

Slotted-hole bolted joint with top-and-seat angles for eccentric core-tube frame structures: Experiments and simulations

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ABSTRACT This study proposes a slotted-hole bolted joint with top-and-seat angles (TSA-SHBJ) for beam-to-column connections in eccentric core-tube high-rise structures. The TSA-SHBJ accommodates structural eccentricity through controlled rotational freedom while utilizing top and seat angles as replaceable energy dissipation components. Cyclic loading experiments are conducted to examine the failure mechanism and mechanical performance of TSA-SHBJ in comparison with SHBJ. An experimentally calibrated equivalent model of TSA-SHBJ is developed and integrated within the numerical model of an eccentric core-tube frame structure to investigate the dynamic performance improvement of this enhanced structural system under both wind and seismic excitations. Results reveal the phased failure mechanism of TSA-SHBJ, progressing from friction-slip to bolt failure, which ensures ductile collapse resistance. TSA-SHBJ shows superior performance over SHBJ joints, demonstrating 99.5% higher rotational stiffness, 35.6% greater ductility, and 9.7-fold energy dissipation capacity. The eccentric core-tube frame structure integrated with TSA-SHBJs achieves more than 30% vibration response reduction with only 0.3% mass increase, offering a cost-effective solution for eccentric core-tube frame structures under wind and seismic excitations.

KEYWORDS SHBJ, eccentric core-tube frame structure, energy dissipation, failure mechanism

1 Introduction

The frame-core tube structural system represents the predominant solution for supertall buildings, where conventional designs typically position the core tube at the geometric center to ensure symmetrical lateral stiffness distribution. However, this conventional spatial configuration inherently creates compartmentalization that disrupts spatial continuity, failing to meet contemporary architectural demands for openness and interaction with the surrounding environment. Consequently, the

eccentric core-tube frame structure has emerged, where strategically offsetting the core tube to one side of the frame significantly enhances spatial continuity and panoramic visibility [1,2]. However, the eccentric core tube configuration introduces notable mass-stiffness eccentricity and torsional stiffness asymmetry, which significantly complicate seismic performance and deformation control [1]. Ensuring structural safety while maintaining architectural functionality constitutes the primary design challenge for this structural system.

In frame-core tube structural systems, a significant disparity exists between the lateral load-resisting capacities of the frame and core tube components. To

accurately capture the dynamic behavior of real structures during the design phase, researchers have conducted extensive investigations into the dynamic performance of these structural systems. Li et al. [3] systematically evaluated the seismic performance of reinforced concrete core tube structures, examining the influence of frame shear distribution ratio, frame bearing capacity, and column configuration patterns. Zhang and Far [4] studied the seismic response of frame-core tube structures with various structural heights, height-width ratios, foundation types and soil types. Bu et al. [1] investigated pulse-type ground motion effects on eccentric gravity column-core tube structures, demonstrating that increasing eccentricity in gravity column core tube structures reduces collapse resistance and raises failure probability. Vulnerability analysis plays a central role in evaluating structural performance and guiding design and maintenance decisions under hazards [5]. Building on this approach, Wang et al. [6] and Zhang et al. [7] investigated the seismic vulnerability and conducted damage assessment of steel-reinforced concrete frame-core tube structures. Existing studies have thoroughly characterized the mechanical performance of conventional symmetric frame-core tube structures, while research on eccentric core-tube systems remains limited. For geometrically irregular structures like eccentric core-tube systems, vertical differential deformations can induce additional lateral displacements through second-order effects [8], which presents additional challenges to the dynamic performance of the structure.

To mitigate dynamic responses of core-tube frame structures under seismic or wind loads, various vibration control strategies have been proposed. Xiong et al. [9] developed an innovative self-centering Y-eccentrically braced frame system and evaluated its seismic performance through low-cycle reversed loading tests, demonstrating self-recentering characteristics and energy dissipation capacity that effectively mitigate post-earthquake residual deformations. Shan et al. [10] performed a collapse risk-targeted analysis to evaluate the seismic performance of braced mega frame-core tube structures, quantifying nonlinear deformation behavior and energy dissipation mechanisms in critical lateral load-resisting components. Xie et al. [11] investigated the cable-bracing-self-balanced inerter system for frame-core tube structure vibration control, using topology optimization to enhance the inerter system efficiency. Li et al. [3] investigated the effectiveness of tuned mass dampers (TMDs) in mitigating wind-induced vibrations in high-rise frame-core tube structures through numerical simulations and experimental analyses, systematically examining vibration control performance across various TMD parameter configurations. These measures typically necessitate additional devices or components, thereby increasing the structural complexity and costs of the engineering structure.

Researchers have improved structural dynamic performance through innovative designs, particularly by implementing energy-dissipating connections and optimized joint configurations focused on symmetrical core-tube frame systems. Shahrooz et al. [12,13] conducted experimental investigations on stud-connected steel beam-to-wall joints, utilizing connector plate yielding to enhance joint strength, deformation capability, and hysteretic energy absorption performance. Chen et al. [14] proposed the frame-distributed rocking-core tube structure by incorporating a rocking mechanism to mitigate soft-story damage resulting from the reduced stiffness of frame-distributed-core tube structures. Li et al. [15] developed novel prefabricated brace connections for core tube-flange column joints in braced frame structures, conducting static tests on eccentric braced frames incorporating these innovative connections. Research indicates that some self-centering joint configurations demonstrate superior seismic resilience through reduced residual drifts and damage control during rare or extremely rare seismic events, but exhibit comparable seismic performance to traditional structures under moderate earthquakes [16].

Aiming at these problems, this study develops a slotted-hole bolted joint with top-and-seat angle (TSA-SHBJ) for beam-to-column connection for eccentric core-tube frame high-rise structures, where the inherent stiffness and mass eccentricities lead to significant differential vertical deformation between the core tube and the surrounding frame. Such deformation incompatibility results in additional joint rotation and undesirable secondary internal forces at the beam-column connections. The TSA-SHBJ allows a certain degree of rotational freedom to accommodate joint deformations resulting from structural eccentricity, with the top and seat angle components offering supplementary resistance to effectively control excessive structural deformation. The top and seat angle steels function as replaceable energy dissipation elements, utilizing substantial joint rotations during seismic events to improve the structure's overall energy dissipation performance. Cyclic loading tests are conducted to systematically examine the failure mechanisms and mechanical performance of TSA-SHBJ, with experimental comparison to SHBJ without TSA. An experimentally calibrated equivalent model of TSA-SHBJ is developed and integrated within the numerical model of an eccentric core-tube frame structure to investigate the dynamic performance improvement of this enhanced structural system under both wind and seismic excitations. Figure 1 presents the overall workflow of the experimental and numerical study on TSA-SHBJs, which summarizes the experimental program, the development of the equivalent joint model, and its integration into the numerical simulation of the eccentric core-tube frame structure.

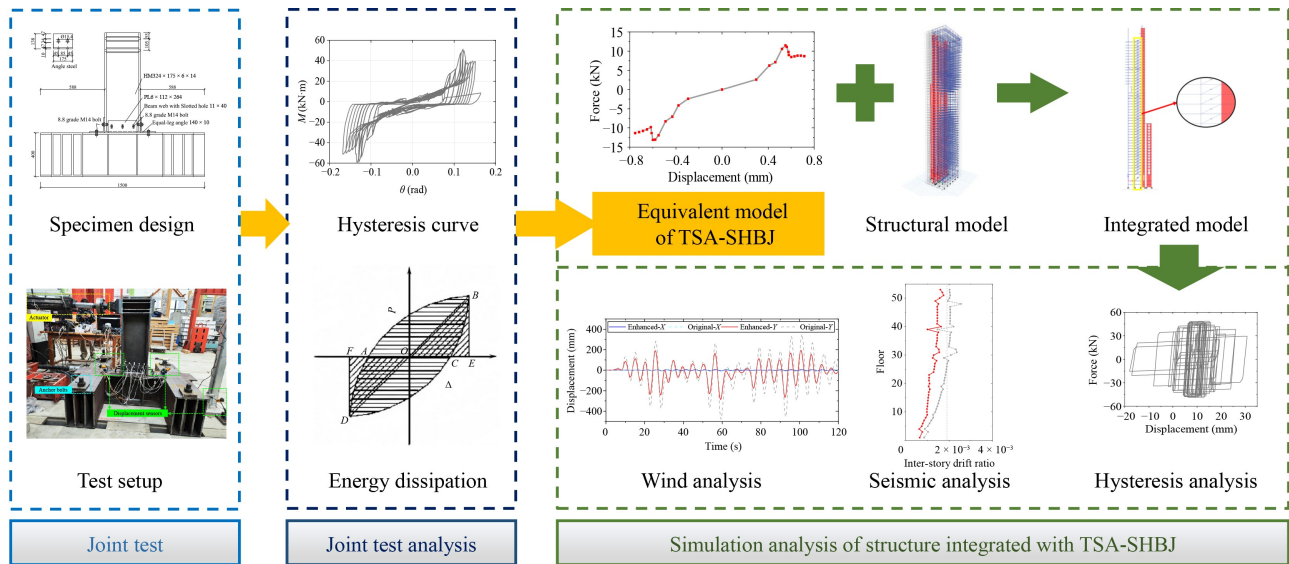


Fig. 1 Overall workflow of the experimental and numerical study on TSA-SHBJs.

2 Experimental setup

2.1 Experimental specimen

In this section, the designs of T-shaped beam–column specimens in compliance with standards [17,18] are presented. The experimental setup adopts horizontally anchored columns with vertically oriented beams, where horizontal actuators apply cyclic loading perpendicular to the beam to induce joint rotation. This arrangement ensures stable anchorage and precise displacement control without altering the fundamental joint force-transfer mechanism. Two specimens are constructed with identical parameters except for the presence/absence of TSA connections between the beam and column.

Figure 2 shows the schematic diagram of TSA-SHBJ and SHBJ. In the design of the specimen, the code requirements, the joint stiffness, and the energy dissipation performance are comprehensively considered to determine the dimensions of the TSA. According to the Standard for design of steel structures [17], the bolt spacing and edge distances are first defined, and the length of the two legs of the angle is then determined by the maximum allowable bolt edge distance to minimize the effect on the joint stiffness. The angle thickness is selected based on the results of pre-calculation, ensuring that the connection exhibits stable and desirable hysteretic behavior. The TSA-SHBJ connection employs M10, Grade 8.8 bolts, with 11 mm × 40 mm slotted holes in the beam web to accommodate bolt movement during joint rotation. The beam member is fabricated using welded H-section steel with dimensions HM324 × 175 × 6 × 14 and a length of $L = 900$ mm. The column member is constructed with welded H-section steel of dimensions HM400 × 250 × 8 × 14. The thickness of the angle steel is 10 mm, and a 15 mm × 34 mm elliptical hole is opened

on the side closely attached to the top and seat of the beam. It is connected by 8.8 grade M14 bolts, and a Φ15 circular hole is opened at the bottom of the angle steel, which is connected to the column component by 8.8 grade M14 bolts. The 10-mm TSA feature 15 mm × 34 mm slotted holes at the beam interface (M14, Grade 8.8 bolts) and Φ15 circular holes at the column interface (M14, Grade 8.8 bolts) for connection. Rectangular-section double flanges are welded near both ends of the column to serve as ground anchoring components, while double stiffening ribs are welded inside the rectangular section to enhance the overall stiffness of the specimen base. The joint specimen SHBJ for comparison has identical dimensions and connection bolt specifications to the TSA-SHBJ, except for the absence of the TSA steels.

2.2 Experimental setup and measurement

Figure 3 presents the experimental setup and loading device. The horizontal actuator is employed to apply beam-end loading. The specimen is anchored to the ground via anchor bolts passing through the double-extended flanges on the column, ensuring no slippage of the test specimen during loading. A servo-hydraulic actuator (maximum load capacity 200 kN, Hangzhou POPWIL Instrument Co., Ltd.) is employed to apply cyclic displacement loading. The actuator is pre-extended 200 mm to accommodate beam displacement while ensuring uniform force transfer. The applied load is measured using a load cell integrated in the actuator, while displacements and rotations are recorded by displacement transducers (Range 100 mm, TM Automation Instrument Co., Ltd.). In addition, strain gauges are attached to the specimen to monitor the local strain development.

