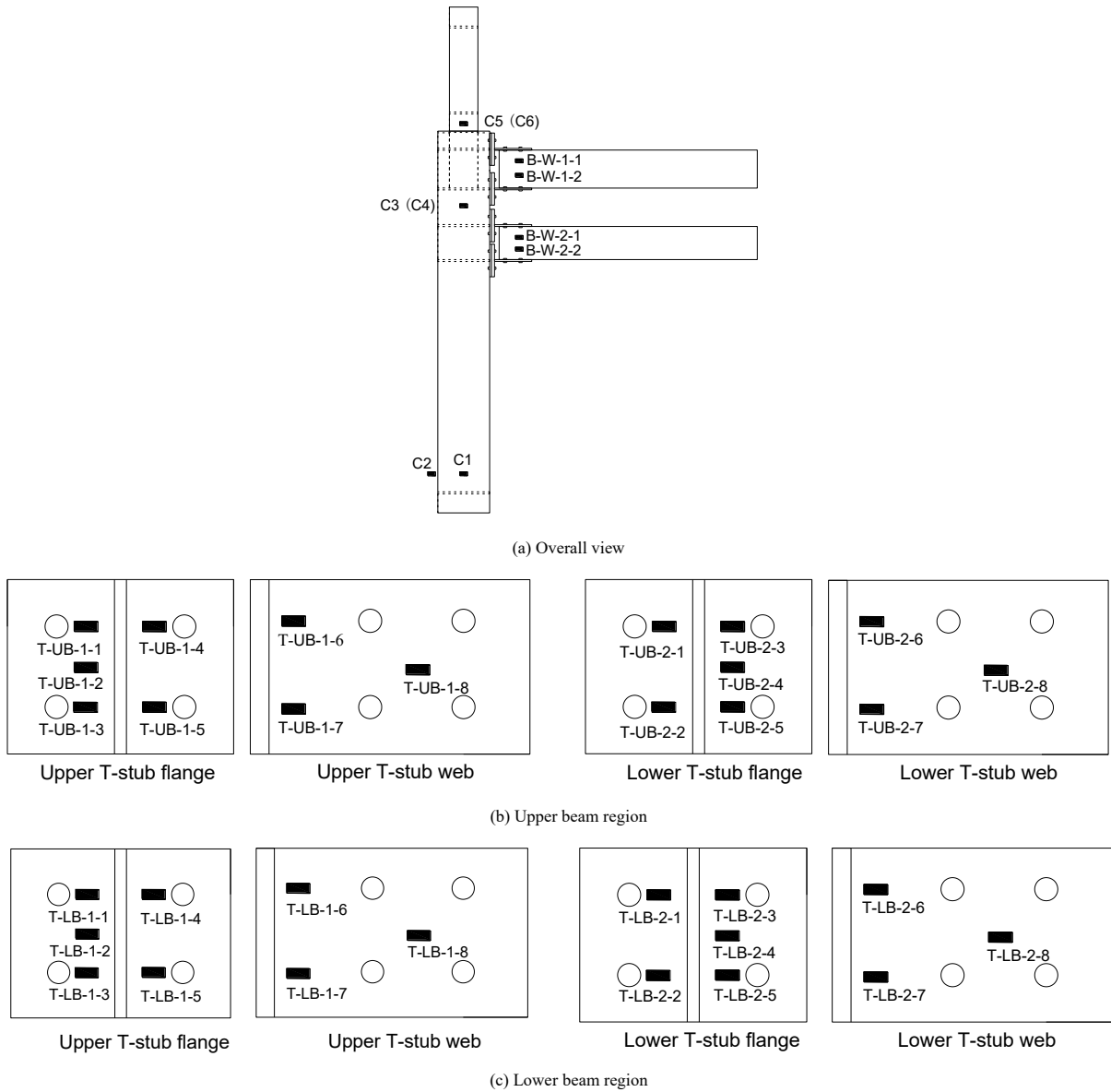


Fig. 6 Arrangement of strain gauges

3. Test results

3.1. Failure process of specimens

Taking the single-beam column joint JD3 as an example, the specimen

remained in the elastic stage during the initial loading phase due to the relatively small applied load. As the load gradually increased, the strain gauge on the lower flange of the T-stub was the first to reach the yield strain of the steel, indicating that local yielding initially occurred in the lower T-stub region. As the loading progressed, a slight gap appeared between the flange of the T-stub

and the external leveling plate, and minor slip occurred at the high-strength bolts at the beam end, as shown in Fig. 7(a). The strain gauges on the web of the lower T-stub and on portions of the beam also reached the yield strain of the steel. When the displacement reached 100 mm, a crack appeared at the junction between the flange and web of the lower T-stub, as shown in Fig. 7(b). Meanwhile, the strain gauge at the upper flange of the beam end reached the yield strain of the steel and increased rapidly, as shown in Fig. 7(c). Significant slip was also observed at the high-strength bolts at the beam end. As the displacement increased to 120 mm, a crack also developed at the junction between the flange and web of the upper T-stub, while the crack at the junction of the lower T-stub continued to propagate. Eventually, fracture occurred at the junction between the flange and web of the lower T-stub, and the test was terminated when the load-carrying capacity dropped by more than 85 percent from the peak load, as shown in Fig. 7(d). For the double-beam column joints, yielding of the T-stub in the upper beam occurred earlier than that in the lower beam. Failure was initiated by a crack at the junction between the flange and

web of the lower T-stub in the upper beam at a displacement of 80 mm. The crack propagated through the section at 100 mm and ultimately led to fracture at 120 mm.

Due to the structural characteristics of the double-beam joint, when fracture occurred at the junction between the flange and web of the lower T-stub in the lower beam, a crack had already developed at the corresponding location in the upper beam but had not yet fractured. At this stage, the specimen still maintained a relatively high load-carrying capacity. As the displacement increased to 140 mm, the crack in the upper beam further propagated and eventually fractured, marking the final failure of the specimen. The failure mode of the specimen is illustrated in Fig. 7.

The strain measurements of the blind bolts indicate that most bolts remained within the elastic range throughout the loading process. Post-test inspection also revealed no significant plastic deformation of the bolt shanks, suggesting that the blind bolts maintained stable fastening conditions during cyclic loading.

