

Article

Seismic Performance of Prefabricated Steel–Timber Hybrid Joint: Experimental and Numerical Study

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Abstract

Existing steel–timber structures may exhibit limited composite action between the two materials. To address this issue, this study proposes a novel sleeve-connected prefabricated steel–timber (SCPST) beam–column joint that utilizes steel sleeves and high-strength bolts for assembly. To assess its seismic performance, five full-scale interior joint specimens with different beam-sleeve thicknesses and lengths were tested under low-cycle reversed loading. The experimental results revealed that deformation and damage were concentrated at the beam ends, whereas the timber columns remained essentially elastic, satisfying the strong-column–weak-beam seismic design objectives. Within the tested specimen matrix, the peak load of the specimen with an 8 mm beam-sleeve thickness was 43.75% higher than that of the 4 mm specimen, while that of the specimen with a 400 mm sleeve length was 17.46% higher than that of the 300 mm specimen. Frictional slip at the timber–steel interfaces and yielding of the stiffeners were identified as the primary energy-dissipation mechanisms. Simplified analytical expressions based on Eurocode 3 and Chinese timber design provisions were developed to evaluate three potential beam-end failure modes, and the calculated resistances differed from the experimental results by less than 10%. A simplified finite element model was also established in ABAQUS. The simulated failure patterns and peak loads were in reasonable agreement with the experimental observations, with average peak-load errors of 2–6%. These results indicate that the analytical resistance estimates and simplified numerical model can provide a preliminary basis for evaluating the resistance and global response of SCPST joints within the investigated parameter range.

Keywords: prefabricated structure; steel–timber hybrid structure; end-plate connection; seismic performance



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1. Introduction

Timber structures are an indispensable component of global architectural heritage. In China, traditional timber buildings utilised sophisticated mortise and tenon systems that enabled load transfer through beam and column frameworks [1–4]. Although these systems demonstrated considerable architectural adaptability, their structural performance was strongly dependent on craftsmanship, connection integrity, and the natural variability of wood. Compared with conventional reinforced-concrete and steel systems, timber structures have reduced structural self-weight and lower environmental impacts. However, the widespread adoption of reinforced concrete and steel in the twentieth century

gradually reduced the use of timber in modern construction. Meanwhile, conventional timber systems became increasingly limited in their ability to meet the increasing demands for large spans, high load-carrying capacity and structural regularity, which have subsequently been alleviated by advances in engineered timber products and modern connection technologies [5,6].

Modern wood structures employ engineered timber products, standardized mechanical connections, numerical design methods and industrialized fabrication. The research on wood structures has gradually expanded from the mechanical properties of wood itself to the coordinated behaviour of members, connections and structural systems.

Studies on glulam frames [7] and post-tensioned timber joints [8] have demonstrated that connection details, contact behaviour and material nonlinearity strongly affect cyclic performance. Reviews by Rebouças [9] and Chen [10] further indicate that current seismic research increasingly emphasizes ductility, damage control, residual deformation and post-earthquake functionality. Hysteretic energy dissipation and damping characteristics are closely related to material degradation mechanisms under repeated loading conditions [11,12]. These findings highlight the importance of evaluating not only strength degradation but also energy dissipation and stiffness evolution when assessing seismic performance. Beam-to-column joints, therefore, form a critical link between member performance and system response. Introducing steel into the joint region can strengthen the load path, redistribute local stresses and provide more controllable deformation and energy dissipation mechanisms, thereby promoting the development of steel–timber composite connections [13].

Research on steel–timber composite connections can be categorised into three branches. The first treats interfacial slip between steel and timber. Yang [14] and Hassanieh [15] investigated the load slip behaviour of steel connections with LVL and CLT components. Their studies demonstrated that the mechanical response of the interface is closely related to connector configuration and local timber properties. For adhesively bonded connections, Ling [16] and Azinović [17] conducted pull-out tests on glued-in steel rods to examine bond behaviour and withdrawal resistance. Ling [18] further developed an analytical model based on Volkersen theory. Collectively, these studies indicate that steel–timber interfacial slip is governed by the combined effects of connector geometry and stiffness, bonded or embedded length, timber properties and grain orientation, adhesive and interface characteristics and the degree of normal restraint at the interface. The second aspect focuses on improving seismic performance through local steel reinforcement. Studies on side steel plates [19], prestressed steel strips [20], double U-shaped welded steel sections [21], knee-brace reinforcement [22] and coupled long steel rods with screwheads [23] revealed that appropriately designed steel components can suppress timber splitting, enhance moment resistance and rotational stiffness, improve energy dissipation and reduce stiffness degradation by transferring part of the deformation and damage demand from the timber members to the steel components. The third aspect concerns damage control and post-earthquake repairability. Chen [24,25] developed self-centring steel–timber connections and frames to reduce residual deformation. Other studies introduced semi-rigid steel coupling components [26], controlled friction devices [27] and replaceable steel fuses [28] to concentrate inelastic deformation in designated steel components and limit irreversible timber damage.

Existing studies have collectively demonstrated that steel components can compensate for the relatively low local stiffness and strength of timber. However, different connection forms also present several limitations.

Conventional bolted or slotted steel plate details generally require holes or slots to be introduced into the timber, which may cause stress concentration, embedment

deformation and splitting near the connection region. Glued rods, post-tensioning devices, friction assemblies and special yielding components can improve structural performance, but they also introduce additional fabrication procedures and connection parts. However, the application of steel–timber hybrid joints in prefabricated structures still requires connection systems that can achieve efficient assembly while maintaining reliable mechanical performance.

This study proposes a sleeve-connected prefabricated steel–timber (SCPST) beam–column joint. Steel sleeves are used to redistribute local stresses and enhance composite action between the timber and steel components, whereas bolted connections facilitate prefabrication, on-site assembly, future maintenance and possible component replacement. The connection proposed in this study does not require embedded steel plates, thereby reducing the number of openings required in the timber members. Only two bolt holes need to be pre-drilled. In addition, the steel sleeves installed at the beam–column connection can not only simplify the assembly process but also provide additional protection to the timber in the joint region. In particular, the sleeves can effectively protect the timber at the connection from insect attack and decay.

The joint configuration is illustrated in Figure 1. Five specimens were designed with beam-sleeve thickness and length as the principal test variables and were subjected to low-cycle reversed loading. The experimental program was used to quantify failure modes, hysteretic response, load-carrying capacity, ductility, energy dissipation, stiffness degradation and strain development. Analytical resistance models were then established for the observed critical failure sections, and three-dimensional finite element models were developed to reproduce the experimental response and further clarify the governing failure mechanisms.

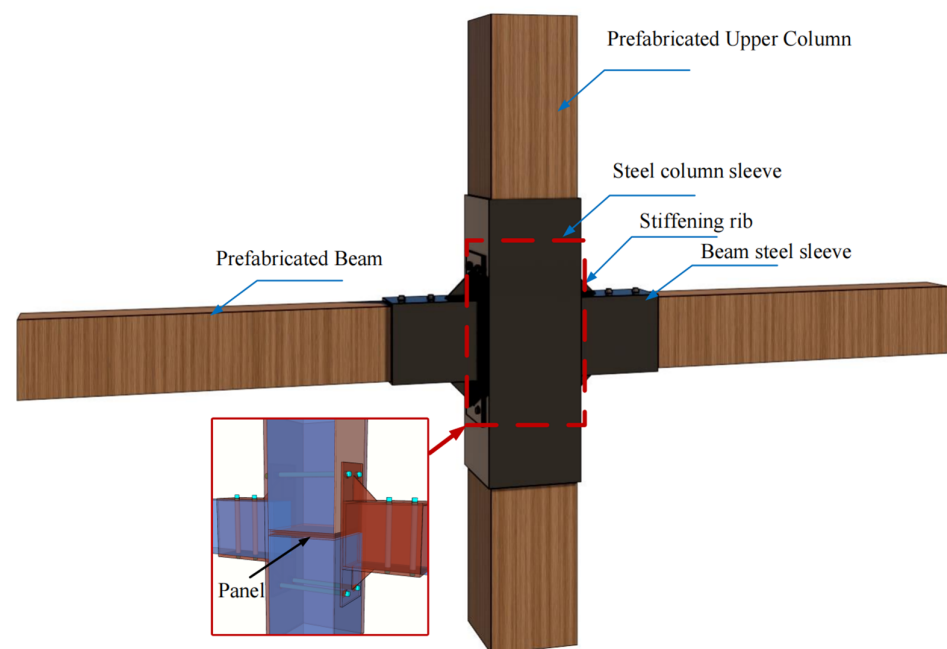


Figure 1. Schematic of the proposed SCPST joint.

2. Experimental Programs

Specimen Design and Testing Program

This experiment investigates typical frame interior joints. Q235 steel, structural-grade Douglas fir lumber, and high-strength bolts were used for the specimens. The assembly sequence for this novel joint is as follows: (1) Prefabricate steel sleeves for beams/columns and timber members off-site. (2) Transport components to site, assemble columns by

inserting upper/lower columns into column sleeves. (3) Insert prefabricated timber beams into beam-end sleeves and connect via bolting. (4) Connect assembled beams to columns with bolts. The complete workflow is detailed in Figures 2 and 3. The prefabricated timber beams (250×150 mm) and columns (300×300 mm), along with steel sleeve dimensions, are detailed in Figure 4 and Table 1.