Figure 3(a) illustrates the displacement measurement

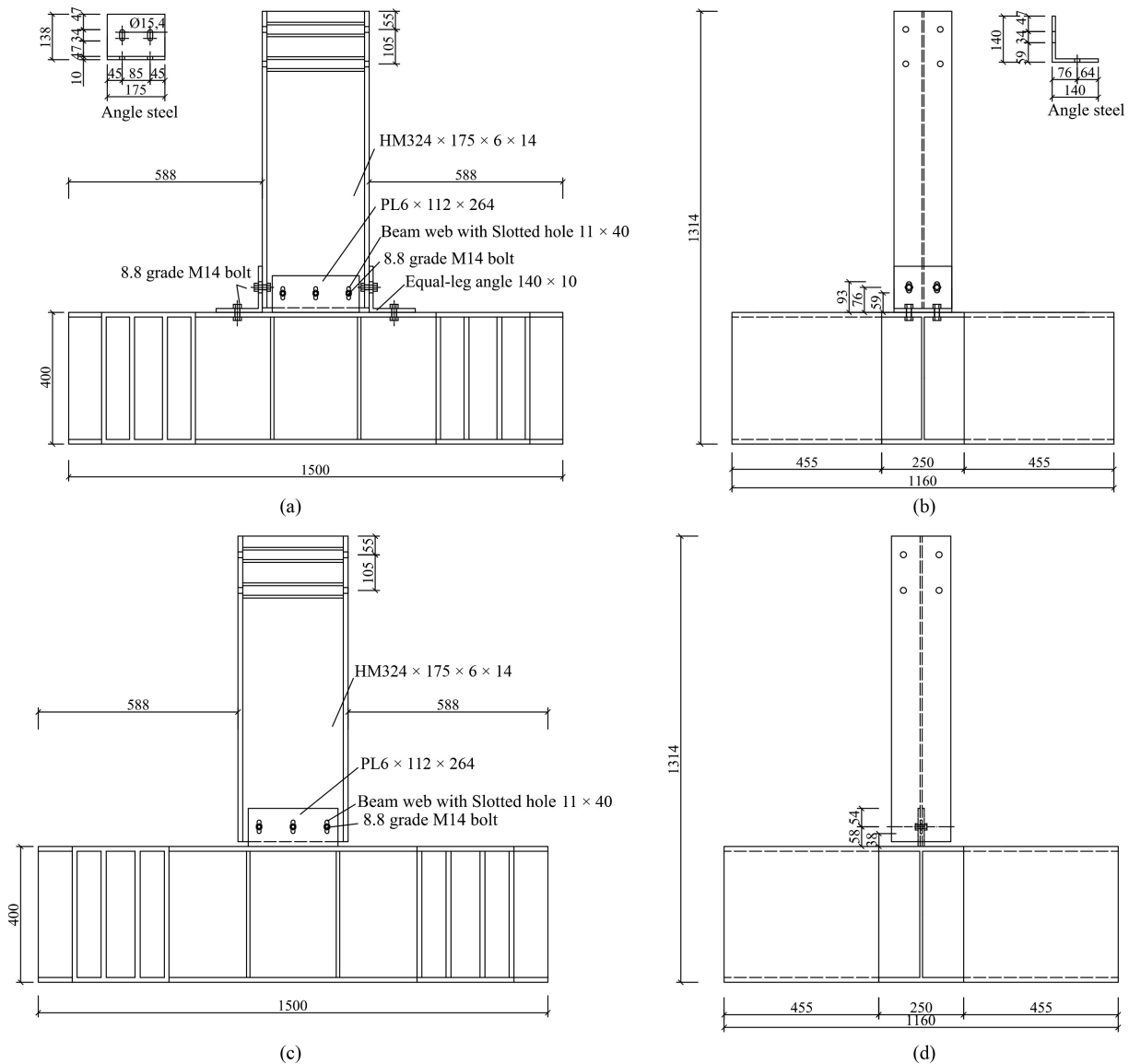


Fig. 2 Schematic diagram of TSA-SHBJ and SHBJ: (a) front view of TSA-SHBJ; (b) side view of TSA-SHBJ; (c) front view of SHBJ; (d) side view of SHBJ.

arrangement, which includes: 1) displacement measurement at the beam-end loading point; 2) a displacement sensor positioned at the column to ensure that no unintended horizontal slip occurs at the column support during loading; 3) joint rotation measurement using a flange-mounted transducer located 300 mm above the column. [Figure 3\(b\)](#) shows the strain measurement arrangement. Strain distribution near the joint is measured using uniaxial strain gauges on the beam web, beam flange, and top and seat angles. Note that the upper leg and lower leg refer to the angle legs connected to the beam and the column, respectively.

2.3 Material testing and loading protocol

All structural components are constructed using Q345 steel, with mechanical properties characterized through

standardized tensile testing compliant with codes [19,20]. The experimental protocol involved testing duplicate standard coupons for each of the three plate thicknesses employed in the specimens, with mean values establishing the representative material properties. Tensile tests are performed on duplicate specimens for each of the three plate thicknesses employed in the structural components, with mean values calculated for material property characterization. [Table 1](#) presents the experimentally determined material properties obtained from tensile testing.

This study examines the damage progression and energy dissipation mechanisms of structural joints under large rotation process through displacement-controlled quasi-static cyclic loading tests [21], with beam-end displacement serving as the control parameter. The loading protocol follows a progressive sequence through

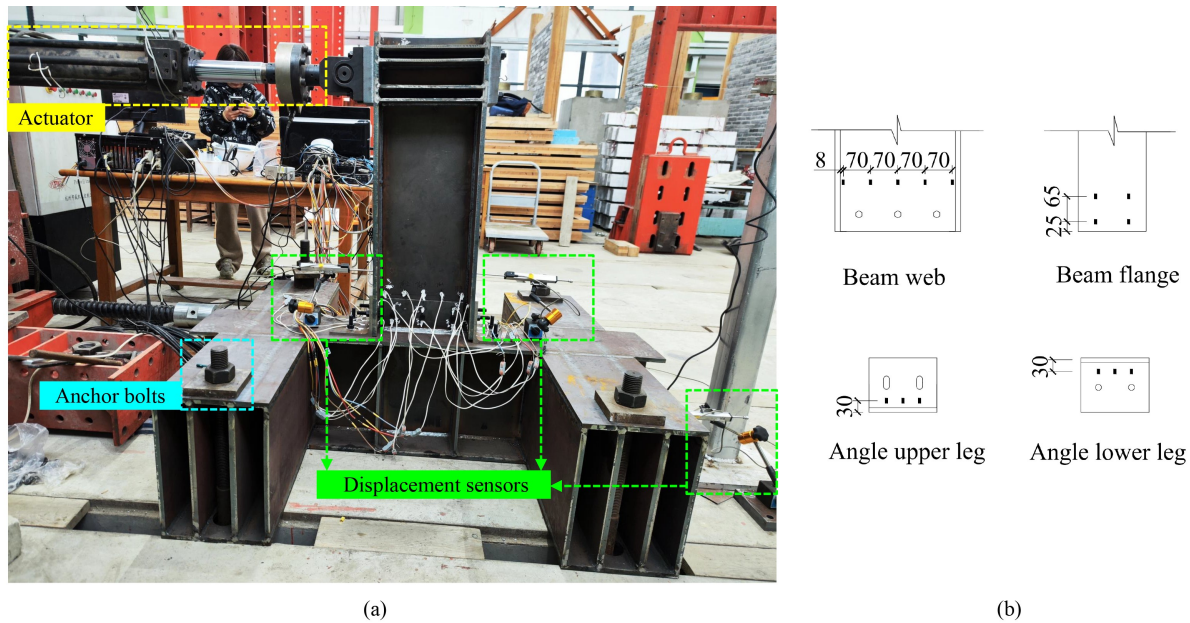


Fig. 3 Experimental setup: (a) experimental photo; (b) strain measurement.

Table 1 Steel material properties

Plate thickness (mm)	Yield strength (MPa)	Ultimate strength (MPa)
6	366.7	530.9
10	477.9	649.7
14	433.2	579.9

elastic, plastic, and ultimate failure states. Given the different deformation characteristics of the TSA-SHBJ and SHBJ, their loading schemes were refined based on preliminary deformation assessments to ensure that the key stiffness transitions and hysteretic features of each joint could be accurately captured. Figure 4 illustrates the specimen loading procedure, with the red dashed line indicating the final loading stage to failure.

The TSA-SHBJ specimen loading protocol consists of four phases: 1) 3 preconditioning cycles at 50 mm, which enable the joint to enter a stable working state and thereby improve the reliability of the results under subsequent large-amplitude loading; 2) primary loading

from 60 to 90 mm with 10 mm increments (2 cycles/level); 3) loading from 95 to 100 mm with 5 mm increments (2 cycles/level); 4) final loading from 105 mm until failure with 5 mm increments (1 cycle/level).

The SHBJ specimen loading protocol consists of 1) initial 20–80 mm amplitude range with 20 mm increments; 2) subsequent 90 mm-to-failure phase with 5 mm increments, applying 2 loading cycles at each amplitude level until specimen failure.

3 Experimental results

By analyzing the load-slip curves, hysteretic curves, and skeleton curves, the fundamental mechanical properties of the joints are evaluated in this section, including stiffness, load-bearing capacity, and ductility. The results provide references for the design and engineering application of this steel joint.

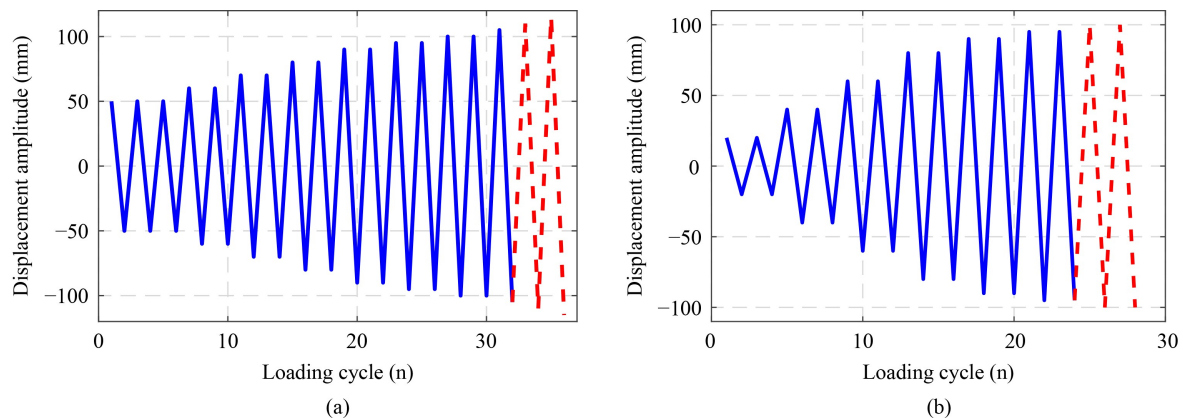


Fig. 4 Specimen loading procedure: (a) TSA-SHBJ; (b) SHBJ.

3.1 Deformation and failure mode

Figure 5 illustrates the progressive deformation development of the joint TSA-SHBJ during the testing process. When the beam-end displacement is less than 70 mm, the top and seat angle steels primarily undergo cyclic uplift and settlement through frictional slip between the angle steels and the beam flange induced by bolt pre-

tensioning. Simultaneously, local bending deformation occurs in the angle steels due to the inclination of the beam. When the displacement reaches 70–90 mm, the angle steel bolts slide to the top ends of the elliptical holes, causing the angle steels to rotate with the joint. The angle steels undergo yield deformation, retaining residual bending even after the joint returns to its initial position. When the beam-end displacement amplitude exceeds