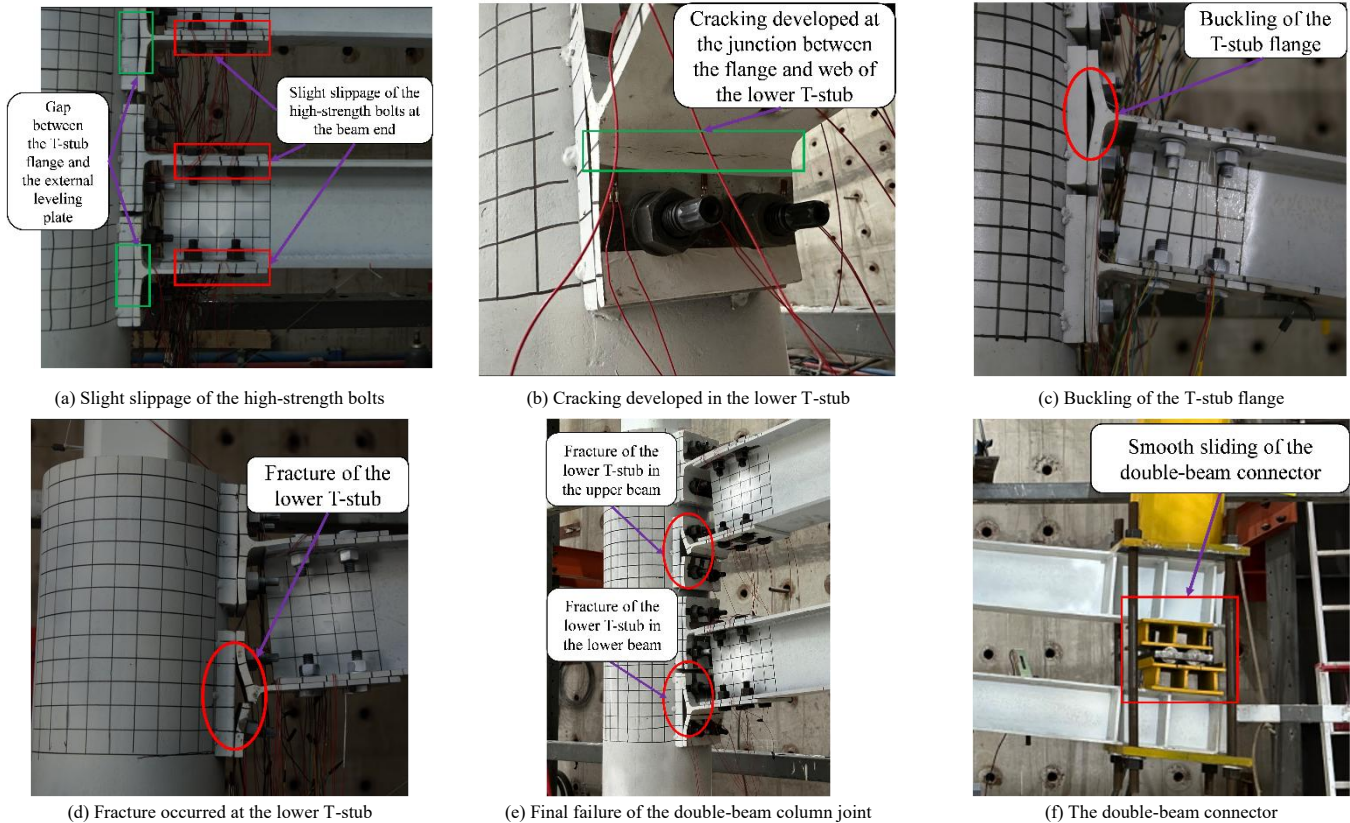


Fig. 7 Failure pattern of specimens

3.2. Failure modes of specimens

The failure modes of all specimens conformed to the strong-column weak-beam design principle. During the test, deformation and damage were primarily concentrated in the T-stub region, and no significant damage was observed in the other components. Joint failure was mainly attributed to the yielding and fracture of the T-stub under the combined action of shear and bending. A summary of the observed failure phenomena is provided in Table 3.

The failure progression of the specimens can be divided into three distinct phases: (i) the initial yielding stage, (ii) the deformation development stage characterized by local buckling and bolt slippage, and (iii) the final fracture stage, involving crack initiation and localized rupture. In the early stage of loading, initial localized yielding occurred in the lower T-stub region, highlighting it as the critical zone of stress concentration. Subsequently, a gap gradually developed between the flange of the T-stub and the external leveling plate, accompanied by slight slippage of the high-strength bolts at the beam end. As the loading continued, the strain gauges on the web of the lower T-stub and on portions of the beam flange successively reached the yield strain. During this loading phase, pronounced buckling deformation developed in the T-stub flange, and the specimen exhibited a significant reduction in stiffness, marking the onset of plastic behavior.

In the later stage of loading, an initial crack developed at the junction between the flange and web of the lower T-stub, accompanied by a sharp increase in bolt slippage, which significantly accelerated the accumulation of damage in the specimen. As deformation continued to progress, crack

propagation was also observed at the corresponding location of the upper T-stub. Eventually, fracture occurred at the junction between the flange and web of the lower T-stub. In the final loading stage, some high-strength bolts exhibited noticeable tensile deformation, while most of the other components remained in the elastic state without significant deformation. Although the double-beam and single-beam column joints differ in configuration, their failure modes were generally similar.

Under the same displacement loading, specimens JD2 and JD4, which had thinner T-stub webs, exhibited more pronounced slippage of the high-strength bolts at the beam ends, as well as larger gaps between the T-stub flange and the external leveling plate. The key difference was observed in the double-beam joint: after fracture occurred at the junction between the flange and web of the lower T-stub in the lower beam, the upper beam was still able to carry load, indicating a higher level of safety reserve.

To further interpret the fracture observed at the T-stub flange–web junction, the local stress mechanism of the flange was examined. In the tested specimens, the T-stub flange was connected by two rows of bolts, and the first bolt row was located approximately 48 mm from the web. Under the beam-end bending moment, the moment was transferred to the bolt group in the form of tensile forces, producing a bending effect in the T-stub flange. Because the web provides a restraint boundary for the flange, the distance between the bolt row and the web forms an effective lever arm that induces bending stresses in the flange section.

According to the equivalent T-stub model recommended in EN3 the flange bending resistance can be evaluated based on the plastic hinge mechanism of an

equivalent T-stub component. The mechanical model and the corresponding plastic hinge mechanism are illustrated in Fig. 8. Based on this mechanism, the tensile resistance of the equivalent T-stub flange can be calculated as follows:

$$F_T = \frac{4M_p}{m} \quad (1a)$$

$$M_p = 0.25 \sum l_{eff} t_f^2 / \gamma_M \quad (1b)$$

where F_T is the tensile resistance of the equivalent T-stub component, t_f is the flange thickness, m is the distance between the bolt row and the flange root considering the fillet radius, and l_{eff} is the effective plastic hinge length. In the present study, the T-stub has a fillet radius of 13 mm, resulting in $m=48-0.8 \times 13=37\text{mm}$. The effective length l_{eff} is taken as the smaller value of $2\pi m$ and $4m+1.25e$. The flange thickness is $t_f=14\text{mm}$.

By substituting the geometric parameters of the specimen into the above equations, the tensile resistance of the equivalent T-stub flange was calculated

as 234 kN. In the experiment, the actuator load corresponding to the first yielding of the strain gauge was 32 kN, which corresponds to a tensile force of approximately 227 kN in the T-stub flange. The close agreement between the calculated resistance and the experimental demand indicates that the flange-web junction approached its plastic capacity at that stage.

As the flange bends under bolt tension, contact between the flange edge and the leveling plate may develop, which can introduce a certain degree of prying action and increase the flange demand. Meanwhile, the leveling plates modify the load transfer path between the blind bolts and the tubular column by providing a flat contact interface, thereby alleviating direct bearing between the blind bolt nut and the curved column wall and reducing local stress concentration. Experimental observations show that the leveling plates remained stable during loading, and the deformation of the blind bolts after the test was comparable to that observed in conventional blind-bolted connections. Although the inclusion of leveling plates slightly increases the effective bolt length, the blind bolts adopted in the specimens were selected with a capacity higher than the design requirement, and no adverse influence on the overall mechanical behavior of the joint was observed.

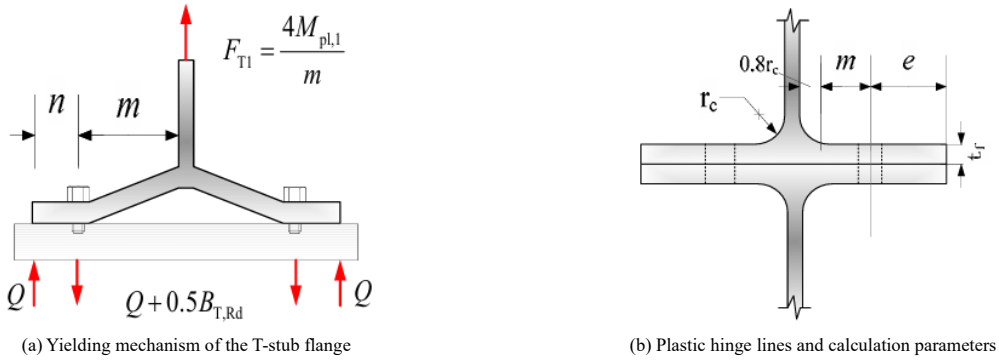


Fig. 8 Mechanical model and plastic hinge mechanism of the equivalent T-stub

Table 3

Main test phenomena of joints

NO	Elastic stage			Deformation development stage			Failure stage			
	Load level	Major events	Load level	Major events	Load level	Major events	Load level	Major events	Load level	Major events
JD1	δ_y	☆	$2\delta_y \sim 3\delta_y$	□◆★	$4\delta_y$	◆◆★	$5\delta_y$	▼	$6\delta_y$	▼▽
JD2	δ_y	☆	$2\delta_y \sim 3\delta_y$	□◆★	$4\delta_y$	◆◆★	$5\delta_y$	▼	$6\delta_y$	▼▽
JD3	δ_y	☆	$2\delta_y \sim 3\delta_y$	□◆★	$4\delta_y$	◆◆★	$5\delta_y$	▼	$6\delta_y$	
JD4	δ_y	☆	$2\delta_y \sim 3\delta_y$	□◆★	$4\delta_y$	◆◆★	$5\delta_y$	▼	$6\delta_y$	

Notes: “☆”denotes the minor slippage of the high-strength bolts at the beam end; “□” denotes the minor gaps appeared between the flange of the T-stub and the outer leveling plate; “★”denotes the significant slippage occurred in the high-strength bolts at the beam end; “◆” denotes the buckling deformation of the T-stub flange; “●”denotes the cracking at the T-stub flange-web junction; “▽”denotes the fracture of the lower T-stub on the upper beam; “▼” denotes the fracture of the lower T-stub on the lower beam.

