

Experimental Evaluation of a Novel Reversible Steel-Timber Shear Connection

Alper Turgut^{1*}, Hervé Degée¹, Jean-François Demonceau², José Henriques¹

¹Construction Engineering Research Group, University Hasselt, Belgium

²Department ArGEnCo, University of Liège, Belgium

* Corresponding author: alper.turgut@uhasselt.be

Abstract

Driven by increasing environmental demands within the construction sector, steel–timber composite structures have gained considerable attention as sustainable load-bearing alternatives. Most existing solutions rely on dowel-type fasteners, which often induce premature timber damage during early loading stages, thereby limiting mechanical performance and disassembly and reuse potential. This study introduces a novel shear connection concept aimed at increasing the yield resistance of timber, thereby promoting disassembly and reuse by reducing damage during the loading phase. The experimental programme consisted of three distinct test series: (i) isolated local timber crushing tests to examine failure mechanisms and assess the performance of existing analytical formulations; (ii) push-out tests on two configurations of the proposed steel block connection; and (iii) push-out tests on conventional bolted connections for sake of comparison. The experimental results were subsequently reproduced using analytical formulations and detailed numerical models. Subsequently, a small parametric study was conducted to identify the critical components governing the connection response. The results demonstrate that the proposed steel block connection significantly improves yield resistance and elastic stiffness compared to conventional bolted systems. By enhancing the yield resistance and thereby delaying the onset of

irreversible damage to higher load levels, the connection offers improved suitability for reuse and disassembly, which is fully aligned with contemporary circular construction objectives. The study also shows that, although Eurocode-based timber crushing models yield reasonable predictions for isolated crushing behaviour, more advanced formulations are required to capture the connection response with greater accuracy.

1 Introduction

Growing demands to reduce the environmental impact of the construction industry have accelerated interest in integrating timber into structural systems, particularly through composite applications [1]. Within this context, steel–timber composite (STC) floors have emerged as a promising alternative to conventional steel–concrete systems, where the concrete slab is replaced by a timber panel, commonly cross-laminated timber (CLT) [2]. The performance of such systems is governed primarily by the shear connectors that enable composite action between the steel beam and the timber panel. When the connection is insufficient, the two components respond largely independently, and the structural benefits of composite action are lost. Consequently, current research efforts in STC systems are increasingly centered on understanding and improving connection behavior.

Numerous types of shear connectors have been investigated in steel–timber composite (STC) systems, with dowel-type connectors (particularly bolts and self-tapping or coach screws) being among the most commonly employed. These connection types have been widely characterized through push-out tests [3, 4], composite beam bending tests [5, 6], cyclic loading experiments [7, 8], and studies on long-term behavior [9]. Comparisons between bolts and screws have shown that screwed connectors generally exhibit higher elastic stiffness due to full-thread engagement, but tend to lose stiffness more rapidly as slip progresses under sustained or cyclic loads [10]. In addition

to conventional bolts and screws, other dowel-type connector types such as cast-in shear studs have also been examined. Comparative studies found that while these studs provide acceptable performance, they tend to yield earlier than fully threaded screws, particularly due to cracking of the surrounding grout, and are generally less favorable in terms of ease of installation. [11].

Beyond individual fastener types, several studies have explored hybrid or enhanced connector arrangements, such as angled screw configurations, screws or bolts embedded in hollow steel pipes, epoxy pockets, or combinations involving welded steel plate interfaces [12, 13]. These configurations are designed to improve load transfer efficiency or promote prefabrication and demountability.

While dowel-type connectors are effective in activating composite action and providing a ductile failure mechanism, primarily due to fastener embedment and plastic hinge formation in the timber, they often fall short when reusability or disassembly is required. This is mainly because the timber surrounding embedded fasteners is irreversibly damaged during yielding, which directly influences the condition of the timber element, a key quantitative metric used in reuse-oriented assessment frameworks [14, 15]. To address this limitation, recent work has proposed demountable shear connectors, where bolts are combined with steel tubes and screws [16], or embedded in grout pockets to distribute stress and limit timber damage [17, 18].

Despite numerous connector solutions reported in the literature, a central limitation persists: most dowel-type and related connectors rely on small bearing surfaces, which cause early embedment damage in the timber. This damage develops even at service load levels and directly reduces the suitability of such systems for reuse-oriented design, where the condition of the main timber element is a key metric. One notable exception is the grout-pocket embedded bolt system, which also shows superior yield resistance and initial stiffness [17]. However, the practical challenges associated with casting grout on-site and the need to drill through the entire thickness of the timber

panel reduce its applicability in modular or reversible systems, as it compromises the opposite face of the panel and limits future reuse. These limitations highlight the need for alternative connector concepts that enlarge the effective steel–timber contact area while remaining fully demountable, ensuring reversibility, and minimizing damage to the timber panel.

To address the limitations of existing connections in STC systems, this study introduces a new reversible connection concept that combines superior mechanical performance with improved disassembly and reuse potential. The connector is designed to provide a more stable and efficient load transfer mechanism, enabling higher yield resistance and elastic stiffness than dowel type solutions. In addition, its detailing avoids fastener embedment and allows straightforward detachment, which supports the general principles of disassembly [19]. By remaining largely elastic up to significantly higher load levels, the connection also limits irreversible deformation in both the timber and the steel components, thereby promoting reuse.