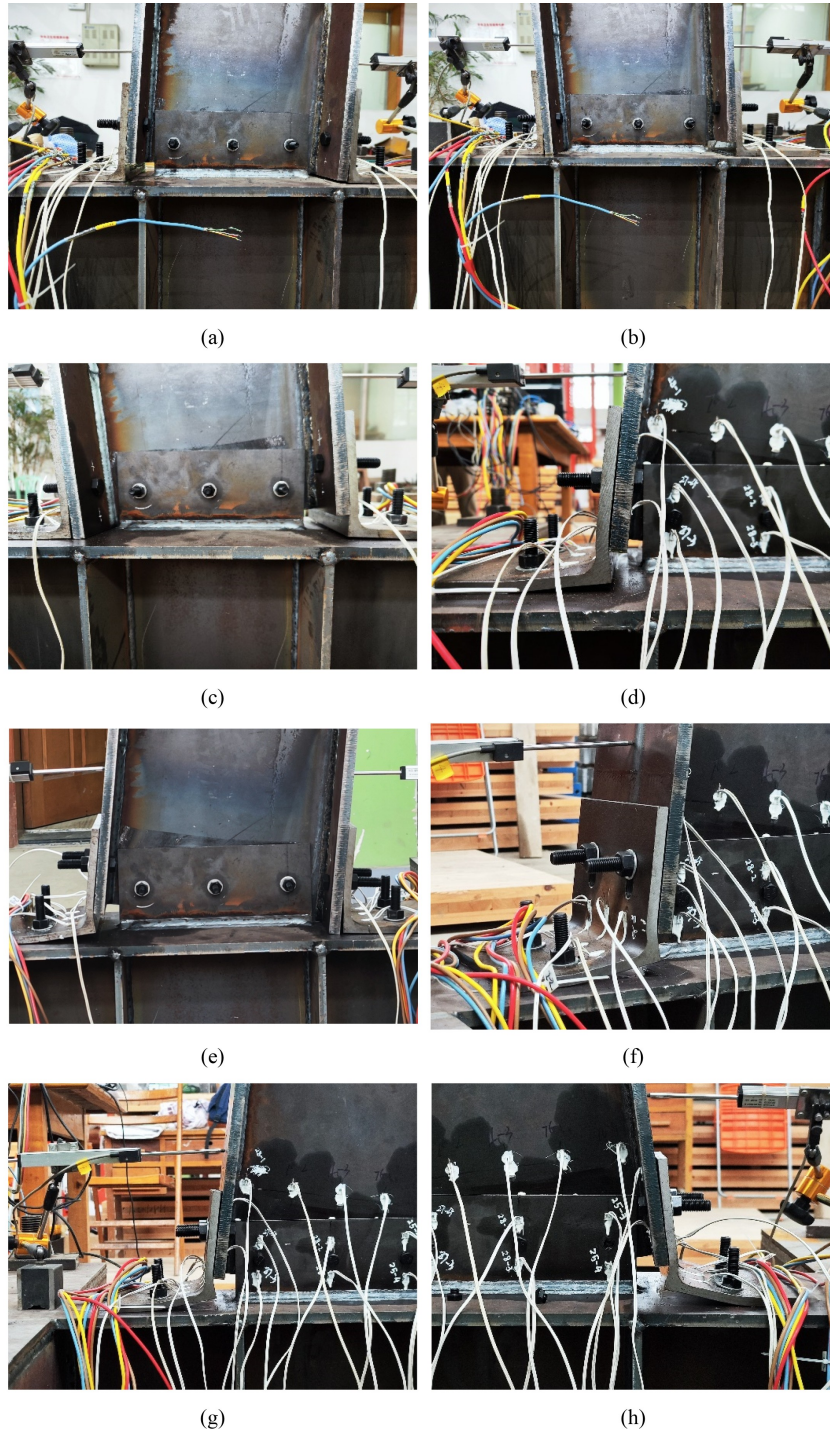


Fig. 5 TSA-SHBJ test rotation with varying excitation amplitude: (a) 50 mm; (b) 60 mm; (c) 70 mm; (d) 80 mm; (e) 90 mm; (f) 95 mm; (g) 100 mm; (h) 105 mm.

95 mm, the uplift magnitude of the angle steels continues to increase, accompanied by a significant vertical rise of the beam during its movement. A distinct plastic hinge forms in the beam flange at the mid-height of the angle steels.

Figure 6 shows the final failure mode of the TSA-SHBJ joint. When the beam-end displacement reaches 95 mm, the bolts at the edge of the beam web undergo double-shear failure (referring to the bolts being subjected to shear forces on two parallel planes simultaneously) and subsequently detach. The fracture surface exhibits typical double-shear failure characteristics, with a smooth fracture plane and distinct slip marks. Notably, as the angle steel bolts partially resist the joint bending moment, the plastic deformation at both ends of the elliptical holes in the beam web becomes more pronounced compared to the SHBJ joint (as shown in Fig. 7), while the failure progression of the web bolts is delayed. The failure of the angle steel bolts exhibits a composite morphology: smooth fracture edges with distinct slip marks (characteristic of shear failure), and rough fracture surfaces at the central region (indicating tensile rupture after cross-section weakening). Moreover, the deep embedding of bolt washers into the bolt holes confirms that the angle steel bolts sustained significant tensile forces during the failure process.

Figure 7 illustrates the deformation progression of the SHBJ joint during testing. As the beam-end load increases, the slotted holes in the beam web induce significant joint rotation. When the joint rotates back to its vertical state (initial position), a slight vertical uplift occurs, exhibiting behavior similar to that observed in the TSA-SHBJ joint. Figure 7(e) presents the experimental failure of the SHBJ joint. When the beam-end displacement reaches 95 mm, the web-edge bolts experience double-shear failure and detach, while significant bearing-induced plastic deformation occurs at both ends of the elliptical holes in the web. In contrast to the TSA-SHBJ joint, the SHBJ exhibits relatively less pronounced plastic deformation around the elliptical holes in the beam web

due to the absence of angle steel bolts resisting the joint bending moment.

3.2 Hysteresis curves

Figure 8 presents the hysteretic curves of the two specimens. For the SHBJ specimen, the joint moment remains relatively stable with narrow hysteresis loops when the rotation angle is less than 0.1 rad. However, when the rotation increases to 0.12 rad (corresponding to a beam-end displacement of 95 mm), the moment suddenly surges. At 0.12 rad, the moment sharply increases until rapid failure occurs. Although this specimen achieves an ultimate rotation capacity of 0.12 rad, its hysteretic performance is limited by the lack of a stable plastic development stage, resulting in thin hysteresis loops and poor overall energy dissipation capability.

For the TSA-SHBJ specimen, when the joint rotation angle does not exceed 0.09 rad, the joint primarily exhibits elastic deformation, with the hysteretic curve displaying pinching behavior and nearly overlapping loops in each cycle. Compared with the SHBJ specimen, the TSA-SHBJ joint exhibits a more pronounced increase in bending moment. On one hand, the compressive force induced by bolt pretension between the angle steel and the beam flange enhances the frictional resistance at the contact interface, thereby improving the joint's moment capacity. On the other hand, the rough contact surfaces between the angle steel and the beam flange, as well as between the joint plate and the beam web, work together with the bolt pretension to provide higher rotational stiffness and promote coordinated rotation between the angle steel and the beam.

When the joint rotation angle exceeds 0.09 rad, the mechanical behavior of the specimen undergoes significant changes. Inter-plate slippage occurs, leading to a progressive reduction in friction, which develops in a stepwise, discontinuous manner. This results in closely spaced fluctuations in the hysteretic curve within the 0.1

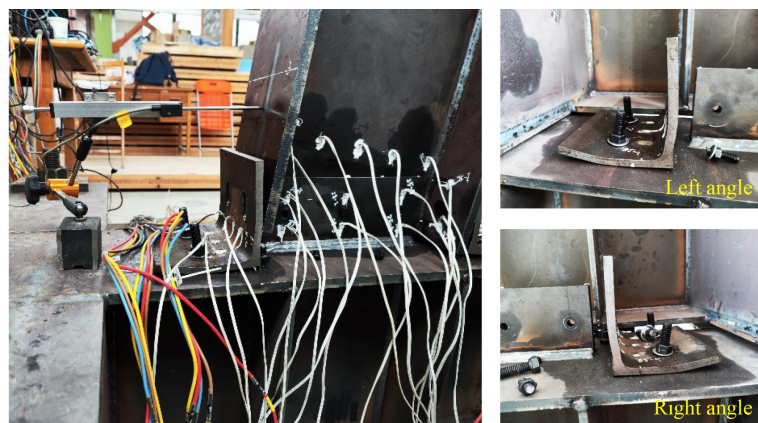


Fig. 6 TSA-SHBJ test failure.

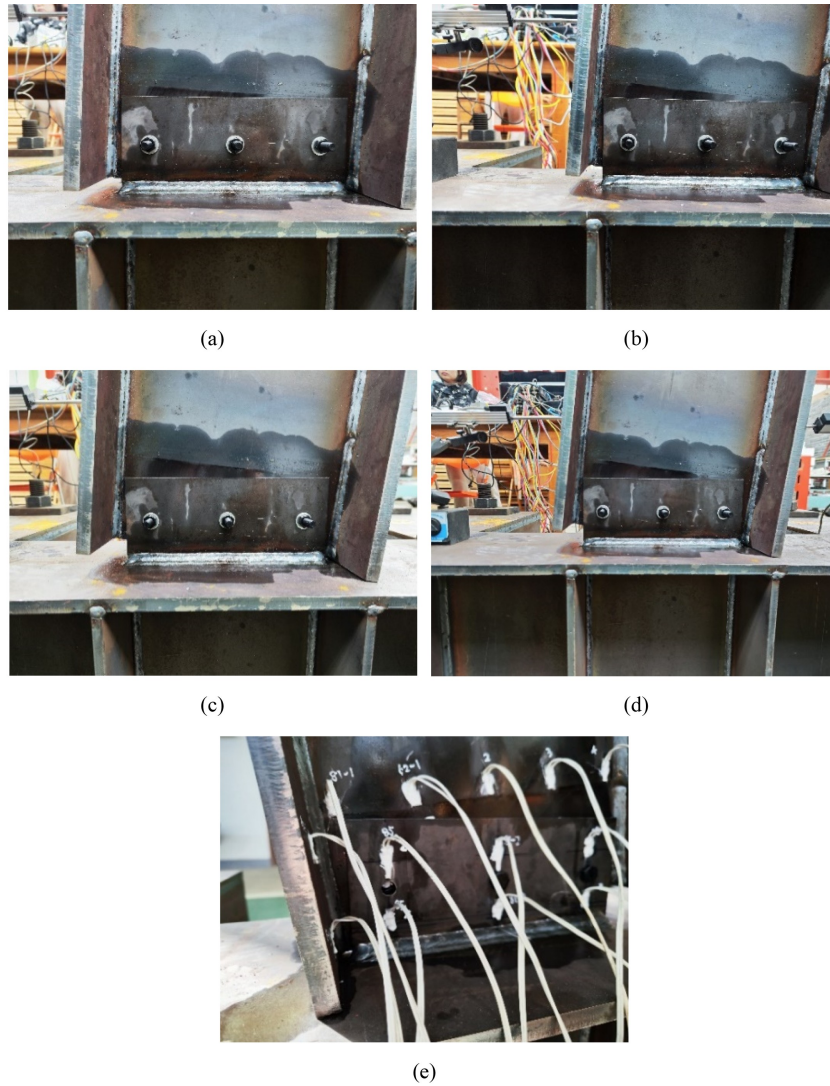


Fig. 7 SHBJ test rotation with varying excitation amplitude: (a) 40 mm; (b) 60 mm; (c) 80 mm; (d) 90 mm; (e) SHBJ test failure.

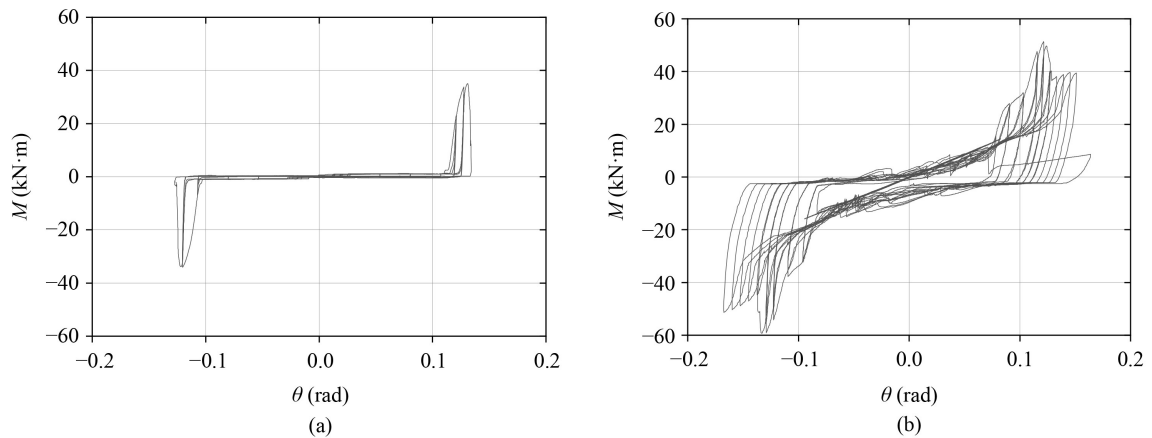


Fig. 8 Hysteresis curve of specimens: (a) SHBJ; (b) TSA-SHBJ.

rad rotation range. Within the 0.08–0.1 rad rotation range, the joint moment growth stabilizes. Ultimately, when the rotation reaches 0.12 rad, the joint moment plummets sharply, coinciding with observed failure and detachment

of the edge bolts in the beam web. Beyond this point, the joint's load transfer relies primarily on the angle steel bolts. This failure progression reveals the complete degradation mechanism of the joint, transitioning from

the friction-slip phase to the bolt failure phase.