3.3. Hysteresis loops

Fig. 9 presents the moment-rotation hysteresis curves at the beam ends of the specimens, where M and θ represent the bending moment and rotation, respectively. As shown in the figure, the hysteresis curves exhibit an overall spindle-shaped pattern and become progressively fuller with increasing loading, indicating favorable energy dissipation capacity. In the initial stage of loading, the specimen remained within the elastic range, and the moment-rotation relationship at the beam end was approximately linear. The hysteresis loops exhibited a small enveloped area, and no significant residual deformation was observed.

The hysteresis curves exhibit increasing fullness with the growth of loading displacement. This behavior can be attributed to three key mechanisms observed during cyclic loading: (i) progressive development of the gap between the T-stub flange and the backing plate, (ii) buckling deformation of the T-stub flange, and (iii) increasing slippage of the high-strength bolts at the beam end. In the later stage of loading, a crack developed and eventually fractured at the junction between the T-stub flange and web, resulting in a sharp drop in the hysteresis curve under negative loading.

Similar experimental observations have been reported in Ref. [44], where T-stub single-sided bolted connections with square steel tubular columns were investigated. Since the surface of the square steel tube is flat, leveling plates were not required in that study. The reported failure modes and hysteresis

characteristics suggest that the cyclic behavior of such connections is mainly governed by T-stub deformation and bolt slip mechanisms.

3.4. Skeleton curve

From the moment-rotation hysteresis curves of the specimens (Fig. 9), the peak points of the first cycle at each loading level were selected and sequentially connected to generate the skeleton curves, as shown in Fig. 9. As illustrated in Fig. 9(e), the skeleton curves of all specimens exhibit similar trends, each undergoing three typical development stages: the elastic stage, the yielding stage, and the failure stage. Before slippage of the high-strength bolts at the beam end occurred, the skeleton curves of similar joints were nearly identical, indicating comparable initial stiffness and load-bearing capacity. As bolt slippage developed and the T-stub began to yield, the skeleton curves started to bend. With continued loading, significant plastic deformation gradually developed in the T-stub flange, and the growth of the skeleton curves became progressively slower. Ultimately, under repeated cyclic loading, fracture occurred at the junction between the flange and web of the lower T-stub, resulting in a rapid reduction in load-carrying capacity and a clear descending segment in the skeleton curve. In addition, compared with the single-beam column joint, the double-beam column joint exhibited significantly higher load-carrying capacity. Furthermore, an appropriate increase in the thickness of the T-stub was found to effectively improve the load-carrying capacity of the joint.

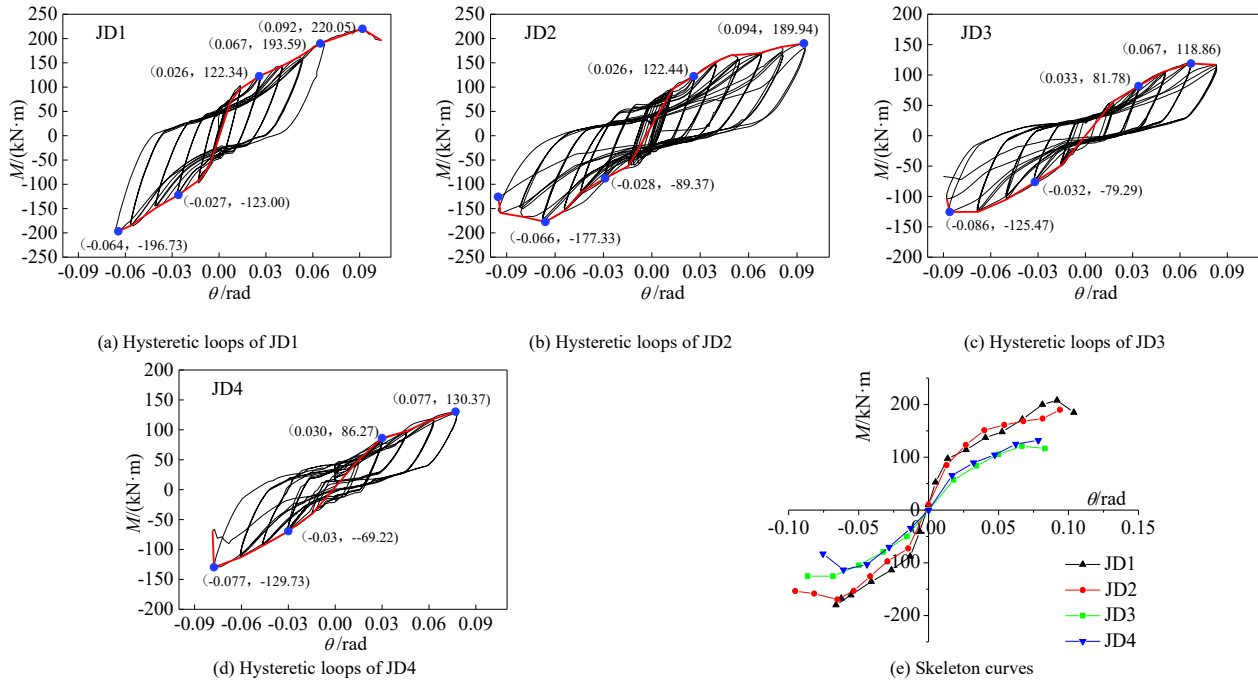


Fig. 9 Hysteretic loops and Skeleton curves of specimens

3.5. Deformation and energy dissipation capacity

In this study, rotational ductility was used to evaluate the deformation capacity of the joint. It is defined as the ratio of the ultimate rotation to the yield rotation, and is calculated as follows:

$$\mu = \frac{\theta_u}{\theta_y} \quad (2)$$

where: θ_u is the rotation angle at joint failure, θ_y is the rotation angle at the yielding of the joint, and μ is the ductility factor. The calculated ductility factors for each specimen are presented in Table 4.

The average ductility coefficient of the joints ranged from 2.62 to 3.12, with the values of the double-beam column joints significantly higher than those of the single-beam column joints. The primary reason is that after fracture of the T-stub in the lower beam, the upper beam was still able to carry load, thereby providing a higher level of safety reserve. As loading progressed, the T-stub underwent large plastic deformation and eventually fractured, leading to a relatively lower overall ductility coefficient.

The energy dissipation capacity of structural members under cyclic loading was assessed using the equivalent viscous damping coefficient h_e , calculated as follows:

$$h_e = \frac{1}{2\pi} \cdot \frac{S_{AABC} + S_{AADC}}{S_{AOBE} + S_{AODF}} \quad (3)$$

where $S_{AABC}+S_{AADC}$ denote the area of hysteretic curves of ABCDA, and $S_{AOBE}+S_{AODF}$ denote the merging area of triangles OBE and ODF, as shown in Figure 12. The calculated results of the test specimens are shown in Table 5.