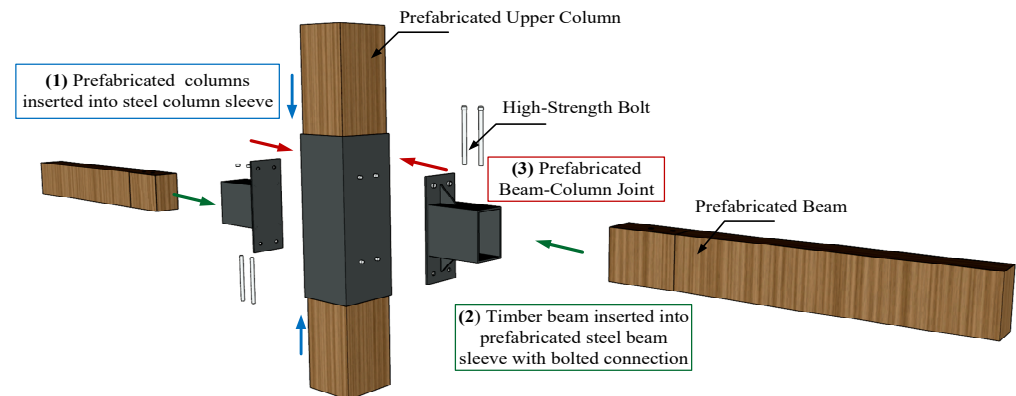


Figure 2. Assembly procedure of the proposed SCPST joint.

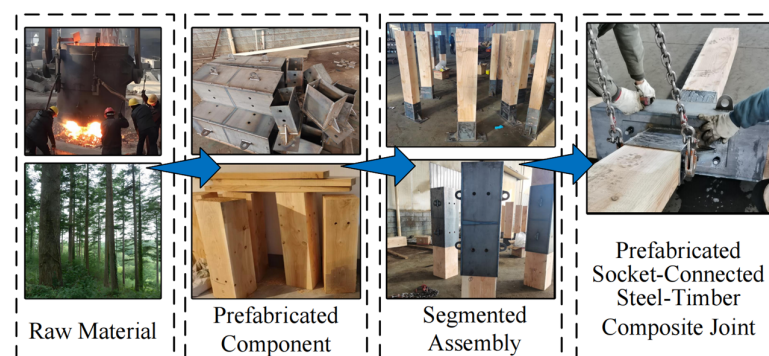


Figure 3. Fabrication process of test specimens.

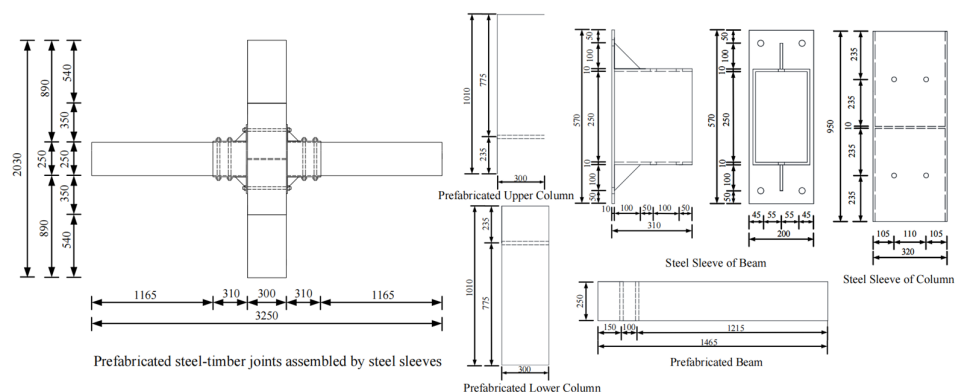


Figure 4. Dimensions and details of test specimens (unit: mm).

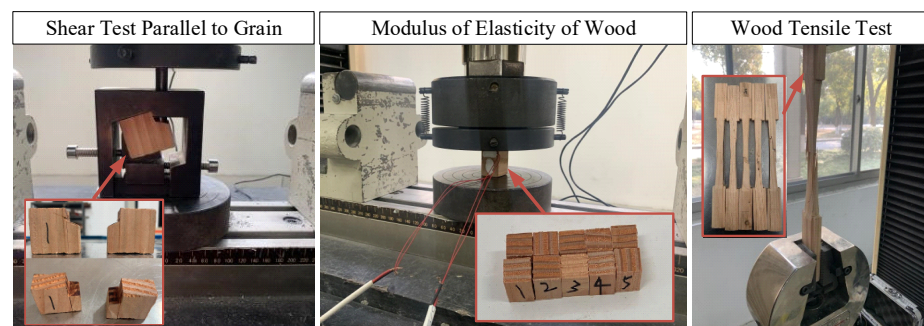
Since the objective of this study was to investigate the influence of connection configuration on the seismic behaviour of the proposed SCPST joint rather than the effect of axial compression ratio, a moderate axial compression ratio was adopted. The axial compression ratio was maintained at 0.08 in this study.

Table 1. Geometric parameters of the specimens.

Specimen	Axial Compression Ratio	Beam Sleeve Thickness (mm)	Beam Sleeve Length (mm)
J1	0.08	4	300
J2	0.08	6	300
J3	0.08	8	300
J4	0.08	6	350
J5	0.08	6	400

It should be noted that, although the bolt diameter was 22 mm, the bolt holes were designed with a diameter of 25 mm to facilitate installation. According to the relevant design code, the calculated bolt pretensioning force was 190 kN. The gaps between the timber members and the steel sleeves were filled with epoxy grout. All welds were designed as groove welds, with the weld strength ensured to exceed that of the steel components so as to prevent weld tearing or failure. For both the beam and column members considered in this study, the timber grain direction was aligned with the longitudinal axis of the member.

Douglas fir, classified as TC15-A in accordance with the code GB 50005-2017 [29], was used in the experimental program. The average measured parallel-to-grain tensile strength, compressive strength and tangential shear strength parallel to the grain were 104.17 MPa, 48.75 MPa and 5.4 MPa, respectively, and the moisture content was maintained at 10.3–14.9%. The beam–column connections were assembled using high-strength bolts in accordance with the relevant provisions of GB 50005-2017 and GB 50017-2017 codes [30]. Material properties were measured following the GB/T 1927-1943 series standards [31–33] for timber specimens and GB/T 228.1-2021 [34] for metallic tensile tests, as illustrated in Figure 5. The measured material parameters are summarized in Tables 2 and 3.

**Figure 5.** Material property tests.**Table 2.** Elastic constants of timber.

Specimen	E_L /MPa	E_R /MPa	E_T /MPa	μ_{LR}	μ_{LT}	μ_{TR}	G_{LT} /MPa	G_{LR} /MPa	G_{TR} /MPa
1	11,341	868	517	0.4	0.47	0.31	680	791	154
2	10,938	826	468	0.35	0.46	0.21	656	801	155
3	12,147	919	497	0.39	0.38	0.21	728	827	159

It should be further noted that: (1) The shear strengths parallel to grain on the radial and tangential sections were determined separately. Five specimens were prepared for each section orientation, and shear tests were conducted using the same universal testing machine. (2) The elastic constants of timber were obtained from clear wood specimens according to the provisions of the timber design standard GB 50005-2017. The clear wood

specimens were machined into dimensions of 30 mm × 20 mm × 20 mm. Three groups of specimens were prepared according to the longitudinal, radial and tangential directions, with five specimens in each group. Strain gauges were attached at the specified locations to measure the deformation response during testing. (3) All material tests were conducted using a UTM5305X microcomputer-controlled electronic universal testing machine (Sansi Zongheng Technology Co., Ltd., Jinan, China). The machine has a maximum test capacity of 300 kN, an accuracy class of 0.5, a force measurement error within $\pm 0.5\%$ of the indicated value, and a displacement resolution of 0.04 μm .

Table 3. Material properties of steel materials.

Components	Diameter & Thickness (mm)	Yield Strength (MPa)	Ultimate Strength (MPa)	Elastic Modulus (GPa)
M22 bolt	22	935.20	1035.00	208.10
Steel plate	4	254.00	401.12	206.00
Steel plate	6	262.12	400.25	206.40
Steel plate	8	266.23	402.45	206.50

In Table 2, E_L , E_R , E_T are the elastic moduli of wood in the longitudinal, radial transverse and tangential transverse directions, respectively; μ_{LR} , μ_{LT} , μ_{TR} are the Poisson's ratios in the three directions, respectively; and G_{LT} , G_{LR} , G_{TR} are the shear moduli in the three directions, respectively.

The loading device is illustrated in Figure 6. According to the equation for calculating the axial compression ratio: $n = N/f_c A$, where, N is the applied axial load, f_c is the measured compressive strength parallel to grain of timber, and A is the column cross-sectional area. For the tested timber columns (300 mm × 300 mm), using the measured compressive strength of 48.75 MPa, the applied axial load is calculated as 360 kN. The loading protocol for this test was referenced from the CUREE (Consortium of Universities for Research in Earthquake Engineering) displacement-controlled cyclic loading protocol [35], which effectively simulates the seismic performance of light wood structures. The loading regime is illustrated in Figure 7. Considering the relatively large deformation capacity of timber connections reported in previous studies, a reference displacement of 100 mm was adopted in this study.

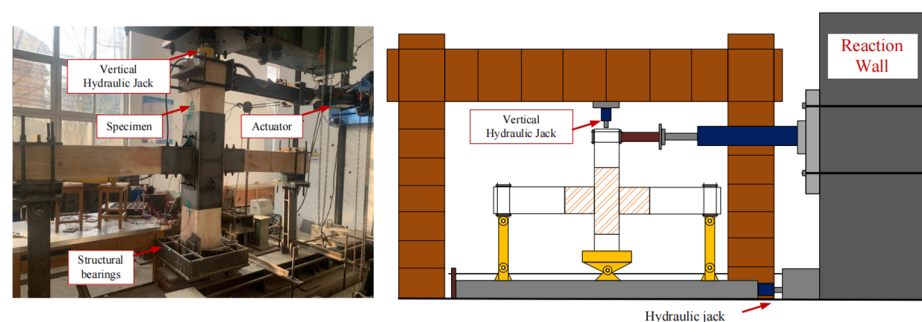


Figure 6. Loading device of the test.