When the joint rotation angle exceeds 0.1 rad, the hysteretic curve exhibits a significantly enlarged energy dissipation area, indicating more robust and efficient energy absorption. This phenomenon originates from the plastic deformation mechanism of the top and seat angle steels. One side of the angle undergoes bending deformation, while the opposite side experiences significant uplift, collectively enabling effective energy absorption and dissipation through full plastic deformation. During this phase, the slope of the loading curve gradually decreases, reflecting the progressive stiffness degradation of the joint under cyclic loading. Furthermore, the hysteretic curves exhibit persistent pinching behavior, which can be attributed to two mechanisms. First, as the rotation angle increases, the bolt pretension gradually decreases, resulting in reduced frictional resistance at the contact interfaces. Second, repeated cyclic sliding smooths the mating surfaces, further weakening the frictional contribution during subsequent loading cycles.

Overall, test results demonstrate that both TSA-SHBJ and SHBJ specimens exhibit good rotational performance under small rotation angles (< 0.1 rad), where the joint bending moment remains relatively low, enabling adaptive rotation in a low-stress state. When the rotation angle exceeds 0.1 rad, the TSA-SHBJ specimen demonstrates superior seismic performance: its hysteretic curves become fuller, energy dissipation capacity improves significantly, and ductility performance excels, fully showcasing the advantages of this joint configuration in seismic energy dissipation.

3.3 Skeleton curves

Figure 9 presents the skeleton curves of the specimens. Test results demonstrate that the TSA-SHBJ specimen exhibits superior mechanical performance under positive loading: its ultimate moment capacity and ultimate displacement at the beam end are significantly higher than those of the SHBJ specimen. This is because the inclusion of angle steels provides an additional load path

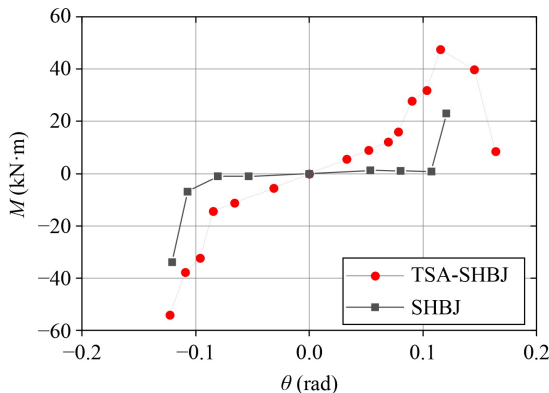


Fig. 9 SHBJ test failure.

that reduces stress concentration in the bolts, and the triangular force-resisting mechanism thereby enhances the joint's rotational stiffness. After the failure of the web bolts, the angle steel bolts continue to maintain joint connectivity, thereby delaying overall collapse. In contrast, the SHBJ specimen experiences complete joint failure immediately after web bolt fracture, exhibiting brittle collapse characteristics that are detrimental to seismic performance.

3.4 Stiffness degradation

Stiffness degradation is a critical indicator for evaluating seismic performance, characterizing the progressive reduction in structural stiffness with increasing load levels and reflecting the degree of component damage. The secant stiffness is used to characterize the rotational stiffness of joints after entering the plastic state, defined as follows:

$$K_i = \frac{|+M_i| + |-M_i|}{|+\theta_i| + |-\theta_i|}, \quad (1)$$

where K_i and M_i are rotational stiffness and peak moment at the i th load level, θ_i is the joint rotation angle at peak moment, with sign convention: '+' for push and '-' for pull.

Figure 10 presents the comparison between joint rotational stiffness degradation under varying load levels, with the initial load level corresponding to a beam end displacement of 50 mm. The experimental results demonstrate that the SHBJ specimen exhibits no progressive stiffness degradation. The connection maintains low rotational stiffness during bolt slippage within the elongated hole followed by abrupt failure. The TSA-SHBJ specimen exhibits a unique stiffness evolution after the bolts reach the end of the slotted holes. Under identical load levels, the rotational stiffness of TSA-SHBJ consistently surpasses that of SHBJ, with its peak rotational stiffness achieving a 99.5% improvement over SHBJ. This demonstrates the critical role of top/sear

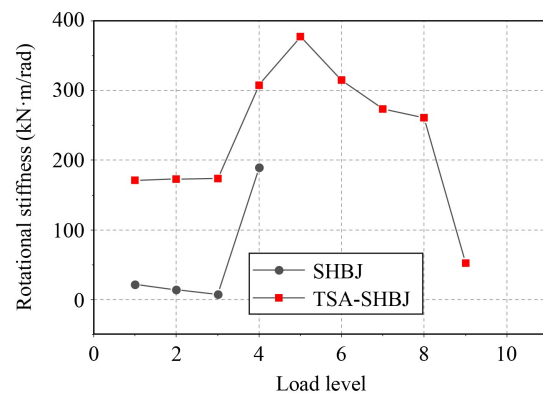


Fig. 10 Stiffness degradation curve.

angle steels in enhancing the overall joint stiffness.

3.5 Cumulative energy dissipation

The analysis employs rotational ductility to evaluate structural members' deformation capacity under seismic excitations, calculated as:

$$\mu_\theta = \theta_u / \theta_y, \quad (2)$$

where θ_u is the maximum joint rotation angle, θ_y is the yield rotation angle, taken as 0.1 rad in this study, corresponding to the bolt reaching the end of the slotted hole.

The energy dissipation capacity of the joint is characterized by equivalent viscous damping ratio ξ_e , energy dissipation coefficient E , and total energy dissipation W_{total} , where ξ_e and E can be obtained from Fig. 11.

$$\xi_e = \frac{1}{2\pi} \frac{S_{(ABC+CDA)}}{S_{(OBE+ODF)}}, \quad (3)$$

$$E = \frac{S_{(ABC+CDA)}}{S_{(OBE+ODF)}}, \quad (4)$$

where $S_{(ABC+CDA)}$ is the area of hysteresis loop $ABCD$, $S_{(OBE)}$ is area of triangle OBE , and $S_{(ODF)}$ is the area of triangle ODF .

Table 2 lists the joint ductility coefficient and energy consumption indicator. The SHBJ specimen exhibits a rotation ductility coefficient of 1.21, demonstrating minimal plastic deformation capacity. Upon reaching its critical state, i.e., when bolts contact the ends of the slotted holes, it undergoes rapid brittle failure, resulting in

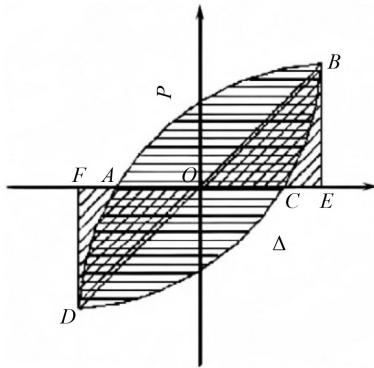


Fig. 11 Calculation of equivalent viscous damping coefficient ξ_e .

Table 2 Joint ductility coefficient and energy consumption indicator

Specimen	θ_y (rad)	θ_u (rad)	μ_θ	W_{total} (kJ)	ξ_e	E
SHBJ	0.10	0.12	1.21	6.70	0.04	0.24
TSA-SHBJ	0.10	0.16	1.64	78.90	0.18	1.12

low overall energy dissipation and negligible equivalent viscous damping and energy dissipation coefficients. With the addition of top/seal angle steels, the joint's performance undergoes significant improvements: joint ductility increased by 35.6%, the overall energy consumption of the joint increased by 9.7 times, and the equivalent viscous damping coefficient and energy dissipation coefficient both increased by 4.1 times. These results demonstrate the substantial enhancement of seismic performance achieved by incorporating top/seal angle steels.

3.6 Strain analysis

This section takes the TSA-SHBJ specimen as an example to investigate the joint strain distribution, with representative strain for the beam and angle steels illustrated in Fig. 12. Figures 12(a) and 12(b) illustrate the strain of beam web near flange regions and flanges. During the initial to mid loading stages, the strain distributions in both the beam web and flanges are primarily governed by frictional effects. As the joint rotation amplitude increases, the frictional forces drive the uplift of the angle steels, leading to a progressive rise in beam flange strain. When the joint rotation reaches the stage where the angle steel bolts contact the ends of the slotted holes, the beam flange strain progressively increases. During advanced loading stages, web bolts experience yielding with sections undergoing shear degradation, progressively reducing shear transfer capacity. Consequently, the strain in the beam web undergoes a decline until specimen fails. In the post-yielding phase, beam flange strain stabilizes as the load transfer mechanism between angle steel and beam is predominantly governed by tensile failure of the connection bolts rather than shear failure. This behavior indicates that the angle bolts maintain consistent load-bearing capacity with negligible strength degradation throughout the deformation process.

Figures 12(c) and 12(d) show that the strain trends in both side angle steels exhibit consistent variation with loading time. However, it is noteworthy that the strain distribution across the angle steels displays marked asymmetry: the strain values on the side connected to the beam flange are significantly larger than those on the bottom side. Experimental observations reveal that this asymmetry primarily stems from load transfer mechanisms. At tension state, the angle steel is lifted entirely, with the upper leg pulling the lower leg, concentrating strain on the beam-connected side. While the bottom leg of the angle steel experiences out-of-plane deformation, which partially offsets the tensile strain, resulting in a significant strain disparity between the two legs.

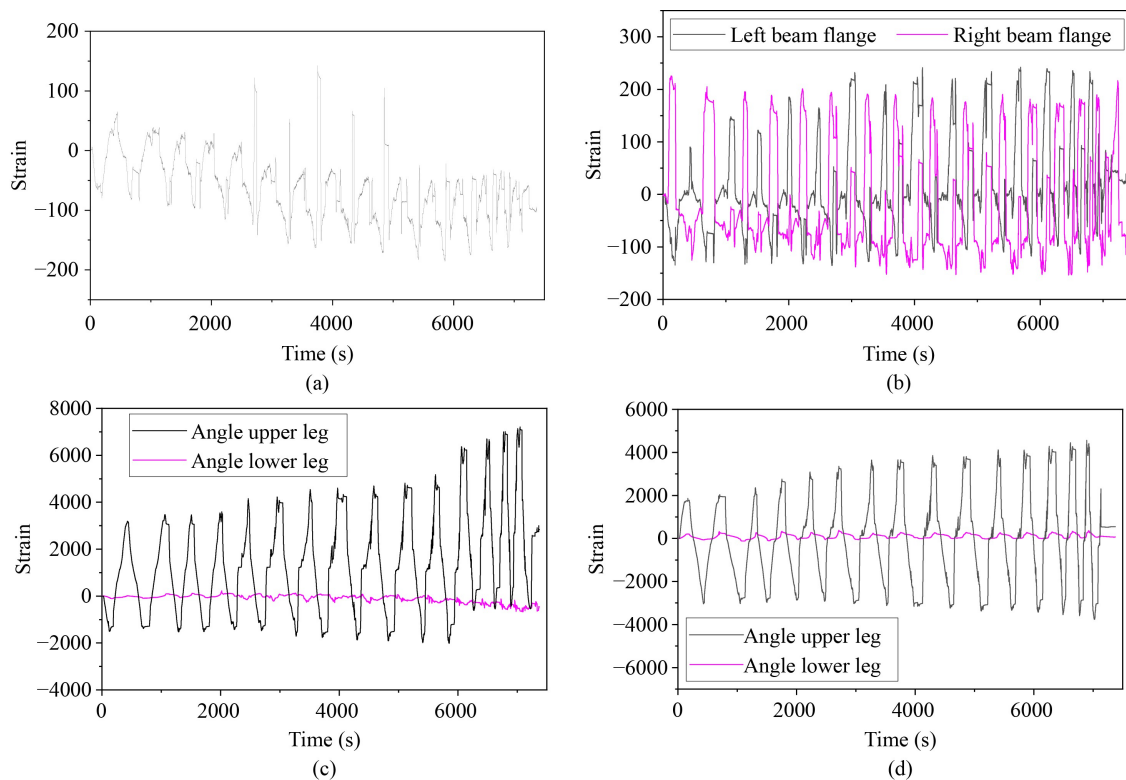


Fig. 12 TSA-SHBJ joint strain: (a) beam web; (b) beam flange; (c) left angle steel; (d) right angle steel.

4 Analysis of frame-core tube structure with slotted-hole bolted joint with top-and-seat angles

4.1 Model of frame-core tube structure

This study conducts a numerical analysis of a frame-core tube structure integrating the proposed TSA-SHBJ. The prototype structure is a high-rise building with eccentric core tube located in New Jiangwan City, Shanghai, China. The structure rises 249.6 m with a 2-level basement and above 47 stories, and extends to 279.0 m with the rooftop crown. Reinforcement zones are provided from the 1st to 5th floors, while strengthening levels are incorporated at the 30th and 40th floors. The building features an asymmetric structural layout: the core tube is offset to the north side, while the steel frame is positioned on the opposite side. These two structural systems remain largely independent in space with weak interconnections, resulting in a pronounced structural eccentricity issue. The core tube adopts a tapered design along the height, and the structure can be divided into three standard layers based on variations in its cross-section.