As shown in Table 5, the equivalent viscous damping ratios of the double-beam column joints and the single-beam column joints ranged from 0.151 to 0.212 and from 0.136 to 0.241, respectively. The equivalent viscous damping ratio of all specimens initially increased and then gradually decreased during the loading process. This trend was primarily attributed to the tight connection between components during initial loading, which resulted in significant energy dissipation through friction and a slight increase in the equivalent viscous damping ratio. With the increase in loading cycles, plastic deformation of the T-stub flange continued to develop, and cracks formed at the junction between the flange and the web, resulting in localized damage. These effects weakened the energy dissipation capacity of the structure and led to a reduction in the equivalent viscous damping ratio. Under the same loading level, the double-beam column joints exhibited superior energy dissipation performance compared with the single-beam column joints. This was mainly because energy dissipation was primarily provided by the T-stubs, and the double-beam column

joints contained a greater number of T-stubs due to the presence of two beams, thereby offering enhanced energy dissipation capacity.

Table 4
Ductility coefficient

Specimen ID	Yield rotation θ_y/rad	Ultimate rotation θ_u/rad	μ
JD1	0.026	0.081	3.12
JD2	0.028	0.082	2.93
JD3	0.032	0.084	2.62
JD4	0.028	0.077	2.75

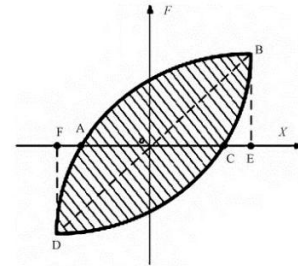


Fig. 10 Equivalent viscous damping coefficient calculation

Table 5
Equivalent viscous damping coefficient of specimens

Specimen ID	h_e				
	$1\delta_y$	$2\delta_y$	$3\delta_y$	$4\delta_y$	$5\delta_y$
JD1	0.176	0.212	0.197	0.197	0.151
JD2	0.175	0.204	0.179	0.161	0.155
JD3	0.188	0.205	0.165	0.145	0.137
JD4	0.199	0.241	0.162	0.150	0.136

The energy dissipation performance of each specimen under different loading levels is shown in Fig. 11. At the initial stage of loading, the energy dissipation of the specimens was relatively low. As the flange of the T-stub gradually entered the buckling stage, the energy dissipation capacity of the specimens was significantly enhanced. For the double-beam column joints, the

energy dissipation increment rates at each loading level compared with the previous level were 257.60%, 46.31%, 35.27%, and 29.17%, respectively. For the single-beam column joints, the corresponding increment rates were 132.00%, 84.41%, 44.48%, and 24.38%, respectively. The maximum energy dissipation of the double-beam column joints reached 11.70 kJ, which was significantly higher than the 8.69 kJ observed in the single-beam column joints, indicating that the double-beam column joints possessed superior energy dissipation capacity.

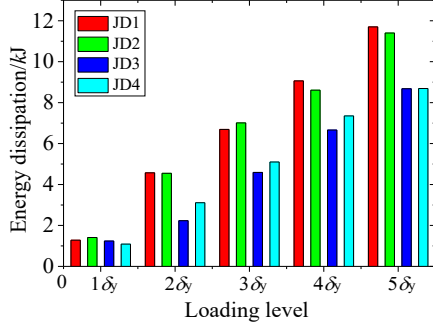


Fig. 11 Energy dissipation of specimens

3.6. Stiffness degradation

The secant stiffness was used to characterize the rotational stiffness of the joint at different load levels, as defined below:

$$K_i = \frac{|+M_i| + |-M_i|}{|+\theta_i| + |-\theta_i|} \quad (4)$$

where K_i is the moment-rotation stiffness at the i th cyclic load level; M_i denotes the maximum moment capacity at the i th cyclic load level; θ_i represents the rotation corresponding to the maximum moment capacity at the i th cyclic load level; and the sign convention follows positive for push and negative for pull directions.

The stiffness degradation curves of the specimens are shown in Fig. 12. With the gradual increase in loading levels, all joints exhibited a clear trend of stiffness degradation, which was mainly attributed to the continuous accumulation of plastic deformation in the T-stubs and the occurrence of slippage between components. Under the same loading level, the stiffness of the

double-beam column joints was significantly higher than that of the single-beam column joints. For example, the initial stiffness of the double-beam column joint JD2 and the single-beam column joint JD4 was 5632.36 $\text{kN}\cdot\text{m}\cdot\text{rad}^{-1}$ and 3907.99 $\text{kN}\cdot\text{m}\cdot\text{rad}^{-1}$, respectively. The initial stiffness of JD2 was approximately 44.12% higher than that of JD4.

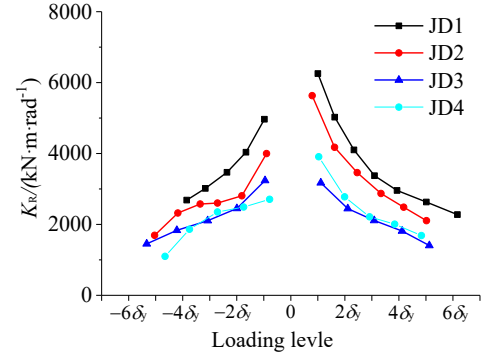
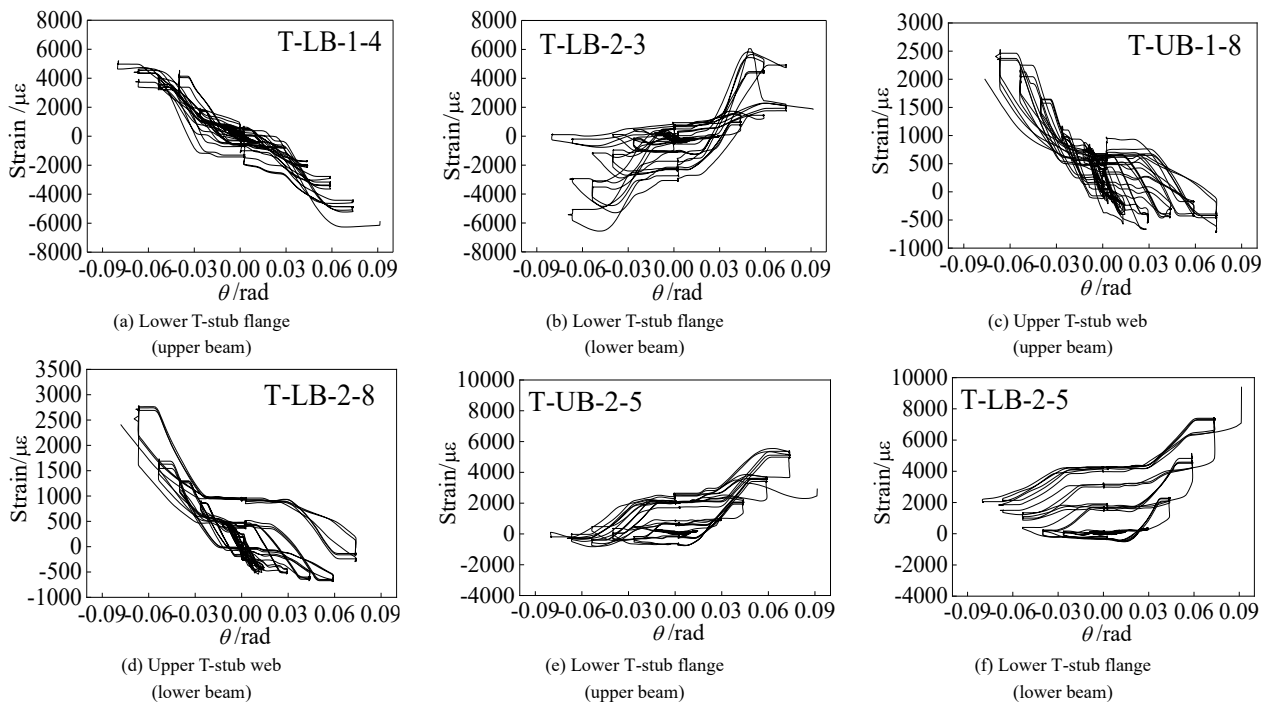


Fig. 12 Stiffness degradation

3.7. Strain analysis

The strain variations at the main measurement points of each component are shown in Fig. 13. During the test, the strain values at the beam and column locations did not exceed 500 $\mu\epsilon$, indicating that these regions remained within the elastic range throughout the loading process and that the steel did not reach the yield strain.