The loading procedure comprised a preloading stage followed by formal cyclic loading. During preloading, the displacement was limited to 10 mm and applied at a constant rate of 5 mm/min. During formal loading, the rate was 20 mm/min until the displacement amplitude reached 70 mm, after which it was increased to 50 mm/min. The initial sequence comprised six constant-amplitude levels of 0.05 Δ , 0.1 Δ , 0.2 Δ , 0.3 Δ , 0.5 Δ and 0.7 Δ . Subsequent loading consisted of primary and secondary cycles. The primary-cycle amplitude

increased progressively from 0.75Δ to 1.5Δ , whereas each secondary level comprised two cycles at 75% of the corresponding primary-cycle amplitude. Load–displacement hysteresis curves and strains at critical locations were continuously recorded. All strain gauges attached to timber members were aligned parallel to the grain.

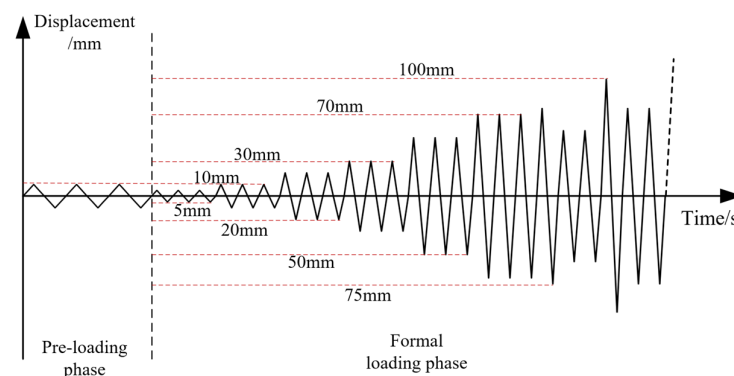


Figure 7. Loading regime of the test.

To facilitate load case definition, column-end loading adopts the convention where actuator movement to the left is designated as positive loading and movement to the right as negative loading. Owing to the actuator's safe displacement limit of ± 200 mm and considering both safety and deformation control, the test termination criteria are as follows:

1. Reaching 150 mm absolute displacement;
2. Severe deformation of the beam-end steel sleeve resulting in load-carrying capacity dropping below 85% of the peak load, at which point structural integrity is critically compromised.

3. Experimental Results of Tests

3.1. Damage Phenomena and Analysis

Withdrawal of the timber beam from the steel sleeve did not occur in any of the specimens during testing. Damage was concentrated at the beam ends and consisted primarily of excessive deformation of the beam steel sleeves or cracking and fracture of the timber beams. Nevertheless, the five specimens developed different dominant failure characteristics.

At the initial loading stages, small construction clearances between the steel sleeves and timber members permitted relative slip despite bolt pretension, resulting in reduced initial stiffness and pinching of the hysteresis loops. At a displacement of approximately ± 50 mm, audible timber cracking occurred in all specimens, and visible cracks developed near the beam end of specimen J3. At approximately ± 75 mm, deformation of the beam-sleeve end plates became noticeable and was accompanied by metallic friction and grinding sounds. When the displacement reached approximately ± 100 mm, pronounced deformation of the beam-end stiffeners was observed in all specimens, together with audible steel yielding. The resistance nevertheless continued to increase with increasing displacement. At ± 150 mm, specimens J1, J2, J4 and J5 exhibited substantial and irreversible deformation of the beam-sleeve end plates, and the tests were terminated due to the failure criteria being reached. In contrast, the end-plate deformation of J3 remained comparatively limited. To identify its ultimate failure mode, J3 was subsequently loaded at 5 mm/min to a displacement of 200 mm. During this stage, the load increased only slightly, whereas sounds of continuous timber cracking were produced near the beam end. At 200 mm, intense cracking was followed by a sudden reduction in load, and fracture was observed near the outer side of the right beam-sleeve region; the test was then terminated.

For all five specimens, deformation and damage were concentrated at the beam ends. Specimens J1, J2, J4 and J5 were governed by large irreversible deformation of the beam-sleeve end plates, indicating that the inelastic response was mainly concentrated in the steel components. In contrast, specimen J3 exhibited diagonal shear cracks near the beam end and subsequent timber damage, indicating a beam-end shear failure mode. The timber columns remained essentially undamaged throughout testing, demonstrating the intended strong-column–weak-beam response. After the tests, the joints were safely disassembled. Significant ovalization and local deformation were observed around the beam-end bolt holes, whereas the bolt holes at the column ends exhibited negligible damage, further confirming that the inelastic demand was mainly concentrated at the beam side of the joint. Representative failure patterns are shown in Figure 8.



Figure 8. Specific failure patterns of specimens.

3.2. Parametric Analysis of the Specimens' Seismic Performance

Load–displacement hysteresis curves reflect the relationship between forces and deformations in frames or joints under simulated seismic actions, serving as the fundamental basis for studying structural seismic performance. The experimentally measured hysteresis curves (P - Δ curves, where P represents horizontal load and Δ denotes horizontal displacement) for the prefabricated steel–timber composite beam–column sleeve connections under low-cycle reversed loading are presented in Figure 9.

The above figure shows the following:

(1) During initial testing (0 mm to ± 25 mm), all five steel–timber frame joints exhibited spindle-shaped hysteresis loops. During the mid-to-late stages (± 25 mm to ± 150 mm), the curves transformed into inverse-S shapes with pronounced pinching effects, indicative of slip between steel sleeves and timber beams. Additionally, the inherent material nonlinearity of wood caused minor asymmetries between positive and negative hysteresis curves.

(2) A comparison of specimens J1, J2, and J3, where steel plate thickness was the sole variable, revealed that specimens with thicker plates exhibited more pronounced inverse-S hysteresis curves. This occurred because thicker plates impart greater stiffness to steel sleeves, reducing coordinated deformation with relatively flexible timber beams.

(3) A comparison of specimens J2, J4, and J5, where steel sleeve length was the sole variable, revealed close resemblance among the hysteresis curves. Within the investigated range, increasing the sleeve length had little influence on the overall shape and pinching of the hysteresis loops, suggesting that sleeve length had a limited effect on interface slip under the present test conditions.

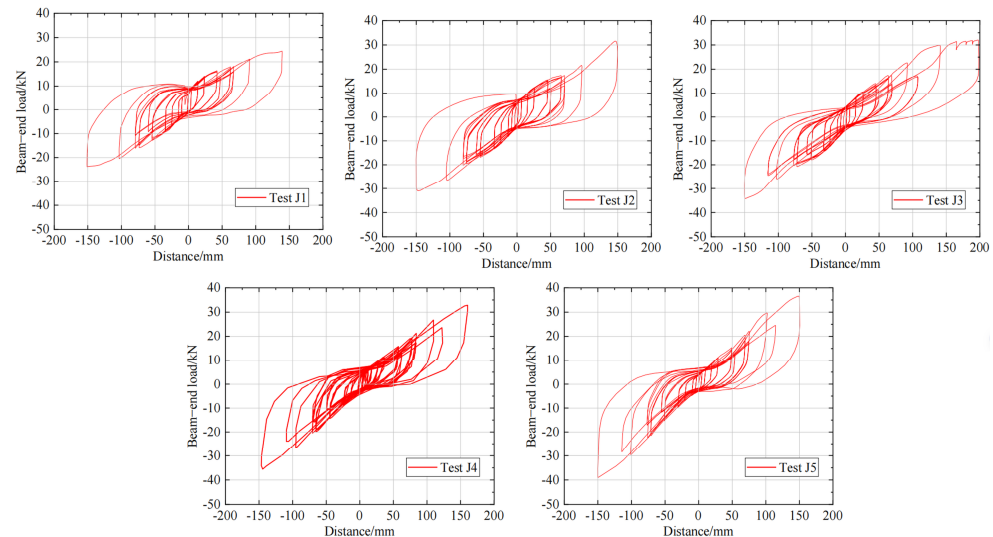


Figure 9. Load–displacement hysteresis curves of specimens.

The skeleton curve was obtained by connecting the load values at corresponding displacements during the first cycle of each loading level in the hysteresis curves, as shown in Figure 10. The ultimate loads in the positive direction for specimens J1–J5 were 24.0 kN, 31.5 kN, 34.5 kN, 33.5 kN and 37.0 kN, respectively. J5 exhibited the highest load-bearing capacity among all specimens. At 150 mm displacement, J5 demonstrated a 20.6% higher load capacity than J2. Compared to J1, J3 (with identical steel sleeve length but 8 mm thick plates) exhibited a 43.7% capacity increase. Within the tested specimen matrix, the peak loads increased progressively from J1 to J3 as the beam-sleeve thickness increased from 4 to 8 mm. Similarly, J2, J4, and J5 exhibited progressively higher peak loads with increasing sleeve length from 300 to 400 mm. Because only one specimen was tested for each configuration, these differences should be interpreted as observed trends among the tested specimens rather than as generally established quantitative parameter effects.

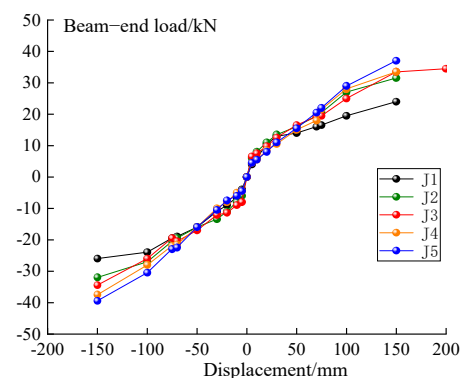


Figure 10. Skeleton curves of specimens.

Under cyclic loading, structural degradation is generally associated with progressive damage accumulation, stiffness deterioration and variation of hysteretic energy dissipation capacity. Similar relationships between cyclic damage evolution and energy dissipation characteristics have been reported in refs [11,12]. Therefore, hysteretic response, equivalent

viscous damping and stiffness degradation were evaluated to characterize the seismic behaviour of the proposed SCPST joints.