Figure 13 presents the simulation model of high-rise building with eccentric core tube developed in ETABS, with the base of the model fixed to simulate the foundation constraint. The main structural components, including beams, columns, and coupling beams, are

modeled using frame elements. The frame columns are concrete-filled steel tube members, with section properties varying by floor, while the frame beams are mainly I-shaped sections. The core tube walls are modeled using shell elements, while floor slabs are represented with membrane elements, which transfer loads to the surrounding beams, columns, and joints. Red regions in Fig. 13 represent the core tube while the blue depicts the steel frame. It should be noted that between the 31st and 35th floors, the frame portion of the structure undergoes multiple layout variations in plan (as marked by the red box in Fig. 13(a)). The cross-section of the core tube transitions gradually from a quasi-rectangular shape to a triangular configuration along the building height, accompanied by a progressive reduction in sectional area. The steel frame connects to the core tube through steel frame beams. These features lead to the structural asymmetry and eccentric characteristics.

In structural dynamic analysis, long-term material deformation and sustained load effects are not considered. The compressive strength of C40, C50, and C60 concrete cylinders specified by the design institute is 39.02, 45.89, and 57.41 MPa, respectively. Since this analysis focuses on the energy dissipation performance of the proposed joints, material nonlinearities of the structure, such as shrinkage and creep, are neglected. Figure 14 illustrates the first three mode shapes of the structure, predominantly characterized by *Y*-direction translation, *X*-direction translation, and torsional vibration. The

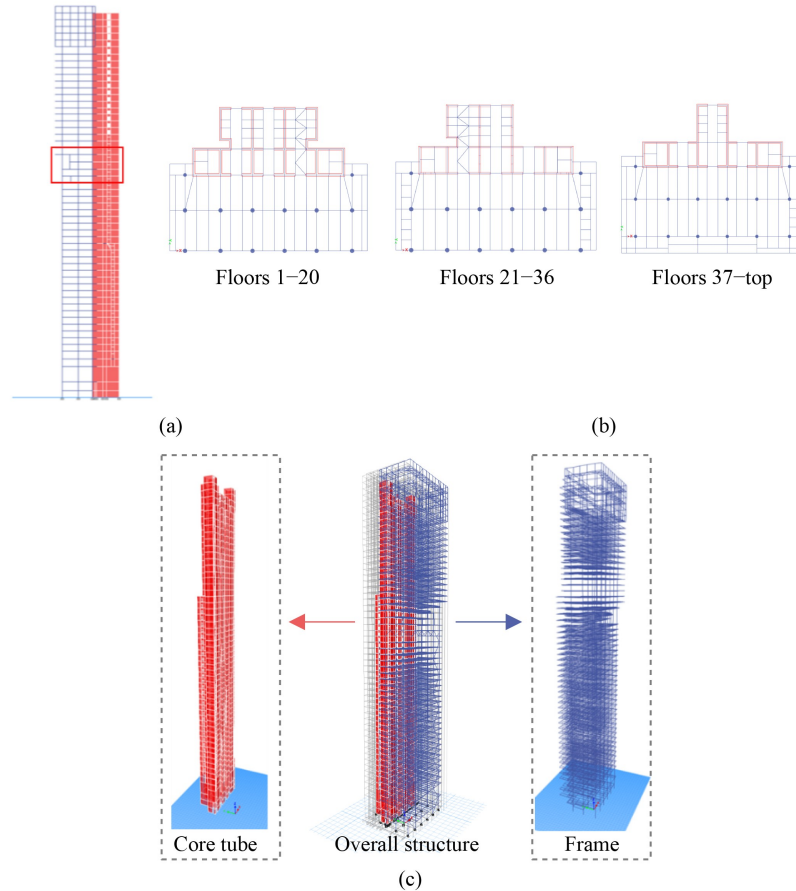


Fig. 13 Simulation model of high-rise building with eccentric core tube: (a) structural elevation diagram; (b) typical floor layout; (c) three-dimensional model.

calculated coordinates of stiffness centers and mass centers reveal that the average Y -direction eccentricity ratio reaches 47.15% across all 47 above-ground floors, indicating significant structural eccentricity in the Y -axis. This finding corroborates the pronounced torsional response observed in the second mode shape. Furthermore, due to the tapered core tube configuration and architectural facade requirements, the structural layout adjustments above the 21st floor introduce additional X -direction eccentricity (averaging 2.13% above this level).

4.2 Equivalent model of slotted-hole bolted joint with top-and-seat angles

This study employs the energy equivalence principle to develop an analytical TSA-SHBJ equivalent model for structural system integration. The damping behavior of the TSA-SHBJ is simulated using spring-dash elements in ETABS, which are specifically designed for modeling energy dissipation devices like viscous dampers. These elements permit customized hysteretic properties and cross-sectional definitions to accurately represent nonlinear cyclic behavior. For the TSA-SHBJ joints studied in this paper, the $M-\theta$ curves demonstrate

negligible degradation in yield strength during loading cycles. Therefore, an isotropic hysteretic model is adopted for equivalent calculation.

Seismic standards have established computational procedures based on the mode decomposition response spectrum method for structural analysis considering damping components. However, this method requires multiple iterations and involves computationally intensive processes. This study employs the energy equivalence method based on equivalent linearization principles, as illustrated in Fig. 15. The fundamental principle involves equating the energy dissipated by the energy-absorbing joint during one complete loading cycle with that consumed by the equivalent spring-dashpot unit over a single period ($S_1 = S_2$ as illustrated in Fig. 15(a)). For TSA-SHBJ, the rotational energy dissipation behavior of the TSA joints is equivalently represented by the axial energy dissipation of spring-dashpot units in the frame system.

Using the maximum rotation angle before joint failure in the test as the reference, the diagonal length variation $\Delta d = d - d_1$ in the frame with installed spring-dashpot units is calculated. This Δd value serves as the displacement for constructing the equivalent hysteretic

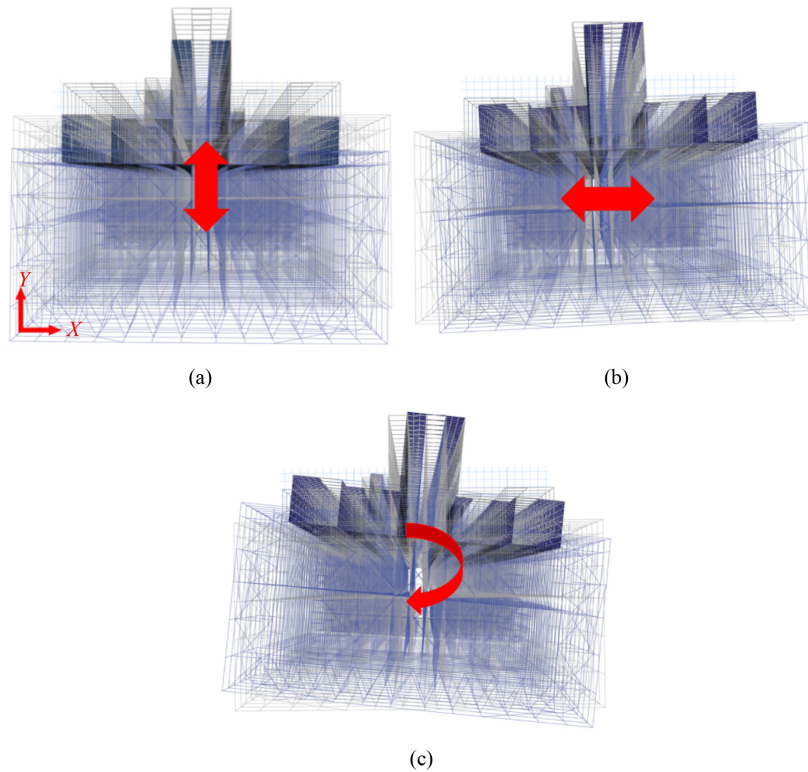


Fig. 14 The first three-order vibration modes: (a) the first mode, 5.493 s; (b) the second mode, 5.172 s; (c) the third mode, 3.682 s.

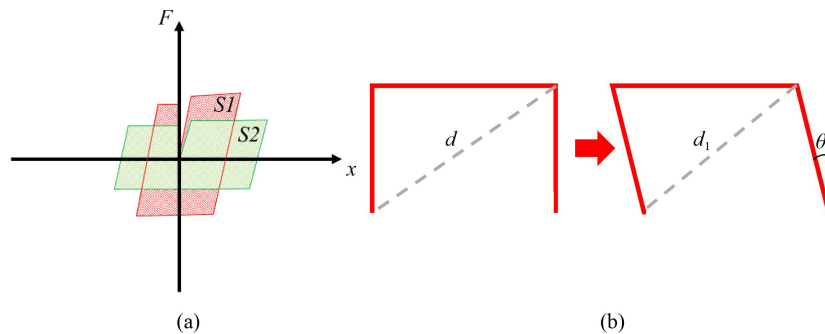


Fig. 15 Energy equivalence method: (a) hysteresis loop area equivalence; (b) geometric equivalence.

loop of the spring-dashpot unit, ensuring equal energy dissipation area. Figure 16 presents the simulated equivalent skeleton curve.

The TSA-SHBJ joints are suitable for connections

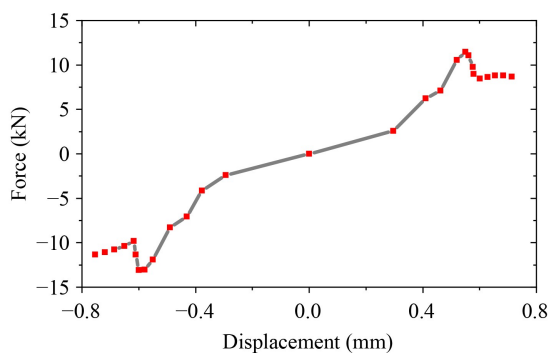


Fig. 16 Equivalent skeleton curve.

between the frame and core tube systems. Accordingly, during the equivalent modeling process, the spring-dashpot units are installed at the frame-to-core-tube connection joints and arranged story-by-story along the building height. Figure 17 illustrates the layout of the simulated joints within the structural system.

4.3 Wind induced vibration analysis

Wind loads in the X and Y directions are considered separately to investigate the structural wind-induced vibration responses. The wind load values specified by the code are applied to the structural model with the assumption of rigid in-plane behavior for each floor level. The design parameters for wind loads are as follows: the basic wind pressure for 50-y and 100-y return period are 0.55 and 0.60 kN/m^2 , respectively. The ground roughness

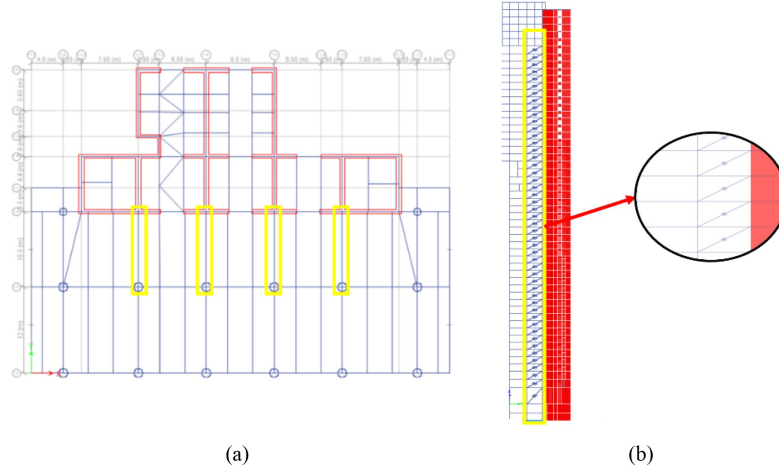


Fig. 17 TSA-SHBJ layout position: (a) planar layout; (b) scope of facade layout.

is Class C, and the structural damping ratio used for wind load calculation is 0.035.

Wind-induced vibration characteristics are evaluated through time-history analysis using Davenport spectrum-generated wind speeds, converted to pressures through the following relationship:

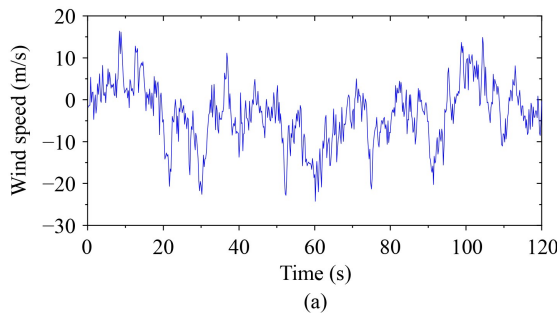
$$w_0 = \frac{1}{2} \rho v_0^2, \quad (5)$$

where v_0 represents the basic wind speed, $\rho = 0.00125e^{-0.0001z}$ is the air density, approximately estimated based on the site elevation z .