The proposed solution does not rely on cast-in or wet construction processes. It consists of a mechanical interlock achieved through a steel block connector that is bolted to the steel part (e.g. upper flange of the steel beam) and screwed into the CLT panel. By providing a substantially larger steel–timber contact surface than dowel-type connectors, the steel block concept distributes bearing stresses more efficiently and delays the onset of local timber damage.

To characterize the mechanical behavior of the proposed steel block connection, the study first focused on the local timber crushing component. This component governs the behavior of the connection, and dedicated tests were therefore performed to characterize its stiffness and resistance and to provide a basis for evaluating the predictive capabilities of both the analytical and numerical models. Subsequently, full connection-level tests were carried out to examine the structural response of the proposed system. Finally, benchmark tests were conducted on a conventional bolted

connection, using the same fixation layout on the steel beam flange (two M16 bolt holes), to allow a consistent mechanical comparison with the proposed system.

To complement the experimental investigation, a finite element model was developed and validated against the test results, providing additional insight into the deformation and stress-transfer mechanisms observed in the tests. In parallel, analytical formulations from the relevant design standards were evaluated together with the numerical and experimental outcomes to determine the extent to which both analytical and numerical tools are applicable for future use in global design procedures. Based on the validated modelling framework, a parametric investigation was subsequently performed to identify the critical components governing the mechanical response of the connection.

This study contributes to the development of demountable steel–timber composite connections by introducing and validating a connection concept that combines enhanced structural performance with potential for disassembly and reuse.

2 Description of the connection

The development of the proposed connector was guided by three fundamental design motivations: (i) achieving superior mechanical performance, (ii) enabling efficient and reliable disassembly, and (iii) ensuring the potential for reuse of the structural components. Among these, mechanical performance constitutes the primary governing requirement. Without sufficient resistance and stiffness, the structural function of the connection would be compromised, and both disassembly and reuse would lose relevance due to early and irreversible deterioration of the timber or connection elements. For this reason, the connector geometry was shaped to maximize elastic load transfer and stiffness before integrating additional disassembly and reuse oriented design considerations.

The need for an improved mechanical response becomes evident when considering the behavior of conventional dowel type connectors. These systems often cause significant local crushing damage to the timber, even under relatively low loads [20, 21]. The onset of this early damage leads to a pronounced reduction in stiffness and a limited yield resistance range. To overcome these limitations, a steel block connector has been developed to enlarge the contact area between steel and timber, thereby enhancing the timber's crushing resistance and stiffness [22]. The concept draws inspiration from the notched connections employed in timber–concrete composite (TCC) systems, which are recognized for their high stiffness and resistance due to larger contact area [23]. In the proposed design, two primary load-transfer interfaces are defined. The steel part is attached to the connector using short bolts threaded into the connector's holes, while the connector itself is embedded into a locally carved groove in the timber panel (Figure 1). The load applied to the steel part is initially transferred to the connector through bolt bearing and shear, and is then transmitted to the timber via the compression stresses developed along the groove peripheral surface. To prevent uplift, the connector is additionally anchored to the timber panel using screws.

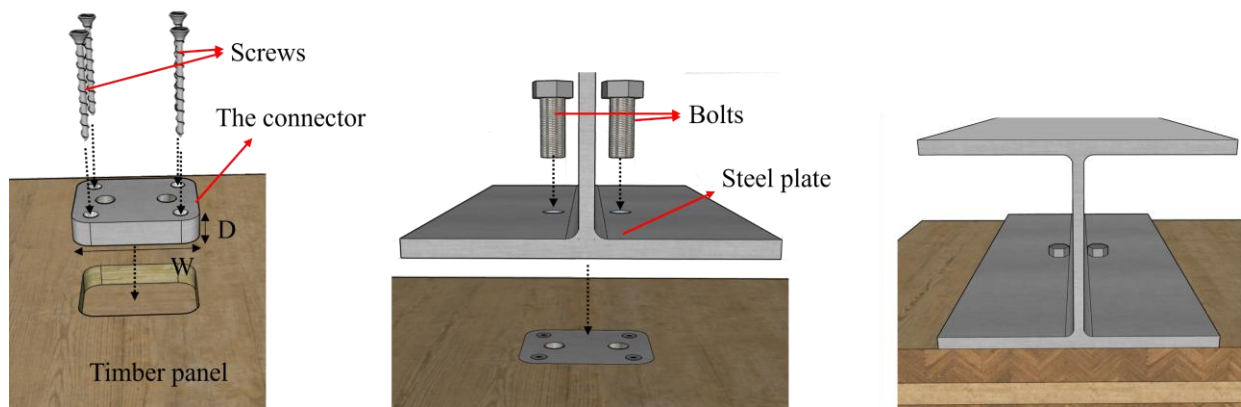


Figure 1 Assembly of the proposed steel block connection.

*A patent application related to this connector is currently pending.