Energy dissipation capacity and ductility coefficients serve as key metrics for evaluating structural seismic performance and can thus quantify energy absorption during earthquakes [36]. Energy dissipation is characterized by the equivalent viscous damping coefficient h_e , calculated using Equation (1). The calculation diagram is shown in Figure 11. For comparative purposes, a test-defined ductility ratio, μ , was calculated as the ratio of the test-defined ultimate displacement Δu to the yield displacement Δy , where Δy was determined using the equal-energy method [37]. Because the specimens generally did not exhibit a clear post-peak strength degradation stage before test termination, Δu was uniformly taken as 150 mm, corresponding to the prescribed displacement-based termination criterion. Therefore, the resulting μ values should be interpreted as test-defined ductility indices for comparison among the tested specimens rather than as the actual ultimate ductility capacities of the joints. The calculated equivalent damping coefficients and test-defined ductility ratios for all specimens are listed in Tables 4 and 5.

$$h_e = S_{ABCD} / 2\pi(S_{OBE} + ODF) \quad (1)$$

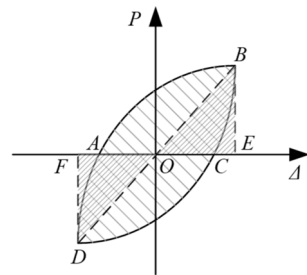


Figure 11. Schematic of equivalent viscosity coefficient calculation.

Table 4. Comparison of test-defined ductility ratios of the specimens.

Specimen	Load Direction	Δy (mm)	Test-Defined Ultimate Displacement (mm)	μ_{Δ}	Average (μ_{Δ})
J1	Positive	84	150	1.78	2.10
	Negative	−62	−150	2.42	
J2	Positive	54	150	2.78	2.73
	Negative	−56	−150	2.68	
J3	Positive	43	150	3.49	3.38
	Negative	−46	−150	3.26	
J4	Positive	41	150	3.66	3.61
	Negative	−42	−150	3.57	
J5	Positive	33	150	4.54	4.25
	Negative	−38	−150	3.95	

The equivalent viscous damping coefficient at each displacement amplitude was calculated based on the first complete loading cycle corresponding to that amplitude. The hysteretic energy dissipation area and elastic strain energy area were obtained from the measured load–displacement curve of the first cycle. Taking specimen J2 at a displacement amplitude of 10 mm as an example, the initial hysteresis curve of which is shown in Figure 12, the enclosed hysteretic energy dissipation area S_{ABCD} is 171.92 kN·mm and

the elastic strain energy areas S_{OBE} and S_{ODF} were 67.93 kN·mm. The equivalent viscous damping coefficient of J2 in 10 mm level is 0.403.

Table 5. Equivalent viscous damping coefficients at different displacement amplitudes.

Specimen	10 mm	30 mm	50 mm	70 mm	75 mm	100 mm	150 mm
J1	0.477	0.275	0.270	0.266	0.265	0.260	0.235
J2	0.403	0.299	0.280	0.252	0.251	0.245	0.227
J3	0.365	0.281	0.245	0.201	0.195	0.184	0.175
J4	0.298	0.198	0.183	0.175	0.172	0.170	0.151
J5	0.315	0.212	0.186	0.168	0.165	0.164	0.157

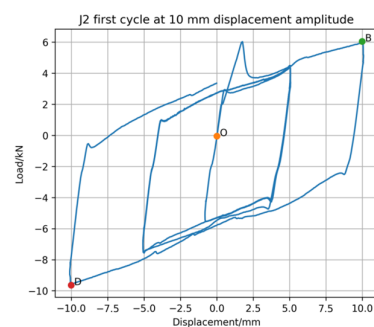


Figure 12. Initial hysteresis curves of J2 in 10 mm level.

Tables 4 and 5 indicate the following:

(1) The energy dissipation capacity of the novel joint decreased with increasing displacement cycles. Primary dissipation mechanisms comprised the following: (a) finite friction sliding at wood–steel interfaces during pre-yield stages and (b) yield dissipation of end-plate stiffeners post-yielding. All five specimens exhibited high initial energy dissipation, with the initial equivalent viscous damping coefficients ranging from 0.298 to 0.477.

(2) Among the tested specimens, higher test-defined ductility ratios were observed for specimens with thicker or longer beam-end steel sleeves. Specifically, the average test-defined ductility ratios of J2 and J3 were 30.00% and 60.95% higher than that of J1, respectively, whereas those of J4 and J5 were 32.23% and 55.68% higher than that of J2. These values are based on the prescribed displacement-based termination criterion and should therefore be interpreted as comparative test-defined indices rather than as measured ultimate ductility capacities.

(3) It should be noted that the equivalent viscous damping coefficient represents the ratio between hysteretic energy dissipation and elastic strain energy rather than the absolute energy dissipation capacity. Therefore, the relatively high values at small displacement amplitudes should not be directly interpreted as higher total energy dissipation. The relatively high damping coefficients at 10 mm displacement amplitude are mainly related to the closure of initial construction clearances and frictional sliding at the timber–steel interfaces. At the initial loading stage, interface friction contributes significantly to the hysteretic response due to relative slip between the steel sleeves and timber members. With increasing displacement amplitude, although frictional sliding remains present, its relative contribution decreases as steel stiffener yielding and local component deformation become the primary energy dissipation mechanisms. Consequently, the equivalent viscous damping coefficient gradually decreases at larger displacement amplitudes.

Under reversed loading, cumulative damage develops between steel sleeves and timber members. The progressive reduction in structural stiffness due to accumulated

damage is termed stiffness degradation. Structural stiffness is defined as secant stiffness, calculated as the ratio of peak horizontal load to corresponding displacement per cycle. The secant stiffness K_i is computed using Equation (2):

$$K_i = \frac{|+P_i| + |-P_i|}{|+\Delta_i| + |-\Delta_i|} \quad (2)$$

Figures 13 and 14 presents the stiffness degradation curves for all five joints tested in this study. Analysis indicates that:

(1) The normalized stiffness degradation curves indicate that all specimens experienced rapid stiffness reduction during the initial loading stage, followed by a relatively stable degradation stage at larger displacement amplitudes.

(2) Among specimens J1, J2, and J3, the specimens with greater beam-end steel sleeve thickness exhibited higher initial stiffness, with J2 and J3 showing values 50% and 62.5% higher than that of J1, respectively. Among J2, J4, and J5, the initial stiffness values were relatively similar, whereas J5 retained a higher secant stiffness at the end of loading. These observations represent trends within the tested specimen matrix.

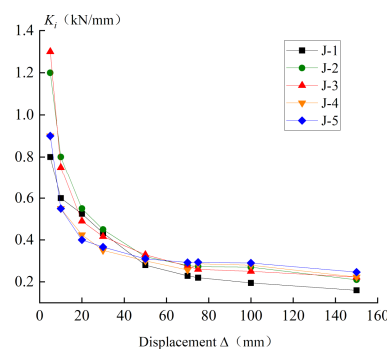


Figure 13. Stiffness degradation curves of specimens.

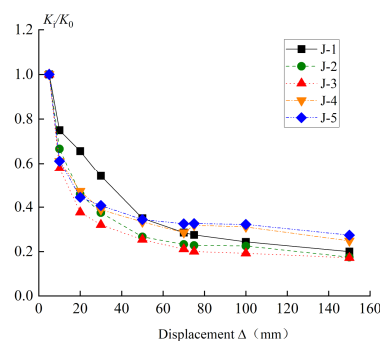


Figure 14. Normalized secant stiffness index of specimens.

3.3. Strain Analysis of Specimens

During the low-cycle reversed-loading tests, strain gauges were used to continuously record the strain histories at the designated measurement locations. The corresponding column-end displacement was synchronized with the strain measurements to facilitate comparison between the global and local responses. For the steel sleeves and stiffeners, the displacement and strain values obtained during the first cycle of each loading level were extracted, and displacement–strain curves were constructed to characterize the evolution of strain during cyclic loading. For the timber beams and columns, strain–time histories were evaluated to examine the development of local deformation throughout the loading process.

Figure 15 reveals the following,

(1) The yield strains of different steel components were calculated based on the measured yield strengths and elastic moduli listed in Table 3. The corresponding yield strains of the Q235 steel plates ranged from approximately $1230 \mu\epsilon$ to $1290 \mu\epsilon$. For simplicity, a representative yield strain of $1250 \mu\epsilon$ was adopted to evaluate the yielding state of the steel components. Based on this criterion, the monitored column sleeve regions remained predominantly elastic throughout the cyclic tests, whereas the beam sleeves and stiffeners reached or exceeded the yield strain during the intermediate loading stages. This indicates that the inelastic deformation was mainly concentrated in the beam-side steel components, particularly the stiffeners, which acted as the designated energy-dissipation components of the SCPST joint.

(2) The measured wood strains at different locations exhibited considerable variation among specimens, which was mainly associated with the inherent variability of timber and localized damage development during cyclic loading. The tensile strain capacity of timber was therefore not directly estimated using Hooke's law, since timber exhibits nonlinear behaviour and progressive damage under cyclic loading. However, the measured strains can still provide valuable information regarding the deformation response and damage evolution of the timber members. The results indicate that the steel sleeve does not directly increase the intrinsic ultimate strain capacity of timber. Instead, it modifies the local stress distribution and provides confinement effects, thereby delaying localized damage and improving the deformation capacity of the connection system.

(3) The measured strains in the timber columns remained within the elastic range throughout the tests, further indicating that the inelastic response was concentrated primarily at the beam ends, consistent with the intended strong-column–weak-beam response.