The Davenport spectrum, a widely utilized wind turbulence model in engineering practice, developed from more than 90 recorded strong wind events across diverse geographical locations. In the Davenport spectrum, the turbulence scale remains constant with height, and its power spectral density function can be expressed by

$$\begin{cases} S_v(n) = \bar{v}_{10}^2 \frac{4kx^2}{n(1+x^2)^{4/3}}, \\ x = \frac{1200n}{\bar{v}_{10}}, \end{cases} \quad (6)$$

where k is the ground roughness coefficient, $n = \omega/2\pi$, $\bar{v}_{10}$ denotes the mean wind speed at 10 m height above ground.



Using the 50-y return period basic wind pressure as the benchmark, the wind speed time-history and power spectrum are shown in Fig. 18.

The structure is vertically divided into 10 zones to address height-varying wind pressure, as shown in Fig. 19. Zone wind loads are calculated as the product of wind pressure and zone area, applied at each zone's mass centroid.

Figure 20 presents the envelope curves of structural inter-story drift ratios under Davenport spectrum-generated wind loads. Under time-history wind loads, the enhanced structure exhibits smaller inter-story drift ratios in both X and Y directions compared to the original configuration. Specifically, the average maximum inter-story drift ratio in the X direction is decreased by 4.77%, and the average decrease in the Y direction achieves 15.81%. This demonstrates that the TSA-SHBJs provide effective vibration reduction under wind excitation, with more pronounced deformation mitigation in the Y -direction where the structural stiffness is comparatively weaker.

Figure 21 illustrates the top displacement responses of original and enhanced structures under X - or Y -direction wind loads. The enhanced structural configuration incorporating TSA-SHBJs achieves effective vibration reduction, with peak displacements decreasing by 31.5% (from 229.93 to 157.49 mm) in the X -dominant direction

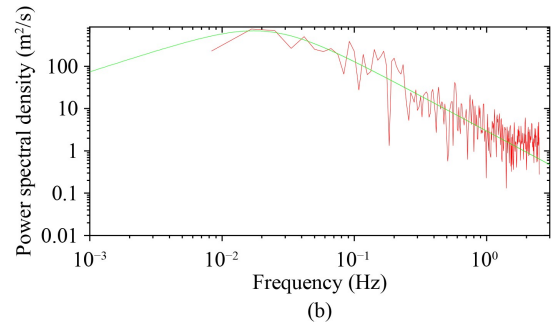


Fig. 18 Wind simulation based on Davenport spectrum: (a) wind speed; (b) power spectral density.

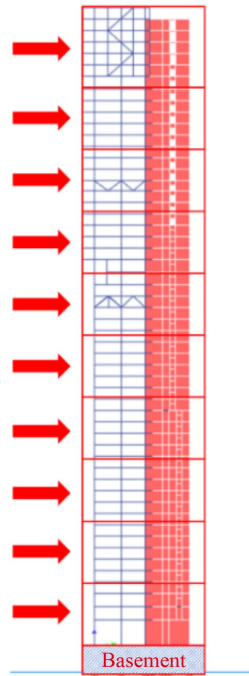


Fig. 19 Schematic diagram of wind load zoning.

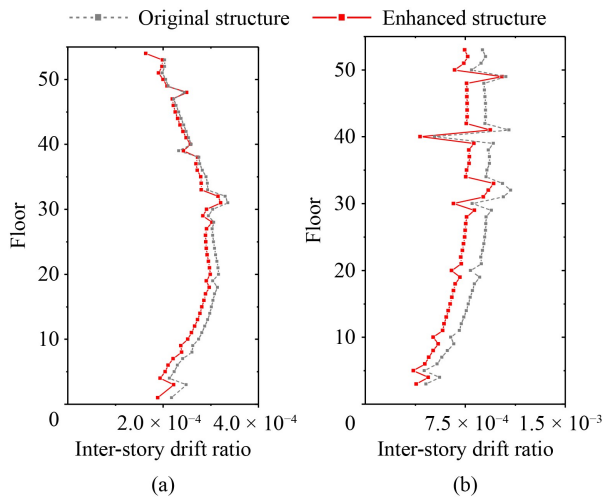


Fig. 20 Maximum inter story drift ratio of the structure under wind load: (a) *X* direction; (b) *Y* direction.

and 40.1% (from 477.96 to 286.37 mm) in the *Y*-dominant direction, while root-mean-square (RMS) displacements are reduced by 28.8% and 41.3% for *X*- and *Y*-dominant excitations, respectively. Furthermore, the larger *Y*-direction top displacement under *X*-wind excitation (versus *X*-displacement under *Y*-wind) demonstrates that the eccentric core tube induces more pronounced coupled lateral-torsional behavior during *X*-direction loading. The enhanced structural system effectively mitigates the coupled lateral-torsional effect, demonstrating 52.2% reduction in *Y*-direction peak displacement and 63.4% decrease in RMS values under *X*-direction wind excitation.

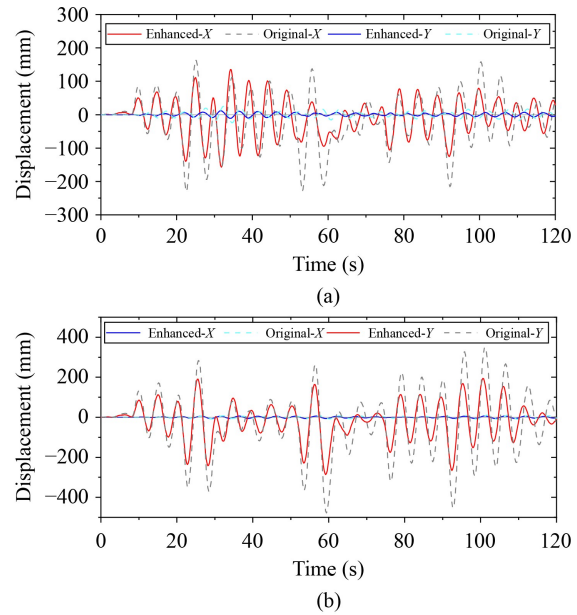


Fig. 21 Time history of displacement response at the top of the structure: (a) *X*-direction wind excitation; (b) *Y*-direction wind excitation.

4.4 Seismic vibration analysis

4.4.1 Seismic excitation

This section employs time-history analysis to investigate the seismic response of the eccentric core-tube structure integrating the proposed TSA-SHBJ and evaluate the energy dissipation performance of the TSA-SHBJ. According to the seismic design standards for site classification [22], Shanghai is categorized as a Type IV site, with characteristic periods of 0.9 s for frequent earthquakes and design basis earthquakes (horizontal direction), while 1.1 s for rare earthquakes. For the two characteristic period cases, two sets of ground motion time-histories are selected based on target response spectra and site conditions, comprising 2 natural records and 1 artificial wave for each set. Two natural records for $T_g = 0.9$ s: 1999 Kocaeli, Turkey (USAK) and 1999 Hector Mine, USA (San Bernardino -Fire Sta. #9) are denoted as NR0.9-1 and NR0.9-2. Two natural records for $T_g = 1.1$ s: 1979 Imperial Valley, USA (El Centro Array #12) and 1999 Chichi, Taiwan, China (CHY058) are denoted as NR1.1-1 and NR1.1-2.

The time-history curves of the seismic waves are shown in Fig. 22, while the corresponding response spectrum curves for design-basis earthquakes are presented in Fig. 23. As evident from Fig. 23, the seismic wave response spectra demonstrate consistent compatibility with the code-specified spectra. The peak ground accelerations (PGAs) of the seismic waves are adjusted to 0.35, 1.00, and 2.00 m/s^2 for frequent, design basis, and rare earthquakes. The structure is subjected to

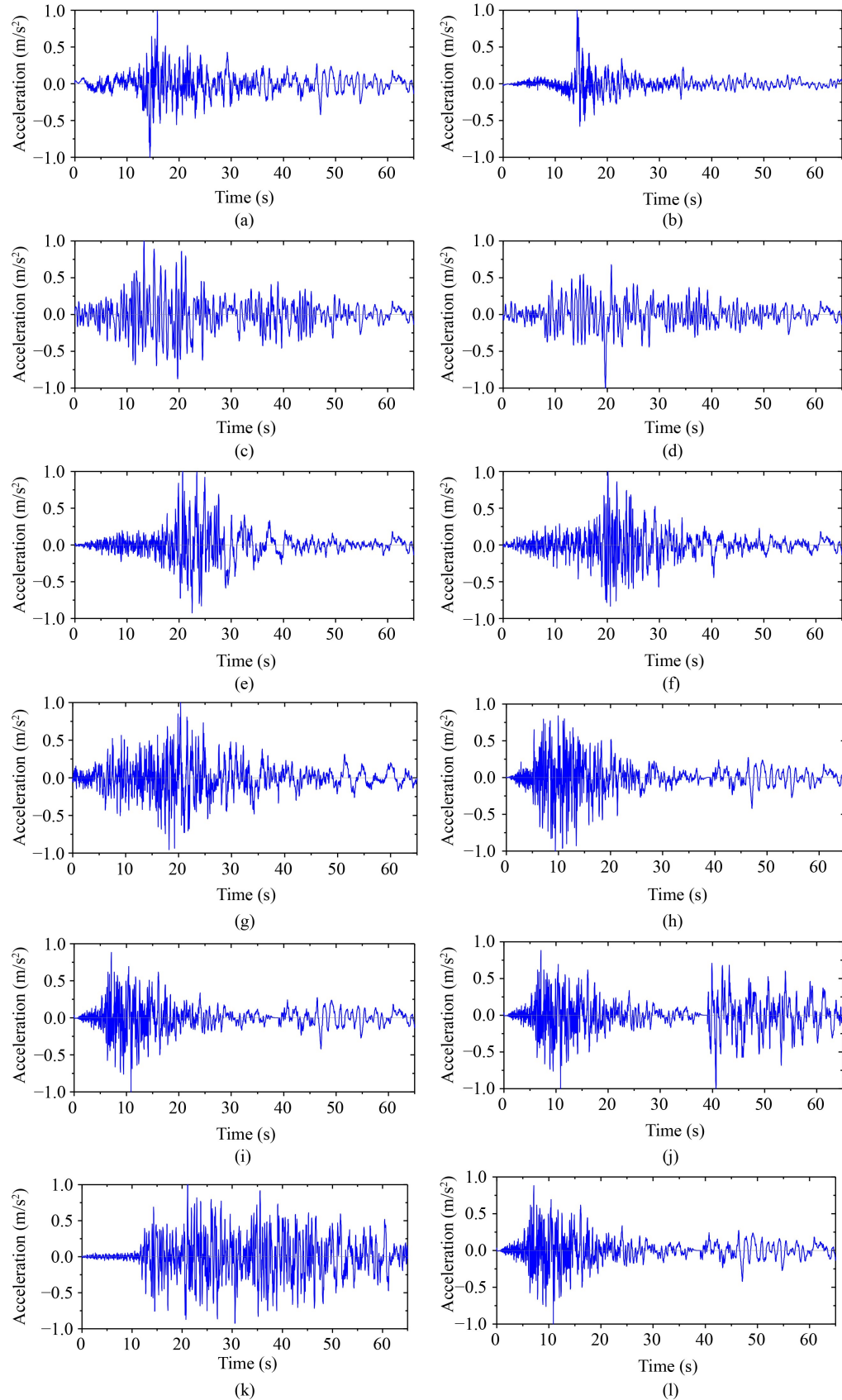


Fig. 22 Seismic wave time history: (a) AWX0.9; (b) AWY0.9; (c) NRX0.9-1; (d) NRY0.9-1; (e) NRX0.9-2; (f) NRY0.9-2; (g) AWX1.1; (h) AWY1.1; (i) NRX1.1-1; (j) NRY1.1-1; (k) NRX1.1-2; (l) NRY1.1-2.

bidirectional seismic excitation along its principal axes, with peak accelerations scaled to a 1:0.85 (primary to secondary directions) in accordance with standard specifications. Due to structural asymmetry, each seismic wave is considered under two loading scenarios with different principal horizontal directions. Table 3 lists the seismic conditions.

4.4.2 Seismic response

Figures 24–26 illustrate the inter-story drift ratios between the original and enhanced structural systems under 0.35, 1.00, and 2.00 m/s² dominant-direction excitations, respectively. Under frequent earthquakes, the X-direction vibration reduction is particularly obvious. Specifically, when subjected to the NRX0.9-1 seismic wave, the system achieves 59.74% reduction in X-direction inter-story drift ratio at 39F. However, under Y-direction-dominant seismic excitation, the Y-direction inter-story drift ratio reduction remains insignificant. This phenomenon stems from the eccentric core-tube structure’s asymmetric deformation under static conditions: uneven force distribution among vertical members creates an initial Y-direction lateral drift equilibrium state. Under low-amplitude seismic excitation, the structure demon-

strates minimal Y-direction dynamic response, limiting joint energy dissipation capacity due to inadequate displacement demand.