The strain analysis indicated that plastic deformation of the joint was primarily concentrated in the flange of the T-stub. The strain in the T-stub flange was the first to reach the yield strain of the steel and increased rapidly thereafter. Most measurement points recorded strain values exceeding 4000 $\mu\epsilon$, which was consistent with the buckling behavior of the T-stub flange observed during the test. As the loading progressed, the web of the T-stub also reached the yield strain. In the later stage of loading, cracks appeared at the junction between the flange and the web of the T-stub and propagated rapidly, eventually leading to fracture. This phenomenon was consistent with the experimental observations. In addition, the strain values in both the flange and the web of the lower T-stub were greater than those of the upper T-stub, which was consistent with the observed failure mode of the double-beam column specimen JD2, where fracture occurred in the lower T-stub.



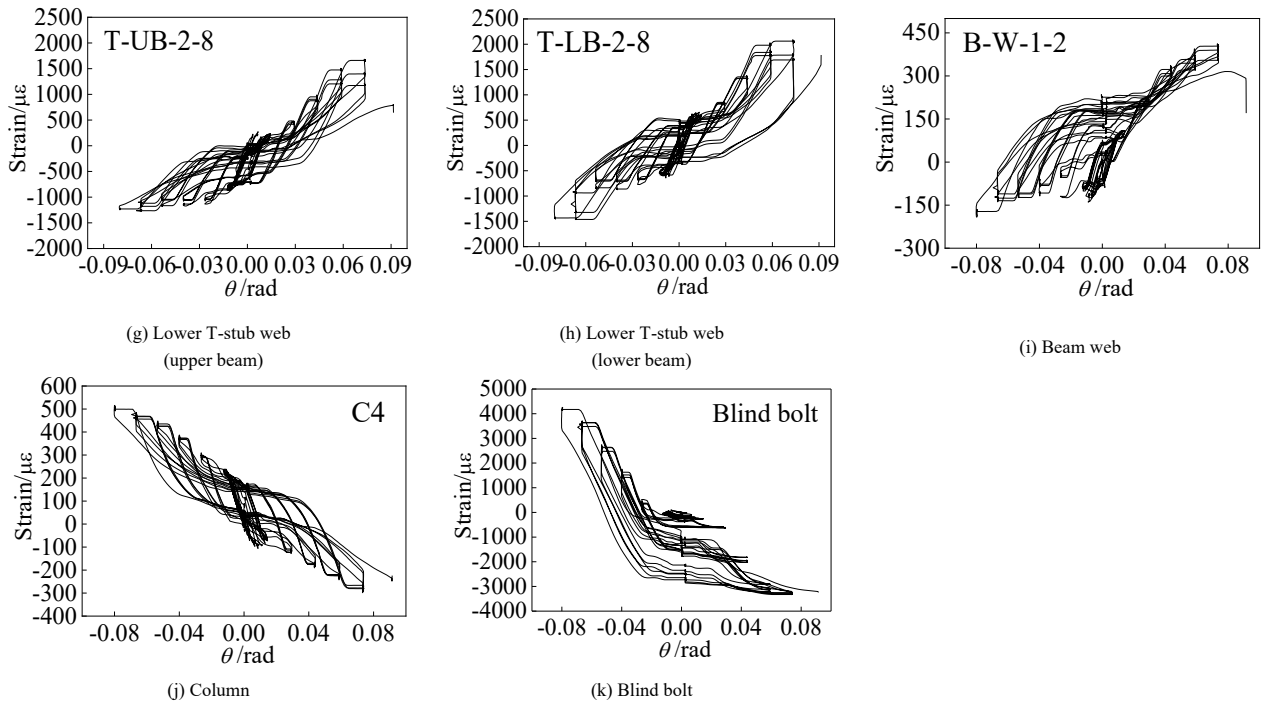


Fig. 13 Strain curves of strain gauge

The strain in the blind bolts gradually increased with the increment of loading displacement. However, the strain values at most measurement points did not exceed $4000 \mu\epsilon$ and remained within the elastic range. Only a few bolts entered the yielding stage in the later phase of loading. After the test, an inspection was conducted on the dismantled blind bolts. Slight wear was observed at the threaded portion, while no obvious deformation was found on the bolt shanks, indicating that the blind bolts exhibited good fastening performance in this type of joint.

4. Numerical analysis

4.1. Establishment of numerical model

To further investigate the plastic development and failure modes of irregular steel joints in modern Chinese traditional-style buildings with blind bolts, a refined finite element model was established in ABAQUS. The numerical model of a typical specimen is illustrated in Fig. 14. All components adopted C3D8R solid elements. Local mesh refinement was applied in the region of the T-stub connections. The mesh size in the core region of the beam-to-column joint was controlled within 5 mm to accurately capture the distribution of plastic strain fields, while the mesh size of beam and column members away from the joint gradually increased to 10–15 mm. A smooth mesh transition was adopted to achieve a balance between computational accuracy and efficiency. The circular column and the RHS column were connected through horizontal stiffening rings and vertical stiffeners, with welds simulated by tie constraints. The contact interaction among all components (including the blind bolts, beams, columns, and leveling plates) was defined as surface-to-surface contact. Hard contact was applied in the normal direction, while Coulomb friction with a penalty formulation was adopted in the tangential direction. The friction coefficient was set to 0.3, consistent with the experimental conditions.

The double-beam connectors between the upper and lower beams in the tested double beam-column joints were simulated using SLOT elements, as shown in Fig. 14. This connector allows relative horizontal displacement between the two beams while restricting vertical movement, thereby preventing the transfer of bending moments between them. Through this configuration, the beam-column joint region is subjected primarily to bending moment and shear force induced by the actuator, which reproduces the mechanical condition of a joint located between two beam inflection points in a frame structure. The effectiveness of SLOT elements in simulating this type of double-beam connection has been verified in the literature [13].

Reference points were established at the column base, column top, and beam end, which were coupled to the surrounding nodes to apply boundary conditions and loading. The loading process was divided into four analysis steps to ensure computational convergence. First, the design pretension force of 170

kN was applied to the blind bolts using the Bolt Load tool. Then, the “fix at current length” option was activated to maintain the bolt length in the subsequent analysis steps, meaning that the pretension force was assumed to remain constant during cyclic loading. In this study, the potential relaxation or loss of bolt pretension under cyclic loading was not explicitly considered. Subsequently, an axial load of 275 kN was applied at the column top. Finally, cyclic displacement loading was imposed at the beam end. To avoid stress concentration during loading, the beam end was coupled to a reference point, and the load was applied through this reference point. The column base was modeled as a hinged support ($U_1=U_2=U_3=UR_2=UR_3=0$). The general static analysis procedure was adopted, taking into account both geometric and material nonlinearities.

Since some specimens shared the same geometric dimensions and connection details, with only a minor difference in the beam flange thickness, this variation was considered to have limited influence on the overall load-bearing mechanism and failure mode. Therefore, finite element modeling was conducted only for the typical specimens.

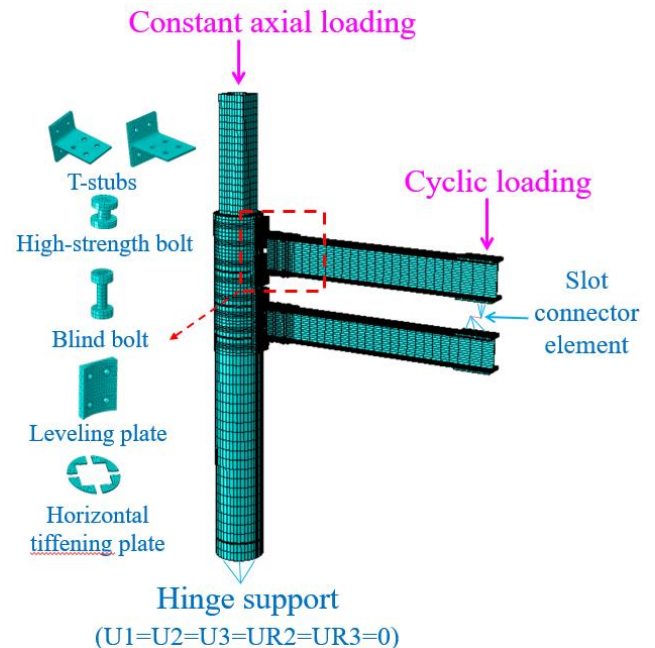


Fig. 14 Finite element model

The stress-strain relationship of the steel was represented by a three-linear-segment model proposed in literature [13], with the material parameters calibrated based on the tensile coupon tests listed in Table 2. Both the blind bolts and the high-strength bolts were modeled using a linear hardening material model.