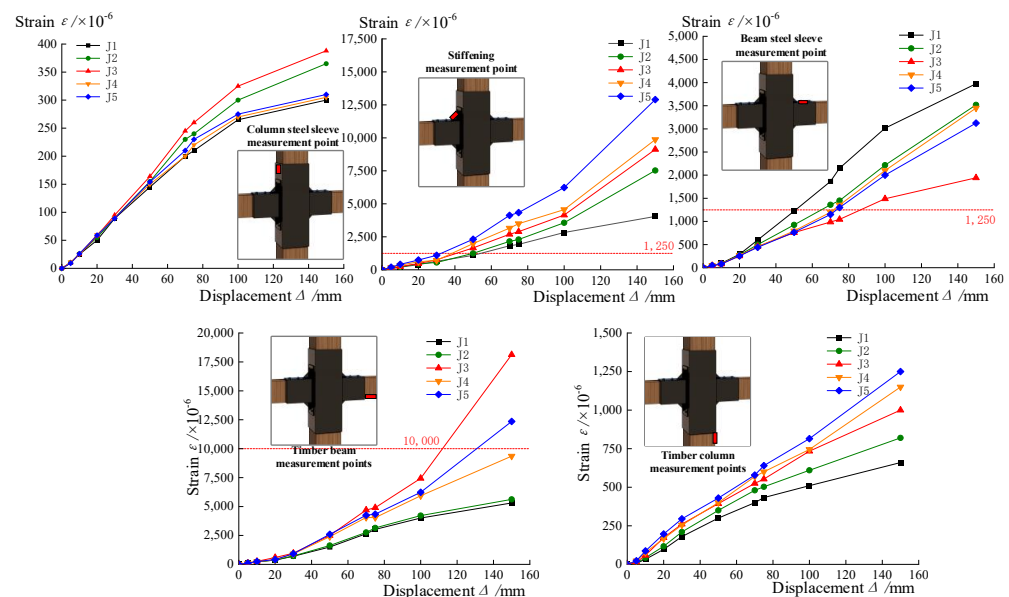


Figure 15. Strain responses at key measurement points of specimens.

4. Bearing Capacity Analysis of Joints

Experimental studies indicate three potential failure zones in the novel beam-end joints: the steel sleeve end-plate section, stiffener section and timber–steel interface section (Figure 16). Failure modes include the following:

- (1) Excessive deformation at sleeve end-plates;
- (2) Stiffener section failure due to stiffener yielding;
- (3) Timber beam fracture (bending or shear) at timber–steel interfaces.

The flexural capacity M_{rd} at the beam end of the novel prefabricated steel–timber hybrid joint can be determined as the minimum value among the capacities of three critical sections, whereas the shear capacity F_{rd} is calculated based on the shear resistance of the timber–steel interfaces. Since the objective of the proposed model is to compare the calculated resistance with experimental results, the measured mean material strengths obtained from the material tests were adopted rather than characteristic or design strengths. The proposed analytical model considers multiple possible failure modes, including steel sleeve deformation, stiffener yielding and timber failure. The governing failure mode is determined by comparing the calculated resistance associated with each failure mechanism.

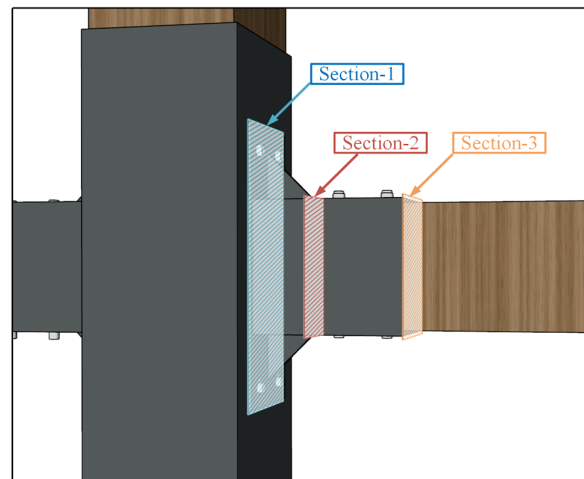


Figure 16. Critical failure sections at the beam end of the SCPST joint.

4.1. Failure Analysis of Excessive Deformation at Sleeve End-Plates

The flexural load capacity was calculated using European code 3 [38], which is based on an equivalent T-piece mode. The upper cover plate and end plates of the beam steel jacket were assumed to be equivalent T-pieces, as shown in Figure 17. The flexural load capacity was derived based on failure mode 1, as shown in Figure 18.

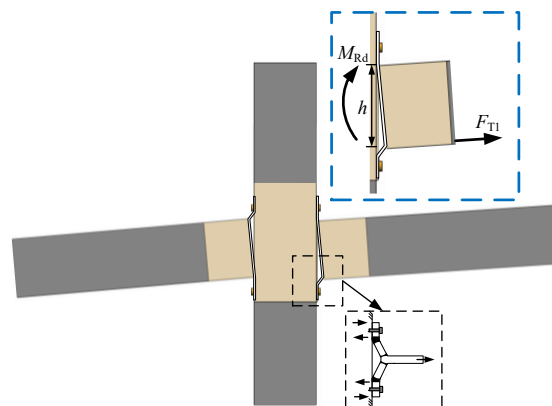


Figure 17. Mechanical model for end-plate deformation.

The equation for the flexural load capacity is as follows:

$$M_{Rd1} = F_{T1}h \quad (3)$$

$$F_{T1} = \frac{(8n - 2e_w)}{2m_e - e_w(m_e + n)} M_{Tp} \quad (4)$$

$$n = \min[e_p, 1.25m_e] \quad (5)$$

$$e_w = 0.25d_n \quad (6)$$

$$M_{Tp} = 0.25l_{eff}t_e^2f_u \quad (7)$$

where M_{Tp} is the bending moment generated by the plastic hinge line at the flange of the equivalent T-piece, m_e is the effective distance between the plastic hinge lines (taken as 175 mm), l_{eff} is the effective length of the plastic hinge line of the equivalent T-piece (600 mm) and t_e is the thickness of the end plate of the steel sleeve of the beam.

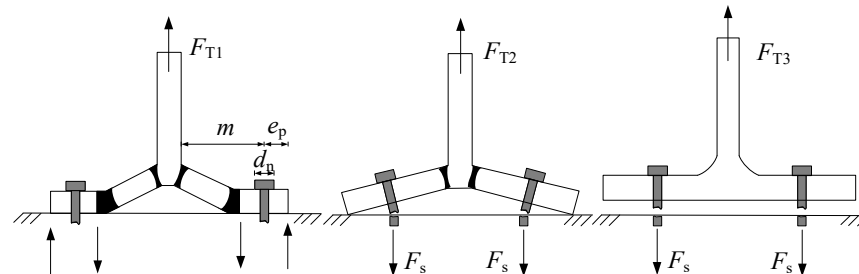


Figure 18. Failure modes of equivalent T-pieces.

4.2. Failure Analysis of Stiffener Yielding

Analysis of experimental failure patterns reveals that stiffener yielding induces local deformations in the cover plate of the steel sleeve. This section thus constitutes a structural weak point. Figure 19 presents the mechanical schematic of this section based on localized deformations and stiffener angles.

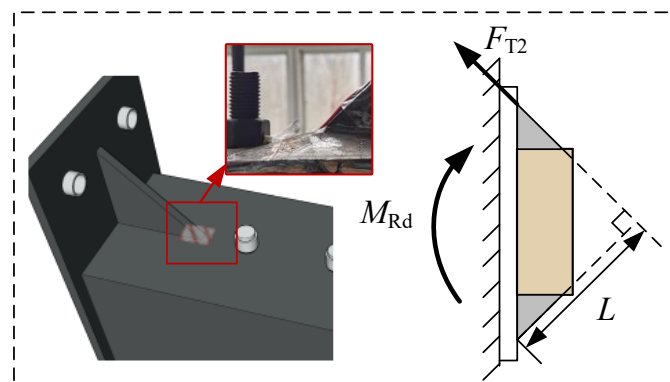


Figure 19. Mechanical model for stiffener yielding.

According to the schematic calculation diagram, the flexural bearing capacity of the joint is:

$$M_{Rd2} = F_{T2}L \quad (8)$$

$$F_{T2} = f_{ty}A_t \quad (9)$$

$$A_t = (t_j + t_b)^2 \quad (10)$$

$$L = \frac{\sqrt{2}}{2}(h_b + 2h_j) \quad (11)$$

where t_j is the stiffener thickness, t_b is the beam steel sleeve thickness, h_b is the beam height, h_j is the stiffener height and f_{ty} is the stiffener yield strength.

4.3. Failure Analysis of Timber-Steel Interface Sections

The timber–steel interface constitutes the joint end and represents a connection weak section. Timber alone resists beam-end transferred bending moments and shear forces,

making this cross-section prone to bending or shear failure. Figure 20 depicts the internal force distribution. The measured timber shear strength was used directly for the shear-resistance estimate, whereas the measured mean tensile strength parallel to grain was used only as a substitute parameter for the approximate bending-resistance estimate, as discussed below.

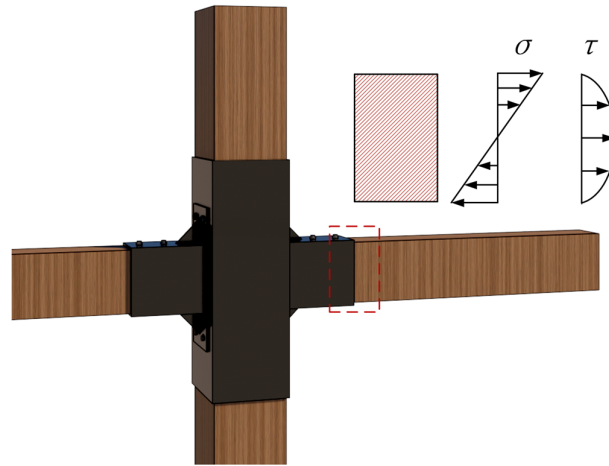


Figure 20. Stress distributions at the timber–steel interface.

For a conventional timber bending check, GB 50005-2017 expresses the sectional bending-resistance relationship as follows:

$$\frac{M}{W_n} \leq f_m \quad (12)$$

$$M_{rd3} = W_n f_m \quad (13)$$

where f_m denotes the timber bending strength required by the conventional formulation and W_n is the section modulus of the rectangular cross-section. However, the timber bending strength was not independently measured in the present study. The measured mean tensile strength parallel to grain (104.17 MPa) was therefore adopted only as a substitute material parameter. Accordingly, the resulting M_{rd3} should be regarded as an approximate bending-resistance estimate used for comparison among the possible failure modes, rather than as a conventional code-based timber bending resistance.