Beyond mechanical performance, the connector was also shaped by Design for Disassembly (DfD) considerations. DfD frameworks emphasise metrics such as time, tools, fixture requirements,

accessibility, instruction complexity, hazard and force demands [19]. In line with these criteria, the proposed connector provides a clear operational advantage: all fastening and release operations can be performed from a single accessible face of the floor system. This single-sided configuration scores favourably in both the time and access metrics, as it avoids the two-side access typically required in through-bolted systems, which increases assembly–disassembly duration and complicates site accessibility. The absence of nuts or similar components also enables one-handed assembly and disassembly, reducing fixture complexity within DfD scoring logic. Although empirical timing measurements were not within the scope of this mechanically oriented study, these qualitative DfD criteria substantially influenced the configuration of the connector.

From a mechanical perspective, two aspects of the connection further support disassembly-focused applications. By increasing the elastic resistance of the connection, the proposed detailing limits the formation of irreversible damage prior to disassembly. This directly addresses one of the key mechanical challenges for disassembly identified by Ottenhaus et al. (2023) [14], namely the occurrence of early embedment and fastener yielding. Secondly, the high mechanical performance achieved by the connector has the potential to reduce the number of connectors required at the structural system level, which could contribute to lower assembly and disassembly effort.

While DfD-oriented detailing facilitates disassembly, it does not inherently ensure reusability. Reuse requires that all structural components remain within their elastic range throughout service life and during the disassembly process. Ottenhaus et al. (2023) [14] identify two primary mechanisms that limit reuse potential in timber joints: (i) irreversible deformation of the fastener hole and (ii) plastic deformation of the steel fastener. These forms of permanent damage typically arise when local stresses exceed the elastic limits of either the timber or the steel components. In contrast to dowel-type connections where such mechanisms may develop at relatively low load levels, the proposed connector was intentionally designed so that both the surrounding timber and

the steel fasteners remain elastic up to significantly higher load levels. Maintaining this elastic response prevents permanent hole deformation and avoids fastener yielding, thereby enhancing the overall potential for reuse after disassembly.

3 Experimental work

3.1 Details of the test specimens

The experimental program consisted of two stages to characterize (i) the local load-transfer mechanism and (ii) the global behavior of the proposed steel block connection. To assess the applicability of existing analytical expressions for local timber crushing, a series of crushing tests were performed using a rigid steel bar with four different contact areas. Subsequently, the proposed steel block connection was tested in two configurations, with and without screws, through push-out tests. Finally, a conventional bolted connection was tested to serve as a benchmark, corresponding to the case where two holes of identical size are drilled through the beam flange. The schematic representation of proposed and conventional connections are given in Figure 2.

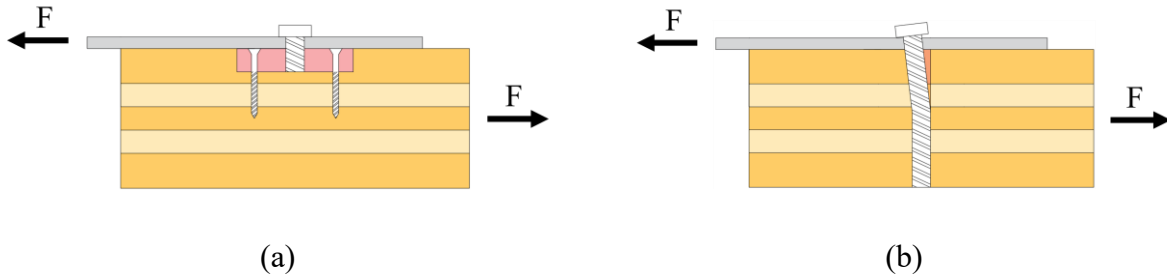


Figure 2 Illustration of the tested connection concepts; (a) proposed steel block connection, (b) conventional bolted connection.

The complete test matrix and specimen designations are provided in Table 1. Specimen labels indicate the connection type and configuration: TC refers to timber-crushing specimens, where the groove dimensions are expressed as $W \times D$ in millimeters (W : crushing width, D : groove depth

(Figure 4)); SBC denotes the proposed steel block connection (B: bolts only, BS: bolts + screws); and CB-REF corresponds to the reference conventional bolted connection. For the steel profiles used in the push-out specimens, the flange thickness (t_f) relevant to bolt bearing is also reported in Table 1.

The timber used in all tests was a five-layer cross-laminated timber (CLT) panel with a layup of 30-20-20-20-30 mm, giving a total thickness of 120 mm. CLT was selected to reflect the panel typologies most commonly adopted in contemporary STC flooring applications [2]. In all specimens, the outer lamella (first layer) was oriented parallel to the loading direction, ensuring longitudinal loading with respect to the grain. As the governing response is driven primarily by this outer layer, the resulting observations are expected to be applicable to other timber panel typologies with a similar face-layer orientation.

Table 1 Summary of tested specimens.

	Specimen Name	Connector elements (mm)	Steel Profile
Timber crushing	TC-20x86	Steel bar (D= 20, W= 86)	
	TC-25x86	Steel bar (D= 25, W= 86)	Steel bar
	TC-20x100	Steel bar (D= 20, W= 100)	(S235)
	TC-25x100	Steel bar (D= 25, W= 100)	
Proposed steel block connection	SBC-B	Steel block connector (D= 22, W= 100), short bolts (2x M16x38)	HE 160 B (t_f = 13 mm)
	SBC-BS	Steel block connector (D= 22, W= 100), short bolts (2x M16x38), screws (4x S5x60)	(S235)
Conventional connection	CB-REF	Long bolts (2x M16x150), washers	IPE 300 (t_f = 10.7 mm) (S355)

3.2 Key parameters of the test results

Key mechanical parameters characterizing the behavior were derived from the experimental load–slip curves. Figure 3 shows a representative experimental curve used to demonstrate the parameter identification procedure.