When the PGA increases to 1.00 m/s², as shown in Fig. 25, the Y-direction inter-story drift ratio reduction demonstrates significant enhancement. Particularly, the NRY0.9-2 wave yields an average 30.04% reduction rate across all stories. Under X-direction rare earthquakes, the controlled structure demonstrates effective X-direction inter-story drift ratio reduction under three seismic waves, with average reduction rates of 12.87%, 36.60%, and 15.07% across all floors, respectively. Under Y-direction rare earthquakes, the Y-direction drift angle reduction shows limited effectiveness under NR1.1-1 and NR1.1-2

Table 3 Seismic conditions

Condition	Structural X-axis seismic input		Structural Y-axis seismic input	
	Seismic wave	PGA (m/s ²)	Seismic wave	PGA (m/s ²)
1	AWX0.9	0.35	AWY0.9	0.30
2	AWY0.9		AWX0.9	
3	NRX0.9-1		NRY0.9-1	
4	NRY0.9-1		NRX0.9-1	
5	NRX0.9-2		NRY0.9-2	
6	NRY0.9-2		NRX0.9-2	
7	AWX0.9	1.00	AWY0.9	0.85
8	AWY0.9		AWX0.9	
9	NRX0.9-1		NRY0.9-1	
10	NRY0.9-1		NRX0.9-1	
11	NRX0.9-2		NRY0.9-2	
12	NRY0.9-2		NRX0.9-2	
13	AWX1.1	2.00	AWY1.1	1.70
14	AWY1.1		AWX1.1	
15	NRX1.1-1		NRY1.1-1	
16	NRY1.1-1		NRX1.1-1	
17	NRX1.1-2		NRY1.1-2	
18	NRY1.1-2		NRX1.1-2	

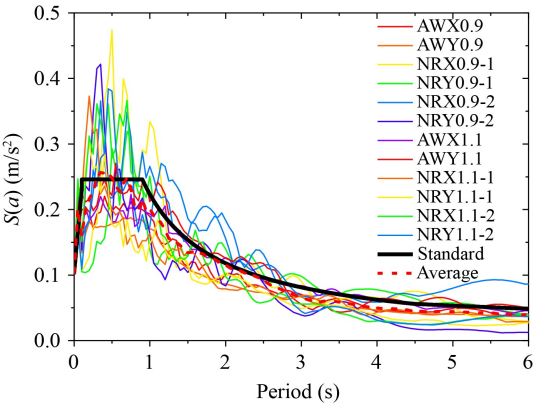
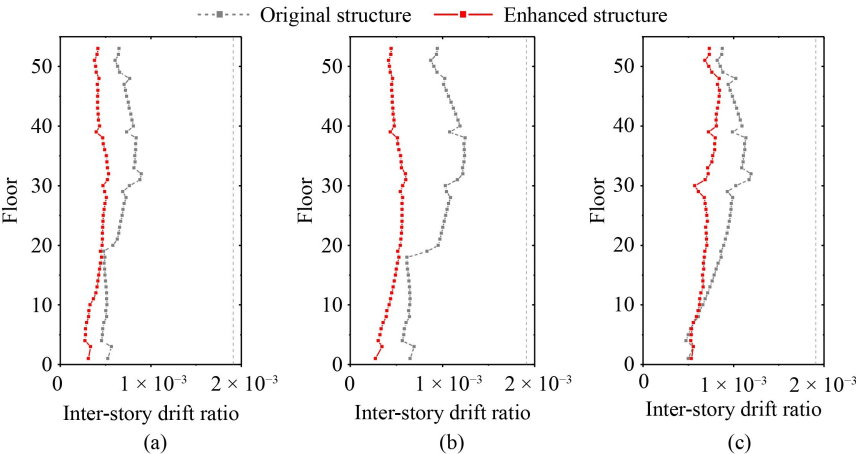


Fig. 23 Seismic response spectrum curve.



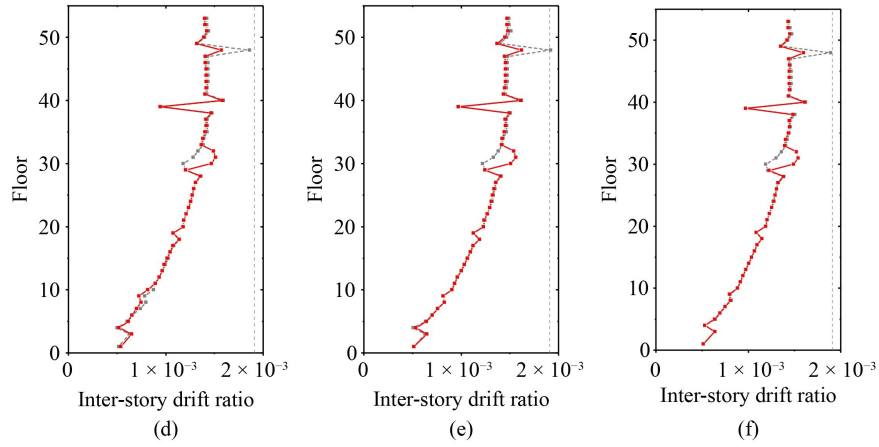


Fig. 24 Inter-story drift ratios under 0.35 m/s^2 dominant-direction excitations: (a) AWX0.9; (b) NRX0.9-1; (c) NRX0.9-2; (d) AWY0.9; (e) NRY0.9-1; (f) NRY0.9-2.

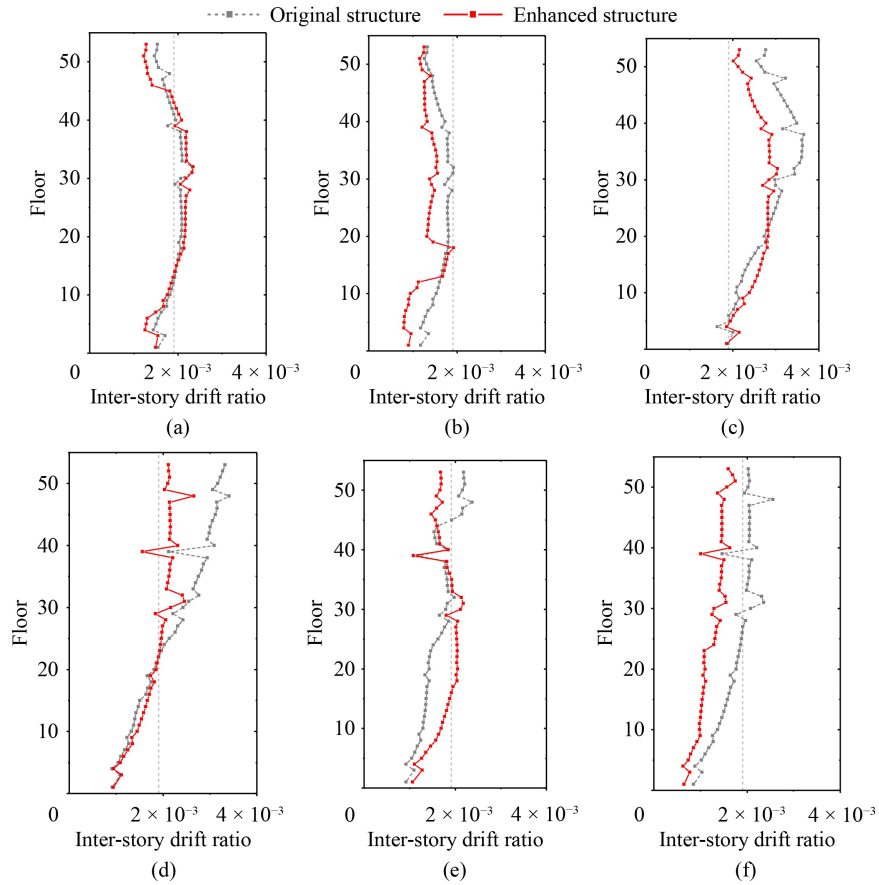


Fig. 25 Inter-story drift ratios under 1.00 m/s^2 dominant-direction excitations: (a) AWX0.9; (b) NRX0.9-1; (c) NRX0.9-2; (d) AWY0.9; (e) NRY0.9-1; (f) NRY0.9-2.

waves whereas improved performance in mid-height regions under the AWY1.1 wave. Overall, the incorporation of TSA-SHBJ effectively reduces inter-story drift ratios across multiple seismic conditions.

4.4.3 Joint hysteresis analysis

In the enhanced structural system with TSA-SHBJs,

hysteresis curves from representative units at 10F, 20F, 30F, 40F and the top floor are presented to reveal the global performance of these energy-dissipating components. Taking the 1.00 m/s^2 AWY0.9 wave and 2.00 m/s^2 AWY1.1 wave as examples, Figs. 27 and 28 show the hysteresis curves of joints. It can be seen that under 1.00 m/s^2 design-basis earthquakes, the structural response intensifies significantly, with TSA-SHBJs

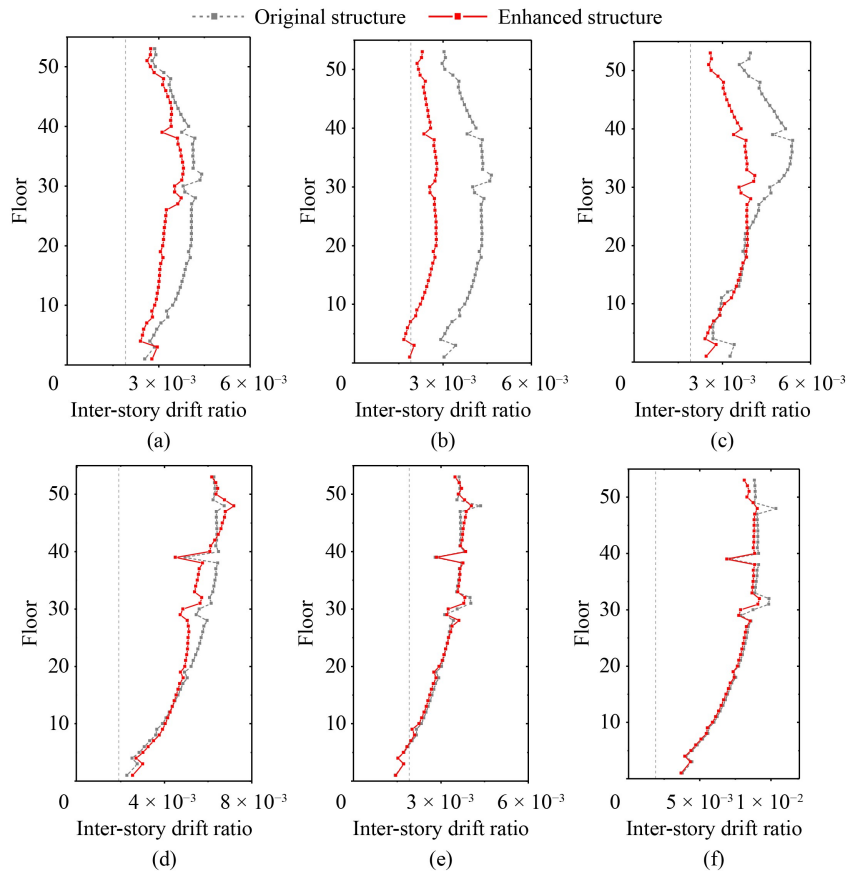


Fig. 26 Inter-story drift ratios under 2.00 m/s^2 dominant-direction excitations: (a) AWX1.1; (b) NRX1.1-1; (c) NRX1.1-2; (d) AWY1.1; (e) NRY1.1-1; (f) NRY1.1-2.