For shear failure at the cross-section, the shear stress distribution decreases farther from the neutral axis, with the maximum shear stress occurring at the neutral axis. As indicated in the literature [39], the maximum shear stress in rectangular timber beams is approximately 1.5 times the average shear stress. The bearing capacity formula for the cross-section, established based on the shear stress distribution, is given below:

$$\frac{F}{A_t} \leq f_v \quad (14)$$

$$F_{rd} = \frac{2}{3} A_w f_v \quad (15)$$

where f_v is the tangential shear strength of timber, F_{rd} is the shear force at the joint, and A_w is the area of the beam cross-section.

4.4. Comparison of Calculated Resistance and Experimental Results

The force diagrams for the five specimens investigated in this study can be obtained from the schematic of the experimental setup, as illustrated in Figure 21. From these force diagrams, the experimental values of the joint beam-end flexural capacity (M_{rdt}) and shear

capacity (F_{rdt}) can be determined. The calculated values of the joint beam-end flexural capacity (M_{rdc}) and shear capacity (F_{rdc}), obtained using the formulas developed in this study, are summarized in Table 6.

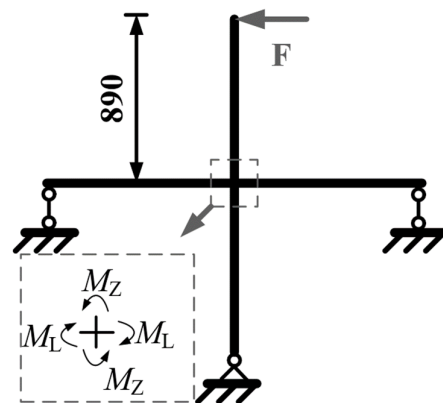


Figure 21. Force diagram of the test specimen.

Table 6. Calculated resistance of different failure modes and comparison with experimental capacity.

Specimen	Calculated Value					Experimental Value		Calculated Value/ Experimental Value	Errors
	M_{rd1} (kN·m)	M_{rd2} (kN·m)	M_{rd3} (kN·m)	M_{rdc} (kN·m)	F_{rdc} (kN)	M_{rdt} (kN·m)	F_{rdt} (kN)		
J1	32.29	22.45	162.5	22.45	21.77	22.76	16.14	0.98	2%
J2	32.29	29.32	162.5	29.32	21.77	28.22	20.02	1.03	3%
J3	32.29	37.11	162.5	32.29	21.77	30.95	21.95	0.99	1%
J4	32.29	37.11	162.5	32.29	27.22	33.68	23.89	0.95	5%
J5	32.29	37.11	162.5	32.29	32.66	35.51	25.18	0.93	7%

Note: All calculated resistances were obtained using the measured mean material strengths from the material property tests. M_{rd3} is an approximate bending-resistance estimate obtained using the measured mean tensile strength parallel to grain as a substitute for the unmeasured timber bending strength; it should not be interpreted as a conventional code-based timber bending resistance.

As shown in Table 6, the calculated resistances were generally consistent with the experimental capacities, with differences of less than 10% for all specimens. These results indicate that the proposed analytical expressions can provide reasonable preliminary estimates of the resistance of the tested SCPST joints. For specimen J3, although timber bending failure was included as one possible failure mode in the analytical framework, the calculated beam-end shear resistance was lower than the bending resistance. Therefore, the governing failure mode was identified as timber shear failure, which agrees with the experimentally observed diagonal shear cracking near the beam end. In contrast, specimens J1, J2, J4 and J5 were governed by steel component deformation, particularly sleeve end-plate and stiffener yielding. Overall, the analytical results were consistent with the governing failure mechanisms observed in the tests; however, further validation over a broader range of geometries, material properties, and loading conditions is required before the proposed expressions can be adopted for general design applications.

5. Finite Element Analysis

To further investigate the seismic behaviour of the proposed prefabricated steel–timber hybrid joint, finite element models corresponding to the five tested specimens were

established in ABAQUS. The finite element models were used to reproduce the observed failure modes and evaluate the load-carrying capacities of the joints.

5.1. Constitutive Relation of Finite Element Model

5.1.1. Timber

This paper simplifies timber as an orthotropic anisotropic material and ignores the creep and relaxation effects of wood. Under compression, timber exhibits marked elastic and plastic deformation, and the tensile and compressive strengths are not equal [40]. Accordingly, the longitudinal compressive response was represented by an ideal elastic–plastic model, the transverse compressive response by an elastic–plastic hardening model and the tensile response in both the longitudinal and transverse directions by a linear elastic model up to failure, as illustrated in Figure 22. In this figure, $f_{c,y}$, $f_{c,u}$ and $f_{t,u}$ denote the compressive yield, ultimate compressive and ultimate tensile strengths of timber, respectively, whereas $\varepsilon_{c,y}$, $\varepsilon_{c,u}$ and $\varepsilon_{t,u}$ denote the corresponding compressive yield, ultimate compressive and ultimate tensile strains. The elastic response was defined using the orthotropic engineering constants, including the elastic moduli, Poisson’s ratios and shear moduli. For the plastic response, the yield stresses and plastic potential function were specified, and anisotropic yielding in the longitudinal and transverse directions was described using the Hill criterion [41]. It should be noted that, after defining the constitutive model, a local material orientation must be assigned to the elements; otherwise, the software cannot correctly identify the longitudinal direction of the timber. The material parameters adopted in the finite element model are summarized in Table 2.

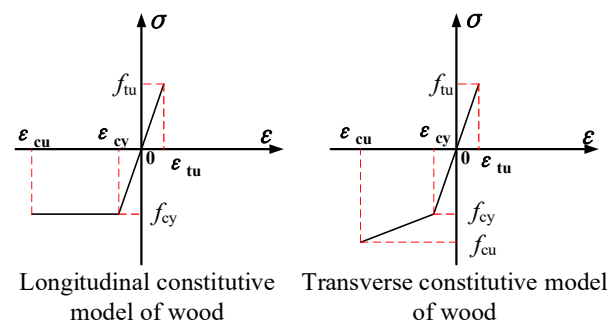


Figure 22. Constitutive relationship of wood.

5.1.2. Q235 Steels and High-Strength Bolts

In this study, Q235 steel and high-strength bolts were modelled using an ideal elastic–perfectly plastic constitutive model [42]. The constitutive relation is given by Equation (16), and the corresponding stress–strain curve is shown in Figure 23.

$$\sigma = \begin{cases} f_{ys} & \varepsilon_y < \varepsilon_s \\ E_{ss}\varepsilon_s & \varepsilon'_y < \varepsilon_s < \varepsilon_y \\ f'_{ys} & \varepsilon_s < \varepsilon'_y \end{cases} \quad (16)$$

In Equation (16), f_{ys} represents the tensile yield strength of Q235 steels and high-strength bolts. E_{ss} denotes the elastic modulus of the two kinds of materials, and f'_{ys} represents the compressive yield strength of the materials. The steel components were modelled using the measured yield strengths obtained from the tensile tests. The yield strengths corresponding to different steel plate thicknesses were assigned according to Table 3.

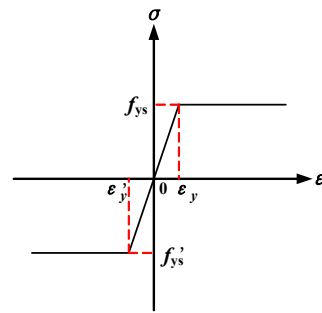


Figure 23. Constitutive relationship of Q235 steels and high-strength bolts.

5.2. Finite Element Modelling Procedure

The timber beams, timber columns, steel plates and high-strength bolts were modelled using eight-joint reduced-integration solid elements (C3D8R). A mesh sensitivity analysis was conducted to evaluate the influence of element size on the numerical response of the SCPST joint. Three critical components, including the timber beam, timber column and steel sleeve, were investigated separately by varying the corresponding mesh sizes while keeping the other modelling parameters unchanged. The load–displacement responses obtained from different mesh configurations are presented in Figure 24. Based on preliminary mesh trials, the mesh sizes were 22.5 mm for the timber beams, 30.0 mm for the timber columns, 15.0 mm for the beam steel sleeves, 25.0 mm for the column steel sleeves and 25.0 mm for the high-strength bolts. Two elements were assigned through the thickness of the plates in the present model. The mesh configurations of several detailed components are shown in Figure 25. The boundary conditions of the finite element model were consistent with those used in the tests, as shown in Figure 26.

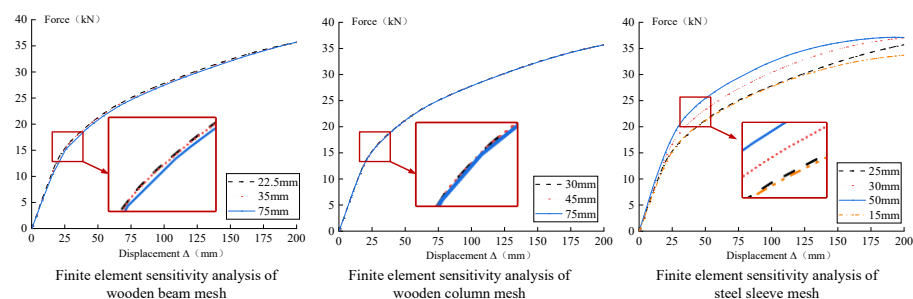


Figure 24. Mesh sensitivity analysis of the model.

Regarding contact interactions, normal contact behaviour was defined as ‘hard contact’, allowing separation after contact to better represent the actual interface conditions. The tangential contact behaviour was modelled using the penalty formulation. The friction coefficient was set to 0.30 for timber–steel interfaces [43] and 0.15 for steel–steel interfaces [44]. The bolt pretension was applied using the built-in bolt load function in ABAQUS. For each steel component, the individual steel plates were first modelled separately and then combined into a single part using the merge operation, thereby avoiding the explicit modelling of welds. Any initial geometric gaps between contacting components were adjusted and eliminated through the contact definition. Particular attention should also be paid to the definition of the analysis steps. The different loading components should be applied in separate steps. Simultaneously applying the bolt pretension, axial compression, and prescribed displacement may lead to convergence difficulties. In addition, the increment size in each analysis step should be carefully controlled to ensure numerical stability and convergence.