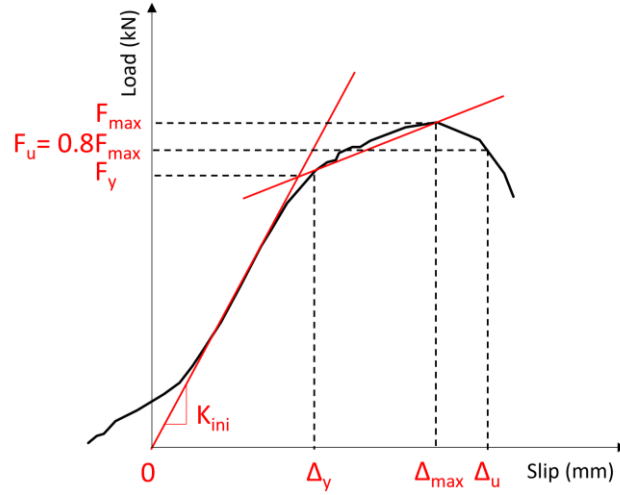


Figure 3 Extraction of key mechanical parameters from the load–slip response.

The stiffness was obtained based on the stabilized linear portion of the curve. To eliminate the non-linear seating effects and low stiffness caused by the test setup at the onset of loading, the linear elastic slope was extrapolated to intersect the horizontal axis. This intersection point served as the corrected origin, effectively removing the initial slip artifact from the ductility calculations. In this study, stiffness refers exclusively to this elastic stiffness; post-elastic stiffness behavior was not evaluated.

The yield point was identified using an equivalent bilinear idealization based on the equal energy principle [24]. In this approach, the area under the idealized bilinear curve is equated to the area under the experimental curve up to the maximum resistance (F_{max}). Consequently, the yield resistance (F_y) and yield slip (Δ_y) were derived from this determined yield point.

For the ultimate limit state, the point where the load dropped by 20% or where specimen failure occurred was selected. The slip value corresponding to this ultimate load (F_u) was designated as the ultimate slip (Δ_u), representing the slip capacity.

Finally, the specimens were classified using the ductility index (μ) proposed by Brühl et al. [25].

The classification criteria employed are as follows:

$$\mu = \frac{\Delta_u - \Delta_y}{\Delta_u}, \quad \begin{cases} \text{Brittle} & \text{if } \mu \leq 0.5 \\ \text{Low} & \text{if } 0.5 < \mu \leq 0.75 \\ \text{Moderate} & \text{if } 0.75 < \mu \leq 0.833 \\ \text{High} & \text{if } \mu > 0.833 \end{cases} \quad (1)$$

3.3 Material properties

3.3.1 Timber

The timber used in this study was classified as C24 grade. To verify its material properties, uniaxial compression tests were carried out on five specimens in accordance with EN 408 [26]. The specimens had a cross-section of 15×120 mm and a height of 90 mm, and the tests were performed to determine both the compressive strength and the modulus of elasticity parallel to the grain. The mean compressive strength and elastic modulus parallel to the grain were measured as 38 MPa and 10.5 GPa, respectively. These findings align with the typical mean characteristics of C24-grade timber [27, 28]. As the objective of these tests was to validate the assigned strength class, and considering that timber properties are interdependent, selectively using the measured values would lead to an inconsistent material definition. Therefore, mean values were consistently adopted for all material parameters in the following sections. Additionally, the oven-dry method was employed to determine the moisture content, which was approximately 10% at the time of testing.

3.3.2 Steel parts

The steel elements used in this study include the steel bar for the timber-crushing tests, the steel profiles for the push-out specimens, the steel block connector, the screws, and the bolts. The steel bar for the timber-crushing tests and the HE 160B profile in the proposed steel block connection were made of S235 steel. In the conventional bolted connection, an IPE 300 section in S355 steel was adopted as the available profile; since the beam flange is not a governing component in the investigated failure mechanisms and its contribution to global stiffness is minor, this difference in steel grade is not expected to affect the comparative response. The screws were manufactured from 4.6-grade steel. The M16 bolts used in the proposed steel block connection were of strength class 12.9 to provide an adequate safety margin for the higher load levels targeted in these tests, while the bolts in the conventional connection were of grade 8.8. This choice was made for test safety and does not modify the resistance hierarchy or the stiffness trends discussed in the following.