4.2. Model verification

Taking the typical double-beam-column specimen JD2 as an example, the FEA results were compared with the experimental observations, as shown in Fig. 15. The deformation contour from FEA indicates that the specimen deformation was primarily concentrated in the T-stub region, with minor slippage observed at the high-strength bolts connecting the T-stub web to the beam end. The stress contour shows that, at the initial loading stage, the high-strength bolts at the beam end and the blind bolts at the column end reached relatively high stress levels, indicating that the T-stub effectively transferred bending moments and shear forces between the beam and column ends. As loading progressed, gaps gradually appeared between the upper and lower T-stub flanges and the outer leveling plate, accompanied by slight slippage of the beam-end high-strength bolts.

Further analysis revealed that the junction between the T-stub web and flange first reached the steel yield stress, which corresponded with the yielding detected by the strain gauges at the same location in the experiment. In the later loading stage, stress in this region continued to increase and expand, while the tension-side flange underwent significant bending, with the web arching upward. The FEA results were consistent with the experimentally observed cracking and fracture locations of the flange, and the model was able to reproduce the relative slippage between the two beams, indicating that the simulation of the double-beam connector was effective, in agreement with the findings of literature [14].

The stress distribution further demonstrated that, under cyclic loading at the beam end, both blind bolts and high-strength bolts exhibited high stress levels, with the maximum stress of the blind bolts reaching approximately 850 MPa, close to the nominal yield stress of 900 MPa. At the end of the test, slight thread wear was observed, while no noticeable deformation of the bolt shank occurred, which was consistent with the simulation. Limited by the modular constraints of traditional Chinese architecture, only four high-strength bolts and blind bolts could be arranged on each side. However, their calculated load-bearing capacity satisfied the strong-column weak-beam design principle, resulting in the primary deformation being concentrated in the T-stub.

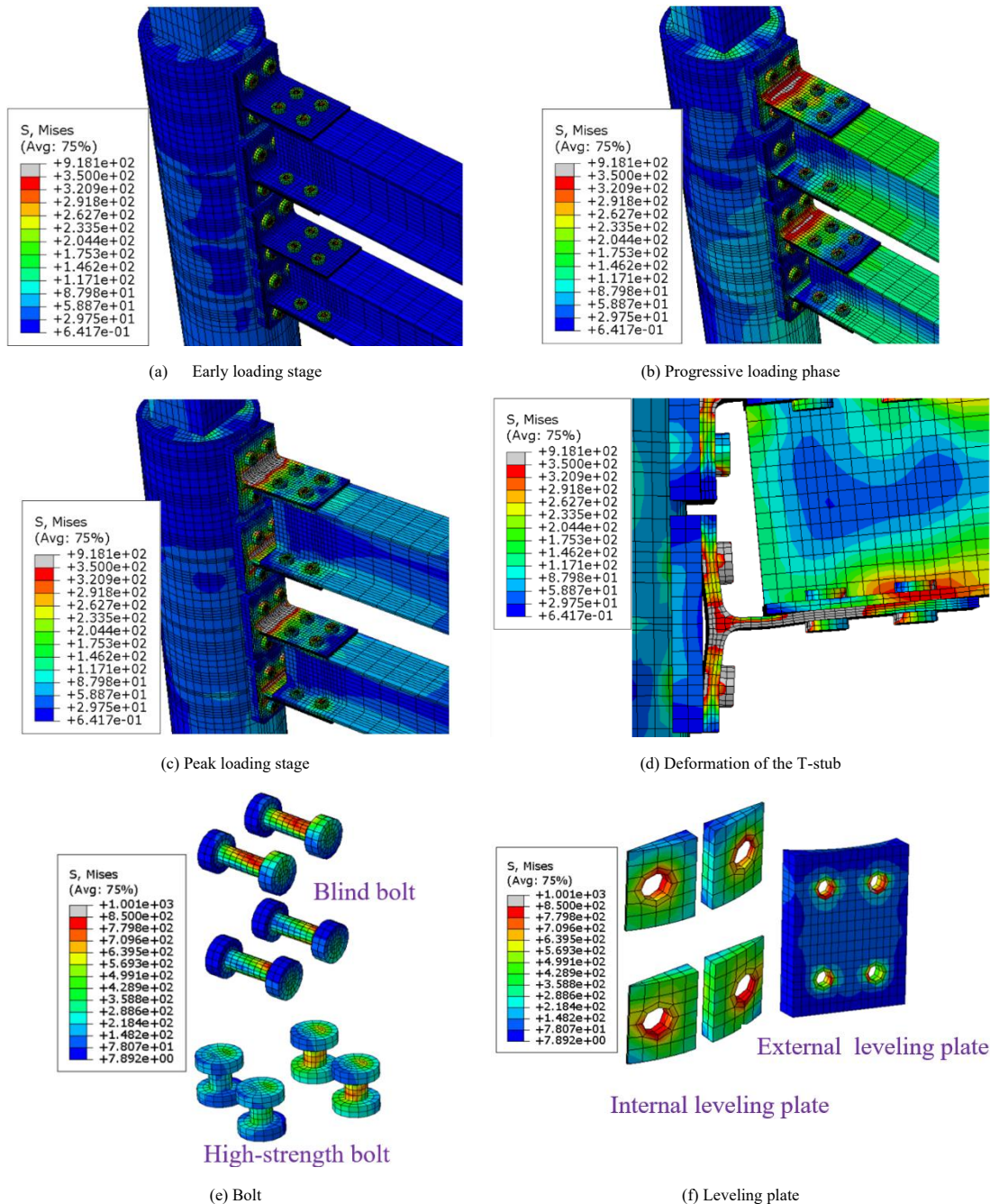


Fig. 15 Deformation of FEA

FEA also indicated that the bolt holes at the inner and outer leveling plates connecting the T-stub and the CHS column experienced high stress levels, due to the large beam-end displacements inducing significant deformation between the T-stub and the column, which caused the blind bolt shanks to press against the hole edges. Based on previous experimental experience in the research group, protective sleeves were added to the blind bolts in this test to reduce wear, and only minor wear of a few blind bolts was observed. The deformation of the core region in the FEA resembled that under monotonic loading, primarily because the simulation could not fully capture the cumulative deformation and damage under low-cycle reversed loading. Overall, however, the FEA results matched the experimental observations well, confirming the reliability of the model.

During the specimen design, the T-stub width was selected to comply with the layout requirements of traditional Chinese architecture. Based on prior studies in the research group, the flange and web thickness of the T-stub were specifically designed to ensure that the T-stub would deform preferentially, satisfying the strong-column weak-beam principle while maintaining adequate energy dissipation. Both the experimental and FEA results indicate that the blind bolt-T-stub connection in these irregular steel joints for MCTBs is structurally reasonable, with clear load transfer paths, expected deformation patterns, and favorable seismic performance.