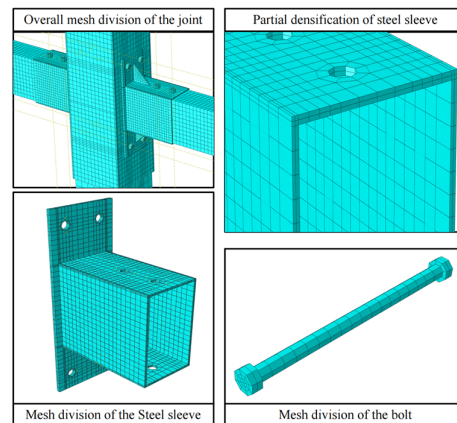


Figure 25. Mesh configurations of several detailed components.

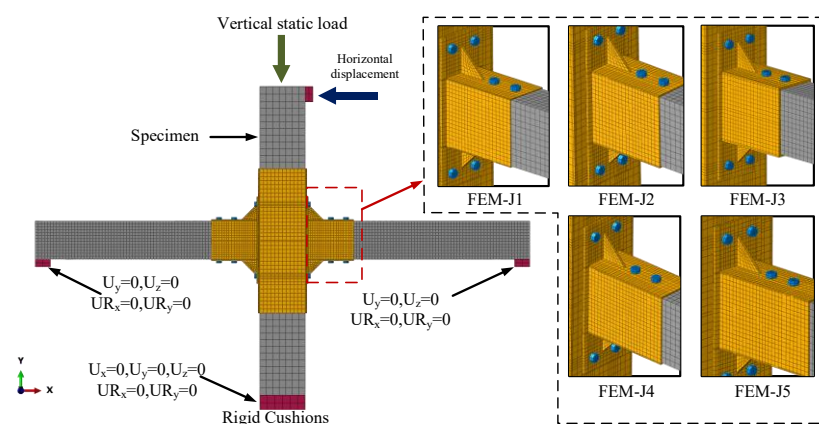


Figure 26. Boundary conditions of the model.

5.3. Finite Element Model Comparison with Experimental Results

5.3.1. Failure Patterns

Figure 27 shows the deformed configurations of three representative FE specimens at failure. For FEM-J1 and FEM-J2, with beam steel-sleeve thicknesses of 4 mm and 6 mm, respectively, the simulated failure patterns agreed well with the experimental observations. Local outward bulging developed in the steel near the connection between the top plate of the beam sleeve and the stiffener, accompanied by pronounced deformation of the beam-sleeve end plate. For FEM-J3, with an 8 mm sleeve thickness, severe deformation of the timber elements developed during the later loading stages, indicating large local timber strain and reproducing the experimental failure characteristics. Compared with those of FEM-J1 and FEM-J2, the deformation of the beam-sleeve end plate in FEM-J3 was substantially reduced.

To further clarify the failure mechanisms, stress contours of the timber and steel components were extracted for all specimens, as shown in Figures 28–30.

Figure 28 presents the longitudinal stress distribution in the timber. For all five specimens, the maximum compressive stress occurred at the interface section between the beam steel sleeve and the timber. The corresponding peak stresses for FEM-J1–FEM-J5 were 39.403 MPa, 46.317 MPa, 49.522 MPa, 47.659 MPa and 46.793 MPa, respectively. As the beam-sleeve thickness increased from 4 mm to 8 mm (FEM-J1–FEM-J3), the peak timber stress increased by 25.68%. In FEM-J3, the maximum compressive stress reached 49.522 MPa, slightly exceeding the measured compressive strength of 48.75 MPa. This result indicates that the beam-end region was susceptible to localized compressive damage, although such damage was not explicitly represented by the simplified constitutive model.

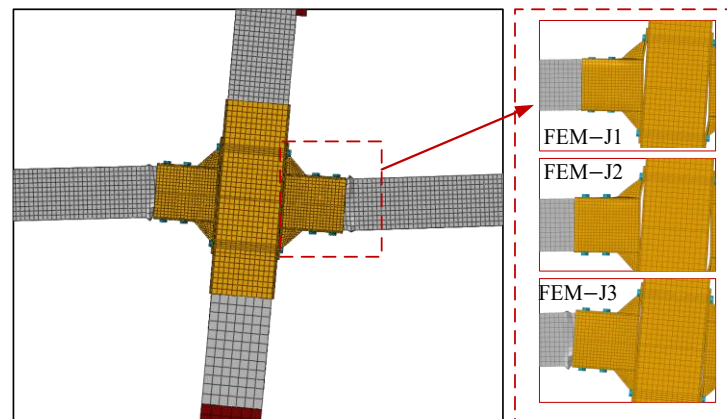


Figure 27. Simulated failure patterns of representative specimens.

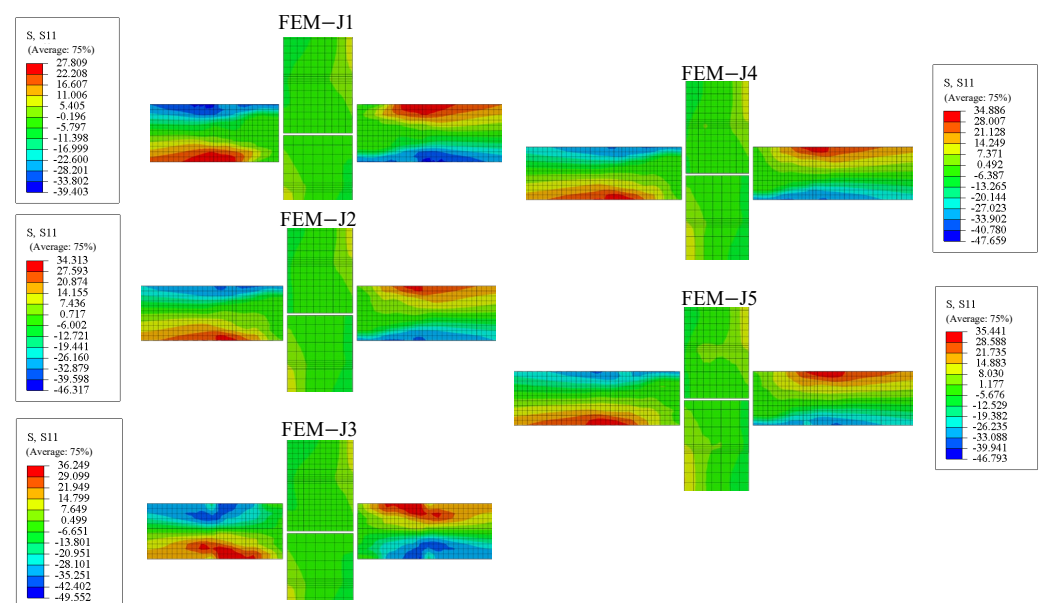


Figure 28. Longitudinal stress distribution in timber.

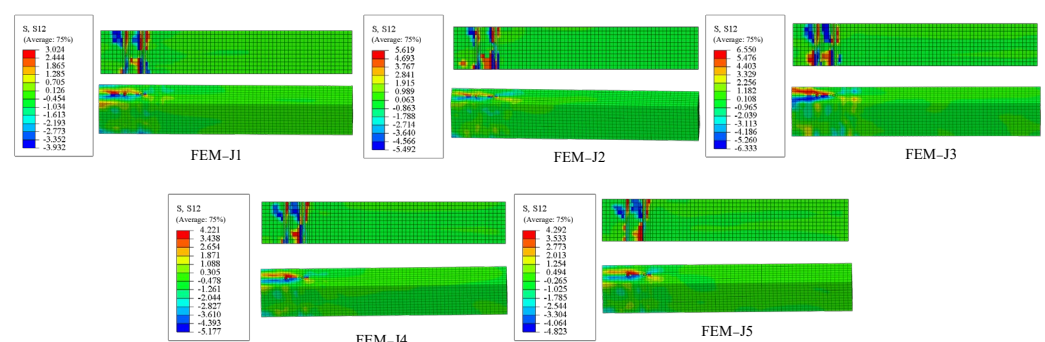


Figure 29. Shear stress distribution in timber.

Figure 29 shows the shear stress of the timber. All five specimens exhibited pronounced shear stress concentrations around the bolt-hole sections. The maximum shear stress in FEM-J3 reached 6.55 MPa, exceeding the measured tangential shear strength parallel to the grain (5.4 MPa). This result indicates the potential for timber shear failure and agrees with the theoretical shear-capacity calculation for specimen J3 presented in Section 4.4. The bolt-hole section at the steel sleeve–timber interface should therefore be regarded as a critical region for shear failure in design.

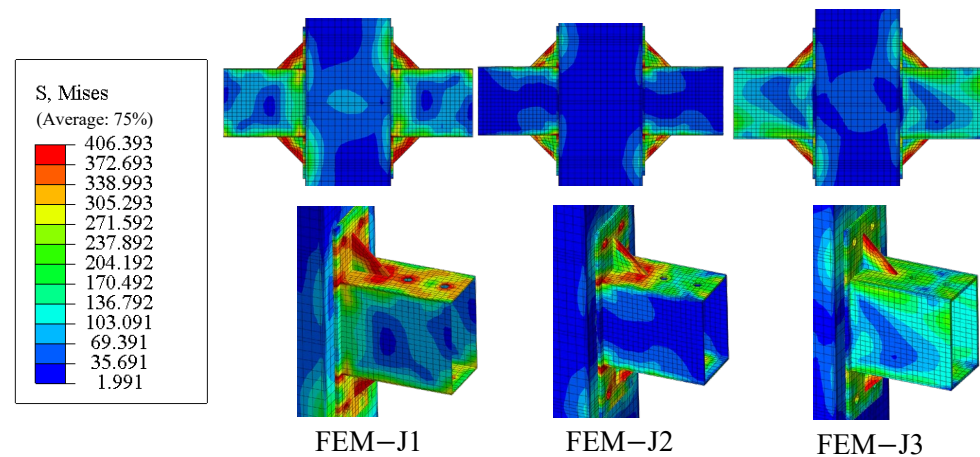


Figure 30. Mises stress of beam steel sleeves.