3.4 Timber crushing tests

3.4.1 Test setup

The timber crushing tests were conducted on four specimens with different cross-sectional configurations. For each timber specimen, local grooves were first milled into the timber panels. Two panels were then positioned face-to-face, and a steel beam was inserted into the aligned grooves. The timber elements were placed on a rigid base and laterally restrained to prevent any out-of-plane bending during loading. A hydraulic jack was used to apply a compressive force at the midspan of the steel bar, thereby inducing localized crushing in the timber grooves. Four LVDTs were used per timber panel in order to isolate the response of the local crushing component: two positioned beneath the steel bar and two placed near the panel edges. The displacement data were obtained by subtracting the mean displacement of the side LVDTs from the average

displacement of the central LVDTs. This arrangement also allowed the identification of unintended movements during the tests. Rotation was detected when the central steel bar twisted, producing an uneven crushing pattern beneath it. Eccentricity was identified when the crushing displacement differed between the two load transfer sides of the specimen. To capture relative displacement, a steel mounting plate was screwed to the underside of each timber panel, and the LVDTs were fixed to this plate Figure 4a. Three repetitions were conducted for each specimen, except for TC-20×86. Due to an error in the testing system, only two repetitions were completed for this configuration. The loading rate was set to 0.3 mm/min to remain within the limit recommended in EN 408 for compression tests. The complete test setup is illustrated in Figure 4b.

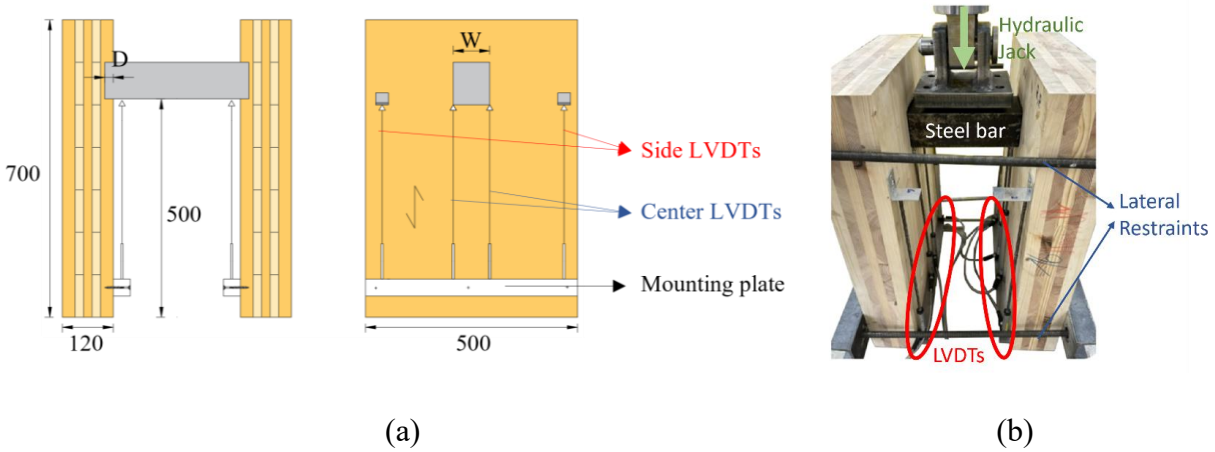


Figure 4 Timber crushing test setup: (a) schematic layout with dimensions and LVDT positions, and (b) complete experimental configuration (dimensions in mm).

3.4.2 Test results

At the beginning of the tests, a low-stiffness response was observed due to the closure of initial gaps in the test setup and the activation of the soft contact behavior of the timber under compression [29]. This initial compliance led to varying levels of initial displacement across specimens. To

enable visual comparison between specimens, the load–slip curves were shifted so that the linear elastic stiffness segments align. This shift was applied solely for visualization purposes and did not affect any calculated results, stiffness values, resistance values, or subsequent analyses.

Figure 5 illustrates the load–slip response measured on one side of the test setup (i.e., per panel).