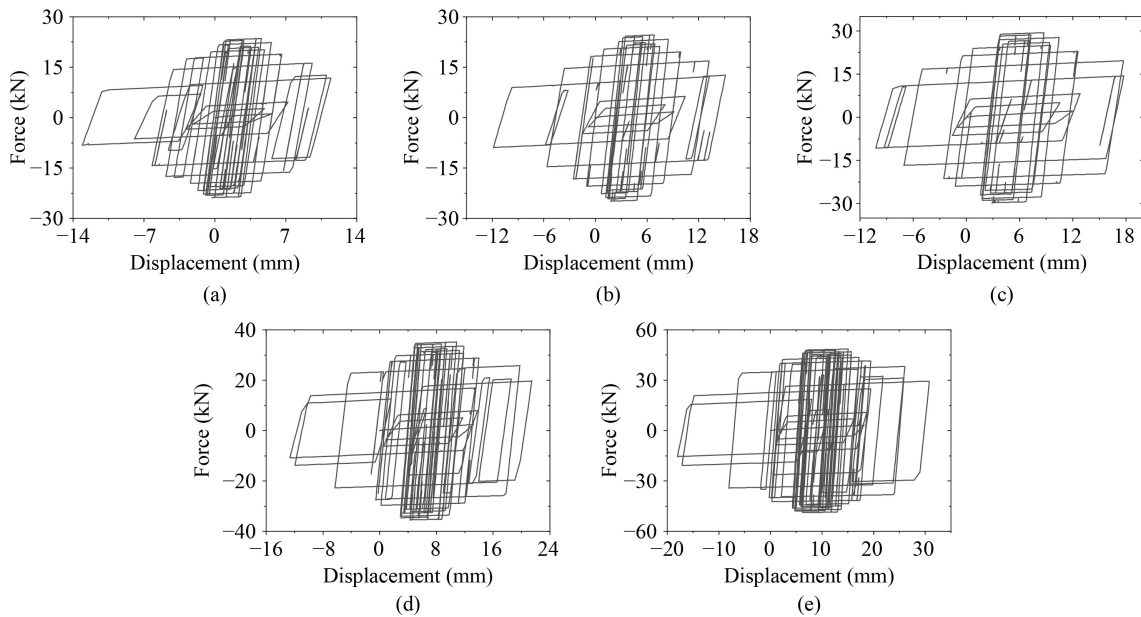


Fig. 27 Hysteresis curve of Y-directional joint under AWY0.9 wave: (a) 10F; (b) 20F; (c) 30F; (d) 40F; (e) top floor.

achieving maximum reciprocating deformations of approximately 50 mm. These joints develop full hysteresis loops, demonstrating substantially enhanced energy dissipation capacity. Under 2.00 m/s^2 rare earthquakes, the structural response intensifies further. Conversely, the

Y-direction joints show no significant improvement in axial force or deformation amplitude, leading to reduced response reduction performance compared with 1.00 m/s^2 earthquakes.

Table 4 details the joint energy distribution within the

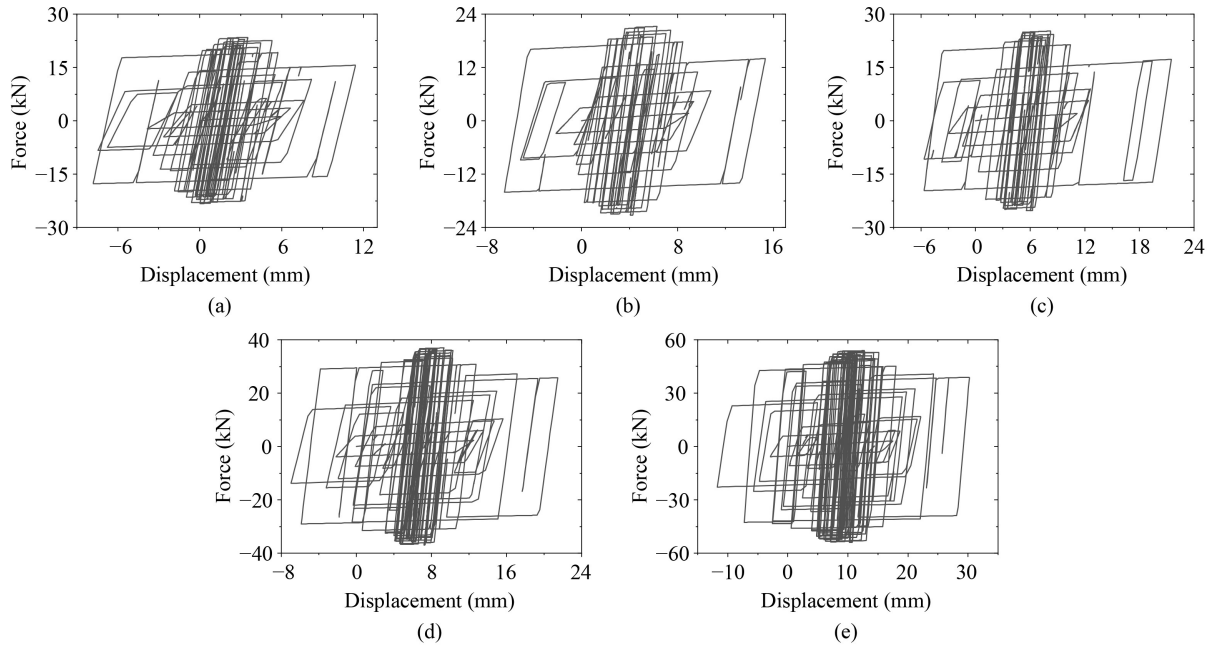


Fig. 28 Hysteresis curve of Y-directional joint under AWY1.1 wave: (a) 10F; (b) 20F; (c) 30F; (d) 40F; (e) top floor.

Table 4 Joint energy dissipation

Seismic condition	PGA (m/s^2)	Input energy	Joints energy dissipation	Ratio of joint energy consumption to input energy (%)
AWX0.9	1.00	95977.9	2403.8	2.5
AWY0.9		92355.0	2253.0	2.4
NRX0.9-1		96787.2	3578.4	3.7
NRX0.9-2		111970.5	2668.3	2.4
NRX0.9-2		100834.9	2500.9	2.5
AWX1.1		336689.7	15158.7	4.5
AWY1.1		334531.3	15882.5	4.7
NRX1.1-1		248007.3	14826.2	6.0
NRX1.1-2		456736.2	30698.3	6.7
NRX1.1-2		501377.0	28963.3	5.8
NRX1.1-1		224984.0	13732.5	6.1
NRX1.1-2		456736.2	30698.3	6.7

structural system. Under design-basis earthquake excitations, the energy dissipation by the joints is relatively limited, accounting for an average of 2.8% of the total input energy. As the PGA intensifies to rare earthquake levels, the hysteretic energy dissipation contribution from the joints increases significantly, reaching up to 6.7% of total energy input (as observed under NRX1.1-2 seismic excitation), demonstrating substantially enhanced energy dissipation performance. For the structural system in this study, the TSA-SHBJ joint modification at the 10th floor introduces 1053 kg of additional mass (0.3 % of story weight) through TSA implementation. Considering the low additional cost and negligible mass increase from incorporating TSA, the TSA-SHBJ joint configuration offers a cost-effective solution that enhances the energy

dissipation capacity of building structures.

5 Conclusions

This study develops a type of TSA-SHBJ for beam-to-column connection for eccentric core-tube frame high-rise structures. Cyclic loading tests are conducted to systematically examine the failure mechanisms and fundamental mechanical performance of TSA-SHBJ. A numerical model of an eccentric core-tube frame structure with TSA-SHBJs is developed to investigate the performance of this enhanced structural system under wind and seismic excitations. Main conclusions are drawn as follows.

1) The deformation mechanism of TSA-SHBJ involves initial bolt slippage along elliptical holes due to beam inclination, leading to angle steel rotation and deformation. The failure progression reveals the complete degradation mechanism of the joint, transitioning from the friction-slip phase to the bolt failure phase.

2) The proposed TSA-SHBJ joint demonstrates superior seismic performance compared to SHBJ joint, exhibiting 99.5% higher rotational stiffness, 35.6% greater ductility, and 9.7-fold energy dissipation capacity. The combined action of friction effects and top/seat angle steels significantly enhances moment resistance and delays collapse, transforming the joint behavior from brittle to ductile failure modes.

3) The enhanced structural system with TSA-SHBJs demonstrates significant vibration reduction under wind loads, decreasing peak displacements more than 30% in the dominant-direction wind and seismic excitations compared with the original structure, with coupled lateral-torsional effects reduced up to 63.4%. The TSA-SHBJ joints significantly improve seismic energy dissipation (up to 6.7% of total input) under rare earthquakes while adding negligible mass (0.3%), offering a highly cost-effective seismic enhancement solution.

This study has confirmed the effectiveness of TSA-SHBJs in enhancing the seismic and wind performance of eccentric core-tube frame structures through improved energy dissipation. Future research could focus on examining the broad applicability of TSA-SHBJs by extending their use to different structural systems, evaluating their functionality under multiple hazards such as mainshock-aftershock sequences, and integrating with machine learning frameworks that leverage large earthquake datasets to enable more comprehensive performance assessments.

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Competing interests The authors declare that they have no competing interests.

References

1. Bu G B, Chen K X Y, Zhou J, Ma X Y, Xu C, Liu F C, He A. Seismic fragility assessment of eccentric gravity column-core tube structures subjected to pulse-like ground motions. *Structures*, 2024, 69: 107406
2. Yin G H, Zhang D K, Ma X X, Wang K, Wang X Y, Lin X H. Simulation and deformation analysis of construction process of large eccentric frame-core tube structure. *The Structural Design of Tall and Special Buildings*, 2025, 34(1): e2198
3. Li X, Liu J P, Wang L, Ma Y F, Chen Y F. Seismic behavior and design of concrete-filled steel tubular frame-RC core tube structures. *Engineering Structures*, 2024, 321: 118945
4. Zhang X F, Far H. Seismic behaviour of high-rise frame-core tube structures considering dynamic soil-structure interaction. *Bulletin of Earthquake Engineering*, 2022, 20(10): 5073–5105
5. Elias S, Beer M, Chen J. Assessing seismic vulnerability of modular buildings under earthquake ground motions. *Engineering Structures*, 2025, 331: 120002
6. Wang P, Shi Q X, Wang F, Guo S. Damage assessment of an SRC frame-core tube structure under the action of a main aftershock sequence. *Advances in Civil Engineering*, 2020, 2020: 1308530
7. Zhang S Y, Shi T, Ma L P. Seismic vulnerability analysis of high-rise SRCFrame-RC core tube hybrid construction. *IOP Conference Series: Earth and Environmental Science*, 2019, 283(1): 012024
8. Taranath B S. *Structural Analysis and Design of Tall Buildings: Steel and Composite Construction*. Boca Raton: CRC Press, 2012
9. Xiong M Q, Xiong J G, Hu S J, Liu Y Q, Wang K. Experiment and analysis on seismic performance of a self-centering Y-eccentrically braced frames structure. *Journal of Building Engineering*, 2024, 97: 110683
10. Shan W C, Zhou X H, Lin X C, Bao L J, Chen Y F. Study on seismic behavior and collapse risk of super high-rise braced mega frame-core tube structural system. *Soil Dynamics and Earthquake Engineering*, 2024, 177: 108409
11. Xie L Y, Yang Z J, Xue S T, Gong L, Tang H S. Topology optimization analysis of a frame-core tube structure using a cable-bracing-self-balanced inerter system. *Journal of Building Engineering*, 2024, 95: 110210
12. Shahrooz B M, Deason J T, Tunc G. Outrigger beam-wall connections. I: Component testing and development of design model. *Journal of Structural Engineering*, 2004, 130(2): 253–261
13. Shahrooz B M, Tunc G, Deason J T. Outrigger beam-wall connections. II: Subassembly testing and further modeling enhancements. *Journal of Structural Engineering*, 2004, 130(2): 262–270
14. Chen Y F, He H X, Wang B S, Lan B J, Cheng Y. Dynamic model, performance analysis and evaluation of frame-distributed rocking-core tube structure. *Structures*, 2023, 57: 105233
15. Li Y L, Zhang Y X, Xiao C Y, Wu B L, Jin B W. Performance study of eccentrically braced steel frames with core tube flange connection. *Journal of Constructional Steel Research*, 2024, 216: 108608
16. Yang Y M, Yang P, Zhu X K, Cui J, Liu X L. Seismic performance of steel frames with different configurations of self-

- centering joints. *Structures*, 2025, 74: 108480
17. China-GB. Standard for Design of Steel Structures, GB 50017-2017. Beijing: China Architecture & Building Press, 2017 (in Chinese)
 18. China-GB/T. Hexagon Head Bolts, GB/T 5782-2016. Beijing: Standards Press of China, 2016 (in Chinese)
 19. China-GB/T. Metallic Materials—Tensile Testing—Part 1: Method of Test at Room Temperature, GB/T 228.1-2021. Beijing: Standards Press of China, 2021 (in Chinese)
 20. China-GB/T. Steel and Steel Products—Location and Preparation of Samples and Test Pieces for Mechanical Testing, GB/T 2975-2018. Beijing: Standards Press of China, 2018 (in Chinese)
 21. China-JGJ/T. Specification for Seismic Test of Buildings, JGJ/T 101-2015. Beijing: China Architecture & Building Press, 2015 (in Chinese)
 22. China-GB. Code for Seismic Design of Buildings, GB 50011-2010. Beijing: China Architecture & Building Press, 2010 (in Chinese)