Figure 30 presents the von Mises stress of the beam steel sleeves. In all specimens, the stresses in the stiffeners exceeded the yield strength of Q235 steel (235 MPa), supporting the intended role of the stiffeners as the primary yielding and energy-dissipating components in the proposed steel–timber sleeve joint. In FEM-J1, yielding also occurred in the top cover plate of the beam sleeve near its connection with the stiffener; this local yielding was suppressed as the sleeve thickness increased. Overall, the stress distributions indicate that increasing the sleeve thickness shifts the critical response from pronounced steel-sleeve deformation towards localized timber compression and shear; however, the stiffeners remain the primary yielding components.

5.3.2. Bearing Capacity

Figure 31 compares the simulated and experimental hysteresis curves, and Table 7 summarizes the corresponding peak load-carrying capacities.

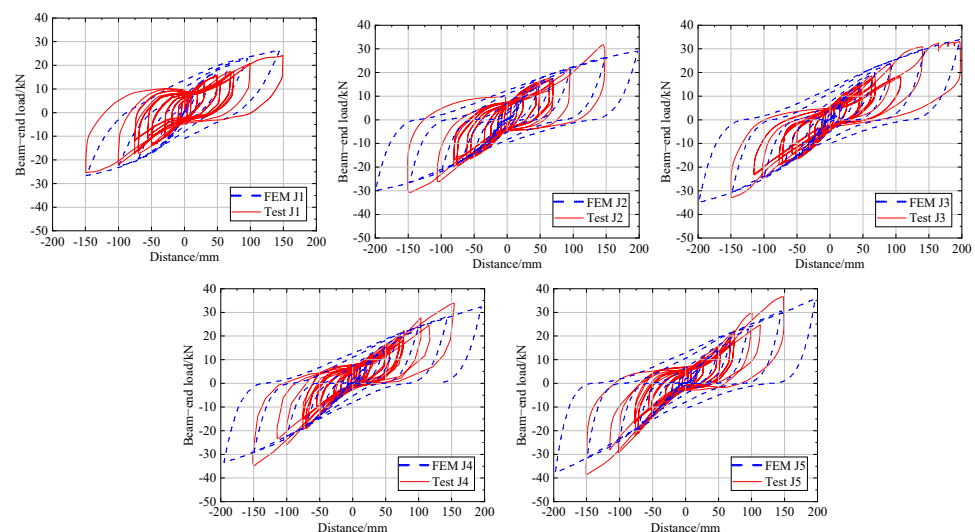


Figure 31. Comparison of simulated and experimental hysteresis curves.

The simplified finite element model focuses on the global cyclic response of the connection and does not aim to simulate detailed crack propagation or fracture processes in timber. Therefore, the comparison between numerical and experimental results mainly considers the load-carrying characteristics.

Table 7. Comparison of bearing capacity between simulation and test.

Specimen	Load Direction	Simulated Result (kN)	Average (kN)	Tested Result (kN)	Average (kN)	Error Value (%)
J1	Positive	25.53	26.13	24.0	24.75	6
	Negative	26.72		25.5		
J2	Positive	29.04	29.53	31.5	30.80	4
	Negative	30.03		30.1		
J3	Positive	33.95	34.47	34.5	34.00	2
	Negative	34.99		33.5		
J4	Positive	32.33	32.84	33.5	34.05	3
	Negative	33.36		34.6		
J5	Positive	35.71	36.89	37.0	37.60	2
	Negative	38.08		38.2		

As presented in Figure 27 and Table 7, the simulated hysteresis curves reproduce the overall shape and peak resistance of the experimental responses reasonably well, with errors in the average peak load ranging from 2% to 6% and remaining below 10% for all specimens. The experimental hysteresis loops nevertheless exhibit more pronounced pinching than the numerical predictions. This discrepancy is mainly attributable to simplifications in the timber constitutive model: interface slip between timber and steel was considered, whereas timber damage evolution and time-dependent creep and stress-relaxation effects were neglected.

6. Discussions

Several limitations of the present study should be acknowledged, and these limitations also indicate directions for future research:

(1) It should be noted that the ultimate displacement adopted in this study was defined based on the experimental termination criterion rather than the conventional 85% post-peak strength degradation criterion. Since the tested specimens did not exhibit a clear descending branch before reaching the displacement limit, this definition provides a consistent test-defined index for comparison among the investigated specimens, rather than a measure of their actual ultimate ductility capacity.

(2) The proposed analytical model is a simplified resistance estimation approach based on the failure mechanisms observed in the present experiments. The model does not consider all possible nonlinear interactions, such as local stress concentration around bolt holes and the coupling effects between bending and shear. Therefore, further validation using a broader range of specimens is required before applying the model for design purposes.

(3) It should be noted that the bending strength of timber was not directly measured in this study. Therefore, the measured mean parallel-to-grain tensile strength was adopted as a simplified substitute parameter for estimating the bending resistance. Accordingly, the calculated M_{rd3} represents an approximate resistance estimate based on a substitute material parameter and should not be interpreted as a conventional code-based bending resistance.

(4) The FE model adopted in this study is a simplified representation of the SCPST joint. Although the model can capture the overall hysteretic response and deformation characteristics, timber crack initiation and damage evolution were not explicitly simulated. Therefore, the predicted failure patterns should be interpreted as stress concentration and deformation trends rather than exact crack propagation processes.

(5) Although the experimental and numerical results demonstrate the feasibility of the proposed SCPST joint, several limitations should be acknowledged. First, the experimental investigation was conducted with different sleeve configurations, and repeated specimens were not included due to the complexity and high cost of full-scale beam–column joint testing. Therefore, the observed parameter effects should be interpreted as trends within the investigated specimen matrix rather than universal quantitative relationships.

(6) The proposed analytical model provides a simplified estimation of the resistance of the SCPST joint based on the observed failure mechanisms. Several assumptions were introduced, including the use of measured tensile strength as a substitute parameter for timber bending resistance, neglecting bending–shear interaction and bolt-hole net-section effects, and applying the equivalent T-stub concept to the sleeve end-plate mechanism. These simplifications may influence the predicted resistance and should be further investigated through additional experimental studies.

(7) It should be noted that the present study was conducted based on a limited experimental matrix. The investigated parameters were restricted to a specific range of sleeve thicknesses (4–8 mm), sleeve lengths (300–400 mm), timber material properties, axial compression ratio (approximately 0.08), and connection geometry. Therefore, the observed mechanical behaviour and the proposed analytical expressions should be interpreted within the investigated parameter range.

(8) The reference displacement of 100 mm adopted in the CUREE-type loading protocol was selected based on previous studies of timber and steel–timber connections and the relatively large deformation characteristics of timber structures. However, this value was not derived from a specific prototype drift demand or a comprehensive parametric study of loading protocols. Therefore, the influence of different reference displacements and loading histories on the cyclic response of the SCPST joint requires further investigation.

(9) It should be noted that the present FE model was evaluated primarily through comparison with the experimental global load–displacement response and peak resistance. The initial stiffness, stiffness degradation and energy dissipation capacity were not quantitatively evaluated in the current study. Therefore, the model should be regarded as a simplified tool for capturing the global response and load-transfer mechanism of the SCPST joint. Further assessment against comprehensive cyclic-response indicators, including initial stiffness, stiffness degradation, and energy dissipation, is required before the model can be used for more detailed prediction of cyclic seismic response.

7. Conclusions

This study investigated the seismic performance of a sleeve-connected prefabricated steel–timber (SCPST) beam–column joint through full-scale cyclic tests, analytical resistance calculations, and finite element simulations. Based on the investigated specimens and parameter ranges, the following conclusions can be drawn:

(1) Damage and inelastic deformation were concentrated primarily at the beam ends, while the timber columns remained essentially elastic throughout the tests, indicating the intended strong-column–weak-beam response. The principal damage and failure modes included deformation of the beam-sleeve end plates, yielding of the stiffeners, and timber bending or shear failure near the timber–steel interfaces.

(2) The SCPST joints exhibited pinched hysteretic responses, mainly because of relative slip at the timber–steel interfaces. Within the tested specimen matrix, the peak load of J3 with an 8 mm beam-sleeve thickness was 43.75% higher than that of J1 with a 4 mm sleeve thickness, whereas the peak load of J5 with a 400 mm sleeve length was 17.46% higher than that of J2 with a 300 mm sleeve length. The tested specimens with thicker or longer sleeves exhibited higher test-defined ductility ratios; however, these ratios are

comparative indices based on the prescribed displacement-based termination criterion rather than measurements of the ultimate ductility capacity. Energy dissipation was governed primarily by frictional sliding at the timber–steel interfaces and yielding of the steel stiffeners.

(3) Simplified analytical expressions were developed for the critical beam-end failure modes associated with sleeve end-plate deformation, stiffener yielding, and timber failure. For the five tested specimens, the calculated resistances differed from the experimental capacities by less than 10%, and the governing analytical failure modes were generally consistent with the experimental observations. The proposed expressions therefore provide a preliminary approach for estimating the resistance of SCPST joints within the investigated parameter range; additional experimental validation is required before their broader application in design.

(4) Comparison with the experimental results showed that the simplified ABAQUS model reproduced the overall deformation patterns, global load–displacement response, and peak resistance of the tested joints reasonably well, with peak-load differences of approximately 2–6%. However, timber cracking and damage evolution were not explicitly simulated, and the experimentally observed pinching was reproduced only approximately. Therefore, the present numerical assessment verifies primarily the global response and peak resistance within the tested configurations and should not be interpreted as comprehensive validation of the detailed cyclic seismic response.

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Conflicts of Interest: Author Leiming Wang was employed by the company Jiangsu Ennas Heavy Industry Technology Co., Ltd. The remaining authors declare that the research was conducted in the absence of any commercial or financial relationships that could be construed as a potential conflict of interest.

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