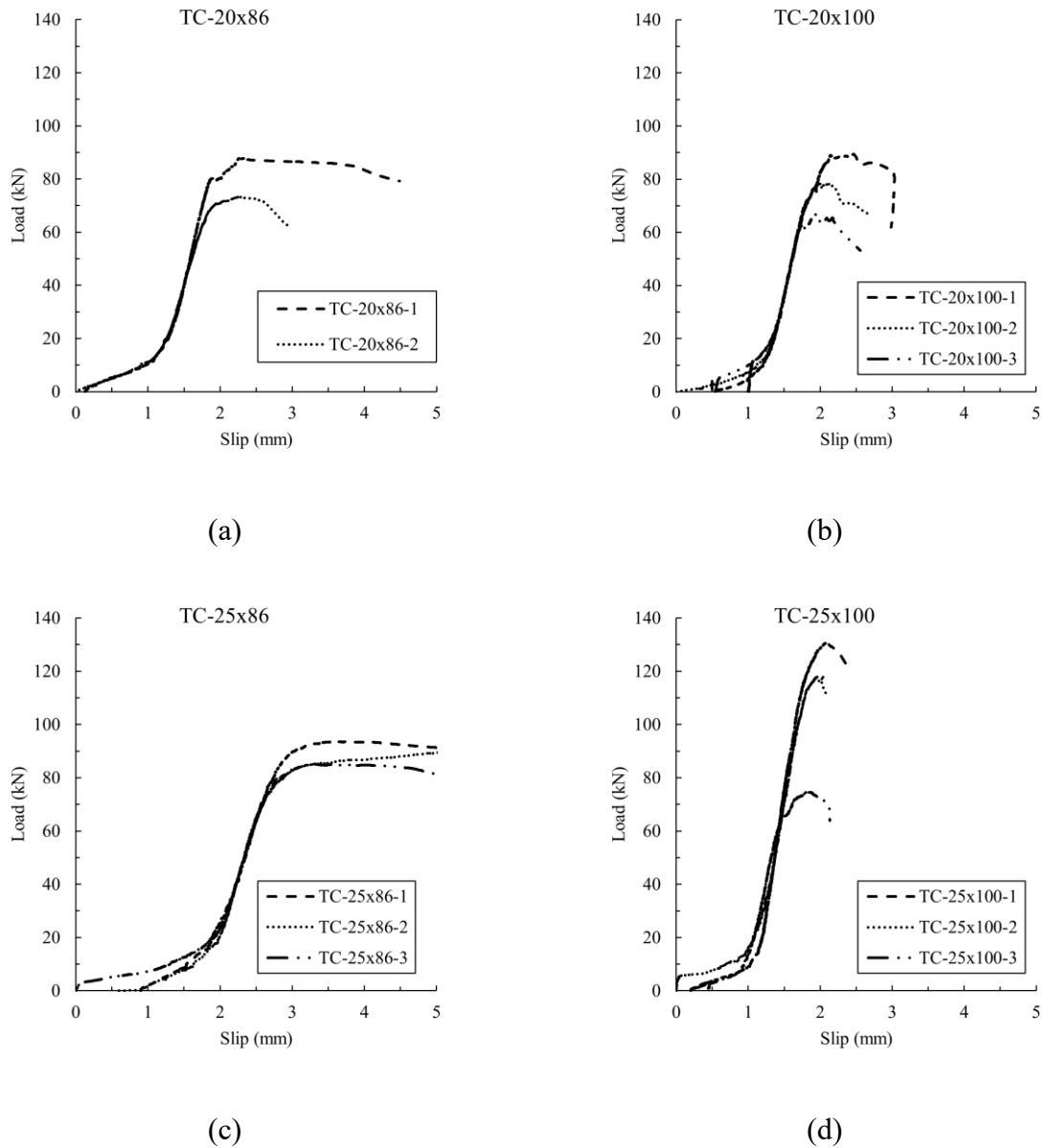


Figure 5 Load–slip responses in the timber crushing tests: (a) TC-20x86, (b) TC-20x100, (c) TC-25x86, (d) TC-25x100.

All specimens failed by timber crushing, although the failure patterns varied depending on the configuration. The maximum resistance, stiffness, and failure mode results of all specimens are summarized in Table 2.

Table 2 Summary of resistance, stiffness, and failure modes in the timber crushing tests.

Specimen	Maximum resistance (kN)	Mean maximum resistance (kN) (%CoV)	Stiffness (kN/mm)	Mean stiffness (kN/mm) (%CoV)	Failure mode
TC-20x86-1	87.7	80.6 (7.1)	115.1	103.7 (11.4)	Uniform crushing + local plug shear
TC-20x86-2	73.4		92.3		Uniform crushing + local plug shear
TC-20x100-1	89.4	78.4 (9.2)	134.3	128.1 (5.5)	Uniform crushing
TC-20x100-2	78.6		121.0		Uniform crushing
TC-20x100-3	67.0		129.0		Uneven crushing
TC-25x86-1	93.5	90.0 (3.5)	80.3	86.6 (5.9)	Uniform crushing
TC-25x86-2	91.2		94.4		Uniform crushing
TC-25x86-3	85.2		85.1		Uniform crushing
TC-25x100-1	130.7	107.8 (24)	197.5	158.8 (31)	Uniform crushing
TC-25x100-2	117.9		158.6		Uniform crushing
TC-25x100-3	74.5		120.3		Uniform crushing

TC-20×86 specimens showed low rotation and limited eccentricity during loading and at failure, with a largely uniform crushing pattern (Figure 6a). Both specimens developed a local plug-shear failure at the end of the test (Figure 6b). In the TC-20×100 series, no notable rotation or eccentricity was observed during loading. At failure, one side of the specimen exhibited more pronounced damage than the other, and post-test inspection showed visible rotation marks in some specimens (e.g., TC-20×100-3) together with uneven crushing of the grooves (Figure 6c). The TC-25×86 specimens did not show visible rotation or eccentricity. In the TC-25×100 series, specimens TC-25×100-1 and TC-25×100-2 exhibited stable behaviour without noticeable rotation. In contrast, specimen TC-25×100-3 showed clear eccentricity during loading, leading to asymmetric crushing, reduced stiffness, and a lower resistance.

The local plug-shear failures observed in the TC-20×86 specimens are consistent with the stress state that develops once the crushing resistance of timber is reached. After localized crushing, the interaction between perpendicular tension and shear becomes more pronounced, especially in regions influenced by natural material imperfections. This combination of stresses can promote a plug-type mechanism, which has also been seen in previous experimental observations of locally compressed timber elements [30]. In the present tests, the crushed zone appears to initiate a local stress concentration that triggers a mixed tension–shear response, ultimately forming a plug-shaped failure region.

Increasing the crushing area generally led to higher maximum resistance across the series, except for TC-20×100, where the asymmetric damage observed at failure limited the achievable resistance.

Stiffness increased with crushing area in all configurations except TC 25×86. Additionally, the stiffness values were more irregular in the 25 mm deep groove specimens. This behaviour is consistent with the fact that deeper grooves are positioned closer to the CLT interlayer, increasing the likelihood of rolling shear deformation and thereby introducing more variability in the stiffness response.