4.3. Comparison of moment-rotation hysteresis curves

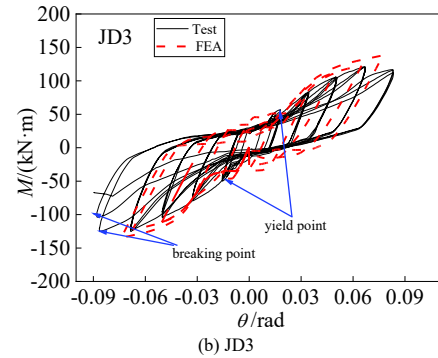
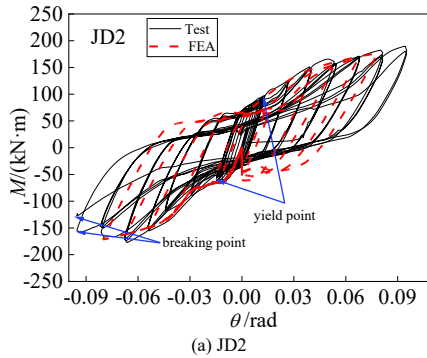


Fig. 16 Comparison of hysteresis curves of typical specimens

5. Damage index calculation and damage assessment

Seismic damage of structural components is typically evaluated based on deformation and hysteretic energy dissipation. To evaluate the seismic damage of components in steel MCTBs, Qi et al. [42] proposed a damage index (DDD) to evaluate the seismic damage of components in steel MCTBs, based on a two-parameter damage model that simultaneously considers deformation and energy dissipation. This index aims to quantitatively describe the influence of damage on the restoring force behavior of steel MCTB components [43]. Liu et al. [44] proposed recommended threshold ranges of the damage index for composite joints of concrete-filled steel tube columns with blind bolts under seismic action, which can be used to evaluate the damage level of structural components.

The results of this study indicated that the seismic damage of irregular steel joints in MCTBs was primarily caused by plastic deformation occurring at the T-stub region. In this study, a modified Park-Ang model that accounts for residual deformation accumulation was adopted to calculate the damage index D of the joint specimens. The calculation formula is expressed as follows in Eq. (5):

$$D = (1 - \beta) \frac{\delta_m - \delta_y}{\delta_u - \delta_y} + \beta \frac{\int dE}{f_y \delta_u} \quad (5)$$

where δ_m is the maximum response deformation under the seismic effect, δ_u and δ_y are the ultimate deformation capacity under monotonic loading and the yield deformation corresponding to the yield point of the specimen, respectively, f_y is the steel yield strength, dE is the cumulative energy dissipation of plastic deformation, and β is a non-negative weighting factor, which is taken as 0.025 for steel structures [45].

To determine a suitable value of the parameter β for the studied joints, the parameter was calibrated using the experimental results. Assuming that the damage index reaches $D=1$ at the ultimate failure state of the specimens, the parameter β was back-calculated for each specimen based on the experimental data using Eq. (6).

Fig. 16 presents the comparison of the moment-rotation hysteresis curves obtained from the experiment and the FEA. At the early loading stage, the hysteresis curves observed in the experiment were closely reproduced by the FEA results, indicating that the numerical model can accurately capture the initial stiffness and elastic response of the joint. As the loading progressed into the elasto-plastic stage, noticeable discrepancies between the experimental and numerical hysteresis loops were observed. The experimental curves exhibited pronounced asymmetry, with the negative peak moment being approximately 5%-8% higher than the positive peak. This asymmetry was mainly attributed to inevitable fabrication imperfections, initial geometric deviations of the specimen, and slight eccentricities in the loading system during the experiment, which were not included in the idealized numerical model. In addition, during cyclic loading, repeated opening and closing of cracks occurred in the T-stub flange region, accompanied by local plastic deformation of the flange. These cumulative damage effects led to stiffness degradation and energy dissipation in the experiment. In contrast, the finite element model adopted an idealized material constitutive relationship and did not explicitly consider crack propagation and damage evolution under low-cycle loading. Consequently, the hysteresis loops obtained from the FEA appear relatively fuller than those observed in the experiment.

Despite these differences, the overall development trend of the hysteresis curves and the peak moment values were reasonably predicted by the FEA results, indicating that the numerical model can effectively reflect the overall seismic response characteristics of the joint.

$$\beta = \frac{f_y(\delta_u - \delta_m)}{\int dE - f_y(\delta_m - \delta_y)} \quad (6)$$

The calculated combination coefficients β are summarized in Table 6. The results indicate that the average value of β is 0.018 with a standard deviation of 0.003.

To further evaluate the influence of the parameter value on the damage index, a comparative analysis was carried out using two parameter values: $\beta = 0.018$ obtained from the experimental back-calculation and $\beta = 0.025$. The corresponding maximum damage indices of the specimens are presented in Table 7.

The results show that when $\beta = 0.018$, the calculated damage indices at the ultimate failure state are closer to the theoretical failure value $D=1$, and the dispersion of the results is slightly reduced compared with those obtained using $\beta = 0.025$. Therefore, considering both calculation accuracy and parameter stability, $\beta = 0.018$ was adopted in the subsequent damage index analysis in this study.

The damage index D of the joints was calculated using Eq. (5), and the results are presented in Table 8. Based on the classification criteria listed in Table 6, the corresponding damage levels were determined, as illustrated in Fig. 17 and 18. The damage evolution of the joint can be divided into five stages: (i) In the initial loading stage, slight slippage occurred at the high-strength bolts connecting the web of the T-stub to the beam end. The damage index was less than 0.25, corresponding to minor damage (see Fig. 7a). (ii) Small gaps appeared between the upper and lower T-stub flanges and the external backing plate, with the damage index remaining below 0.45, still classified as minor damage (see Fig. 7a). (iii) Buckling occurred in the T-stub flange, and the joint entered a distinct plastic deformation development stage, corresponding to moderate damage (see Fig. 7b). (iv) A crack developed at the junction between the lower T-stub flange and the web, accompanied by significant slippage of the high-strength bolts. The damage expanded rapidly, corresponding to severe damage (see Fig. 7c). (v) In the single-beam joint, fracture occurred at the junction between the lower T-stub flange and the web. In the double-beam joint,

sequential fractures occurred at the corresponding junctions of the lower T-stubs in the lower and upper beams, resulting in complete damage (see Fig. 7d).

Under various loading levels, the damage indices of the double-beam joint specimens (JD1 and JD2) consistently remained lower than those of the single-beam joint specimens (JD3 and JD4), indicating a lower degree of damage. At the $4\delta_y$ loading level, the double-beam column joints were still in the moderate damage stage, whereas the single-beam column joints had already entered the severe damage stage. When the loading reached $5\delta_y$, the lower T-stub in the single-beam column joints fractured, and the damage index exceeded 1.0, indicating that complete failure had occurred. At this stage, the upper T-stub in the upper beam of the double-beam column joints remained undamaged, and the specimen had not yet reached complete failure. Further loading to $6\delta_y$ caused fracture of the upper T-stub in the lower beam of the double-beam column joints, marking the onset of complete failure.

The overall trend of the damage index curves obtained in this study was generally consistent with the experimental results reported by Liu et al. [44], primarily due to the similarity in joint configuration, as both adopted blind bolt-T-stub connections. However, RHS columns were used in the study by Liu et al., whereas CHS columns were adopted in the present study. Moreover, a pair of internal and external leveling plates was introduced at the interface between the column and the T-stub to optimize the bolt force transfer path and improve the overall mechanical stability of the connection. The similarity in damage evolution trends confirms that the proposed blind bolt-T-stub connection for irregular steel joints in MCTBs provides reliable structural performance. It achieves performance comparable to that of conventional RHS column connections and offers a feasible and adaptable solution for practical application

in MCTBs.

Table 6

Calculation results of model parameter

Specimen	$f_y(\delta_a - \delta_m)$	$ dE - f_y(\delta_m - \delta_y) $	β
JD1	655.22	38628.28	0.017
JD2	950.78	40131.50	0.023
JD3	305.18	19801.93	0.015
JD4	400.23	253.67.45	0.016

Table 7

Calculation result of damage index

Specimen No	$D_{max} (\beta=0.018)$	$D_{max} (\beta=0.025)$
JD1	1.112	1.156
JD2	1.138	1.192
JD3	1.097	1.135
JD4	1.128	1.178
Mean	1.118	1.1653
Standard Deviation	0.0180	0.0250

Table 8

Evaluation of seismic damage

Damage index	Damage description	Major events		Damage grade
		Single beam-column joint	Double beam-column joint	
0-0.25	None	☆	☆	Basical intactness
0.25-0.45	Damage occurs	☆□	☆□	Minor damage
0.45-0.65	Damage visible	□★◆	□★◆	Moderate damage
0.65-1	Damage develops	★◆●	★◆●	Severe damage
1-1.1	Destruct	▼	▼+▽	Complete failure

Notes: “☆”denotes the minor slippage of the high-strength bolts at the beam end; “□” denotes the minor gaps appeared between the flange of the T-stub and the outer leveling plate; “★”denotes the significant slippage occurred in the high-strength bolts at the beam end; “◆” denotes the buckling deformation of the T-stub flange; “●”denotes the cracking at the T-stub flange-web junction; “▽”denotes the fracture of the lower T-stub on the upper beam; “▼” denotes the fracture of the lower T-stub on the lower beam; “▼”+“▽” denotes the sequential fractures of the lower T-stubs on the lower beam and the upper beam.