(a)



(b)



(c)

Figure 6 Failure modes of timber specimens: (a) Uniform crushing, (b) Local plug shear (TC-20×86), (c) Uneven crushing.

In addition to these observations, the yield resistance and ductility parameters are also summarized in Table 3.

Table 3 Yield resistance and ductility characteristics of the timber crushing specimens

Specimen	Yield resistance (kN)	Mean yield resistance (kN) (% CoV)	Slip capacity (mm)	Mean slip capacity (mm) (% CoV)	Ductility index	Mean ductility index (% CoV)	Ductility classification
TC-20x86-1	78.8	72.1 (9.3)	11.6	8.1 (44.1)	0.94	0.88 (5.9)	High
TC-20x86-2	65.3		4.5		0.83		High
TC-20x100-1	68.7	64.3 (6.3)	1.8	1.7 (8.3)	0.70	0.66 (4.8)	Low
TC-20x100-2	65.4		1.5		0.63		Low
TC-20x100-3	58.9		1.8		0.64		Low
TC-25x86-1	88.5	82.1 (6.0)	10.5	6.9 (41.4)	0.88	0.81 (8.2)	High
TC-25x86-2	81.2		6.7		0.83		Moderate
TC-25x86-3	76.5		3.5		0.72		Low
TC-25x100-1	107.1	92.8 (20.3)	1.4	1.3 (7.4)	0.57	0.49 (13.1)	Low
TC-25x100-2	105.1		1.2		0.42		Brittle
TC-25x100-3	66.1		1.2		0.47		Brittle

The yield resistance followed the same overall trend as the maximum resistance, increasing with larger crushing areas, except for the TC-20×100 series. In terms of ductility, concluding a clear trend is challenging due to the variability in post-peak behavior among the specimens. Nevertheless, with the exception of the TC-25×86 specimens, larger crushing areas generally led to a more brittle response. The main reason for this behavior is that, while the shear planes remain unchanged, the increasing crushing area subjects them to higher stress levels. The TC-25×86 series behaved as an outlier, exhibiting noticeably higher ductility than the TC-20×100 specimens. The atypical response of the TC-25×86 series, in terms of both stiffness and ductility, may be attributed to several factors, including the natural variability of timber and the potential mobilization of

rolling shear due to the 25 mm groove depth. However, more detailed testing would be required to conclusively determine the underlying cause.

3.5 Connection tests

3.5.1 Test setup

This series comprises experimental testing of three distinct connection configurations: (i) the proposed steel block connection with screws, (ii) the proposed steel block connection without screws, and (iii) a conventional bolted connection. The tests were designed to both characterize the behavior of the proposed steel block connection and evaluate its performance, when installed through the beam flange with the same fixation layout (two M16 bolt holes), relative to a conventional bolted connection.

In the proposed steel block connection tests, the configuration described in Section 2 was used with an HE 160B profile, with 18 mm holes for the M16 bolts. The steel connector has a 100×22 mm cross-section, with two threaded holes for M16 bolts and four countersunk holes for 5 mm screws. The measurement setup adopted for the timber-crushing tests was also used for the proposed connection tests (Figure 7a). The loading rate was 0.3 mm/min. The complete test setup is shown in Figure 7b.

In the conventional bolted connection tests, two M16 grade 8.8 bolts were used to connect an IPE300 steel profile, with 18 mm bolt holes. Four LVDTs were directly attached to the steel profile to measure the relative displacement between the timber and steel members (Figure 7c). The loading protocol followed the specific procedure proposed in ISO6891_1983 [31] for timber connections. The test setup is illustrated in Figure 7d.