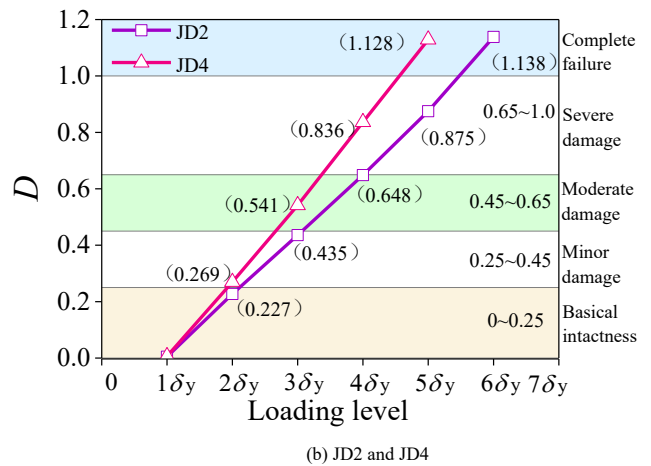
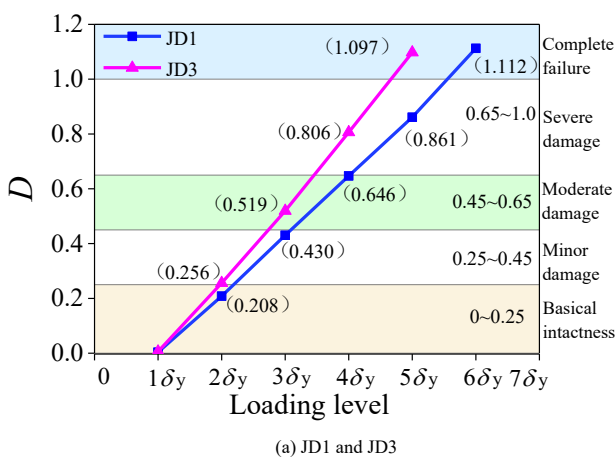


Fig. 17 Damage index distribution

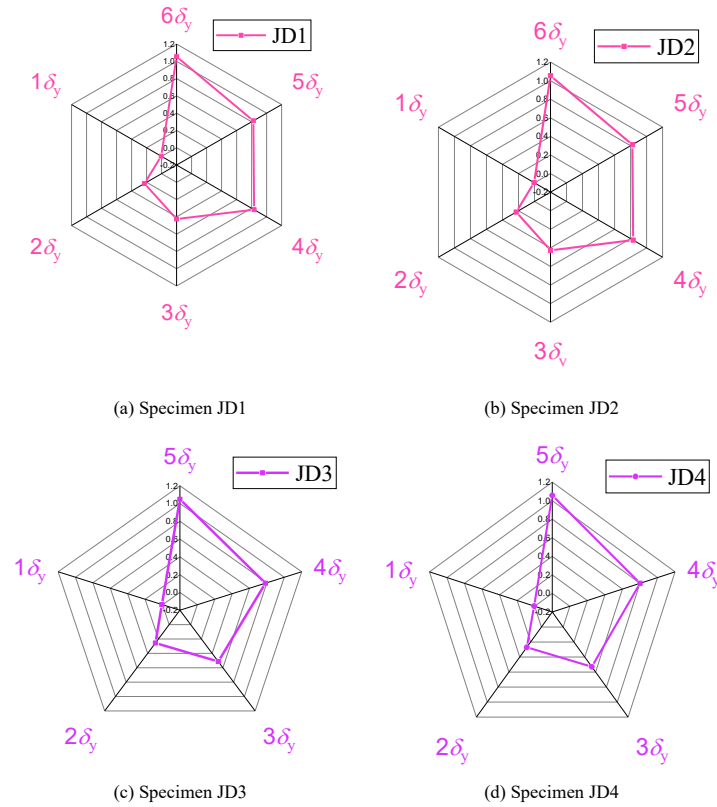


Fig. 18 Damage index curve

6. Conclusions

To address the key technical challenges associated with beam-column connections in prefabricated MCTBs, this study proposed a novel bolted connection scheme for irregular steel joints in MCTBs, based on blind bolt-T-stub assemblies. Low-cycle cyclic tests were conducted to investigate the mechanical performance of the proposed joints, and seismic damage evaluation was performed. The main conclusions are summarized as follows:

(1) All specimens exhibited failure modes primarily concentrated in the T-stub region, including flange buckling, crack initiation and propagation, and eventual fracture. These failure characteristics conformed to the strong-column weak-beam seismic design principle. In the case of double-beam column joints, the upper beam remained capable of bearing loads even after the lower beam had failed, effectively delaying overall structural failure. This behavior highlights the inherent load-carrying redundancy and improved seismic resilience of MCTBs.

(2) Under low-cycle reversed loading, all joints exhibited stable and full hysteresis loops, indicating excellent energy dissipation capacity and pronounced plastic deformation. Compared with single-beam joints, the double-beam configuration exhibited improved hysteretic behavior, higher ductility, and greater energy dissipation capacity. These enhancements are mainly due to the presence of an increased number of T-stub components in the double-beam column joints, which provided greater deformation capacity and delayed the initiation and propagation of damage.

(3) The double-beam column joints exhibited significantly higher load-bearing capacity than the single-beam column joints, and increasing the thickness of the T-stub further improved the overall structural performance. The observed stiffness degradation was primarily attributed to the accumulation of plastic deformation in the T-stub and slippage between the connecting components. Analytical results revealed that the double-beam column joints exhibited greater initial and residual stiffness, indicating enhanced deformation resistance and superior structural ductility.

(4) The refined finite element (FEA) model accurately captured the experimental deformation patterns, stress distribution, and moment-rotation hysteresis behavior of the joints, including the relative slippage of the double-beam connectors. Minor discrepancies between the numerical and experimental hysteresis loops can be attributed to material imperfections, steel fatigue, bolt slippage, and tolerances in the loading apparatus.

(5) Based on the damage assessment results obtained using the modified Park-Ang model, the double-beam column joints exhibited a significantly lower rate of damage accumulation than the single-beam column joints, along with a slower failure progression and superior seismic ductility. Notably, after the

failure of the lower beam, the upper beam remained capable of bearing loads, thereby significantly enhancing overall structural stability and safety redundancy. These findings suggest that the double-beam design concept, inspired by traditional architectural practices, retains strong adaptability and practical engineering value in MCTBs constructed with steel structures.

(6) Compared with RHS column joints connected using blind bolts, the proposed irregular steel joints in MCTBs demonstrate comparable connection strength and ductility, while providing superior load-bearing capacity and enhanced safety redundancy due to the double-beam configuration, which is derived from the unique double-beam joint system evolved in ancient Chinese timber architecture. This connection strategy effectively addresses the engineering challenge of bolted assembly for closed-section columns in MCTBs and presents a practical and adaptable structural solution for their prefabricated and industrialized application.

It should be noted that only the T-stub thickness was varied in the present parametric study due to the constraints associated with using split T-sections from standard rolled H-sections and the architectural configuration requirements. The selected parameter range aims to ensure a ductile failure mechanism governed by T-stub yielding, while avoiding undesirable failure modes such as bolt pull-out or column wall damage. A more comprehensive parametric investigation will be conducted in future studies.

CRedit authorship contribution statement

Zhanjing Wu: Writing - review & editing, Resources. Jinshuang Dong: Writing - original draft, Data curation. Yan Sui: Writing - review & editing, Funding acquisition. Jianing Wang: Data curation, Formal analysis.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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