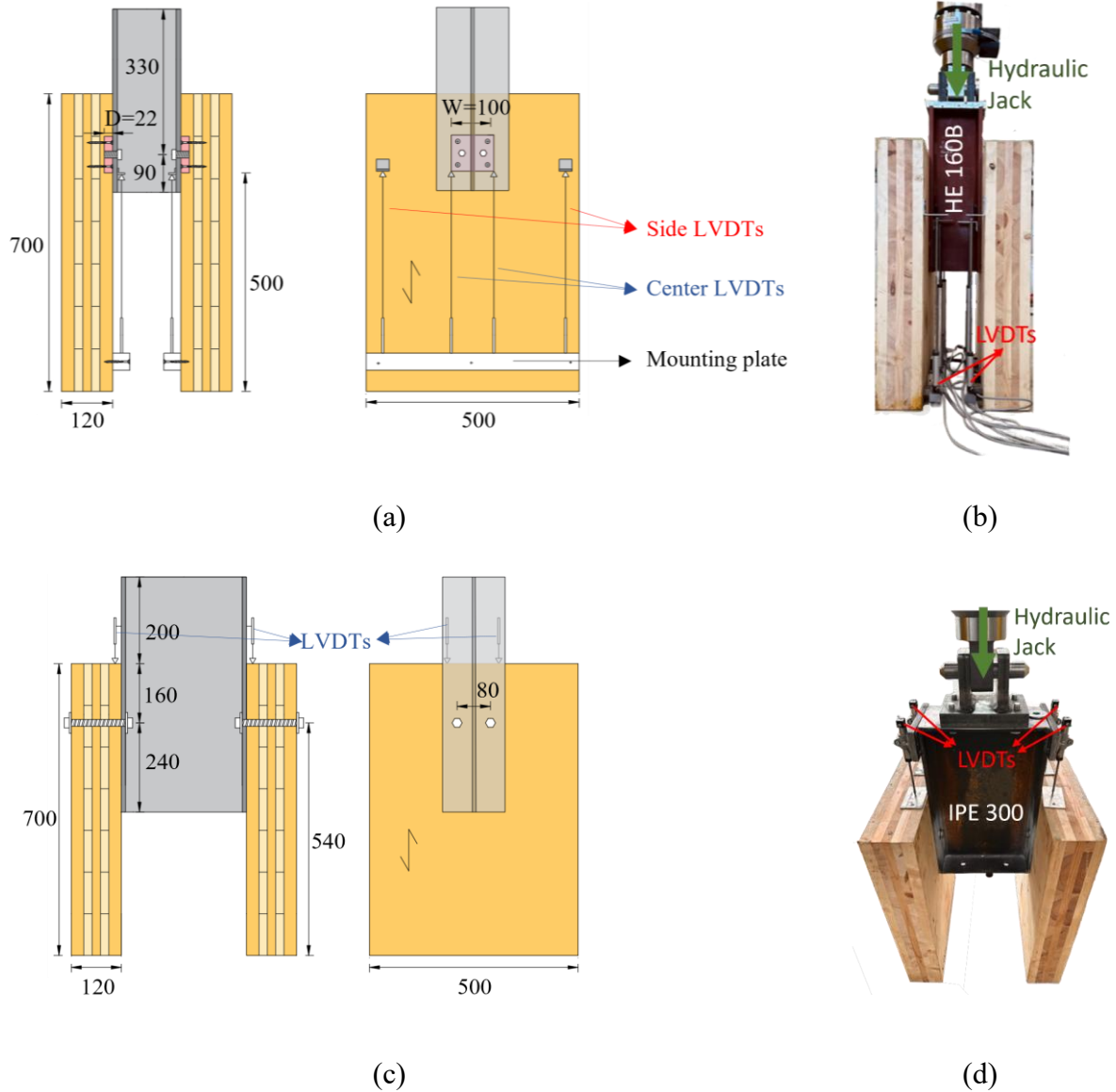


Figure 7 Push-out test setups: (a) details and dimensions of the proposed steel block connection, (b) corresponding test configuration, (c) details and dimensions of the conventional bolted connection, and (d) corresponding test configuration (dimensions in mm).

*A patent application related to this connector is currently pending.

The geometry of the connector, including the bolt-hole spacing, edge distances, and width, was defined in accordance with the minimum detailing requirements of Eurocode 3 Part 1-8 [32]. The dimensions of the steel block connector are shown in Figure 8.

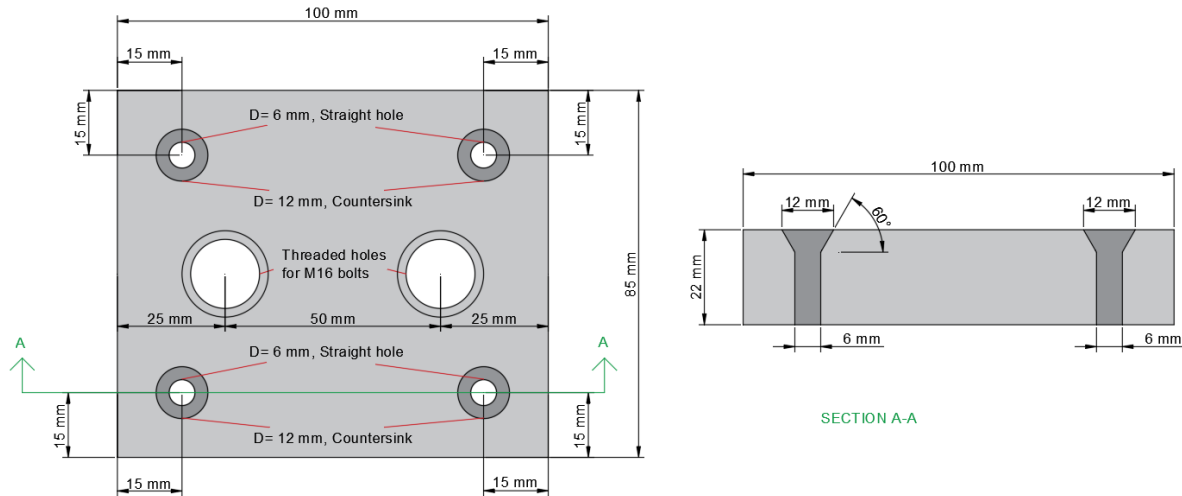


Figure 8 Dimensions of the steel block connector.

Different steel beams were used for the conventional bolted connection and the proposed steel block connection due to component availability during the testing campaign. However, this difference is not expected to influence the comparative interpretation of the results, since the beam flange was not the governing component in either configuration. No influence on the resistance is anticipated, and only a negligible effect on the stiffness may be expected, as the global response was controlled by the connection and the timber interface rather than by the beam related components.

3.5.2 Test results

The load–slip behavior obtained from the connection tests is presented in Figure 9.

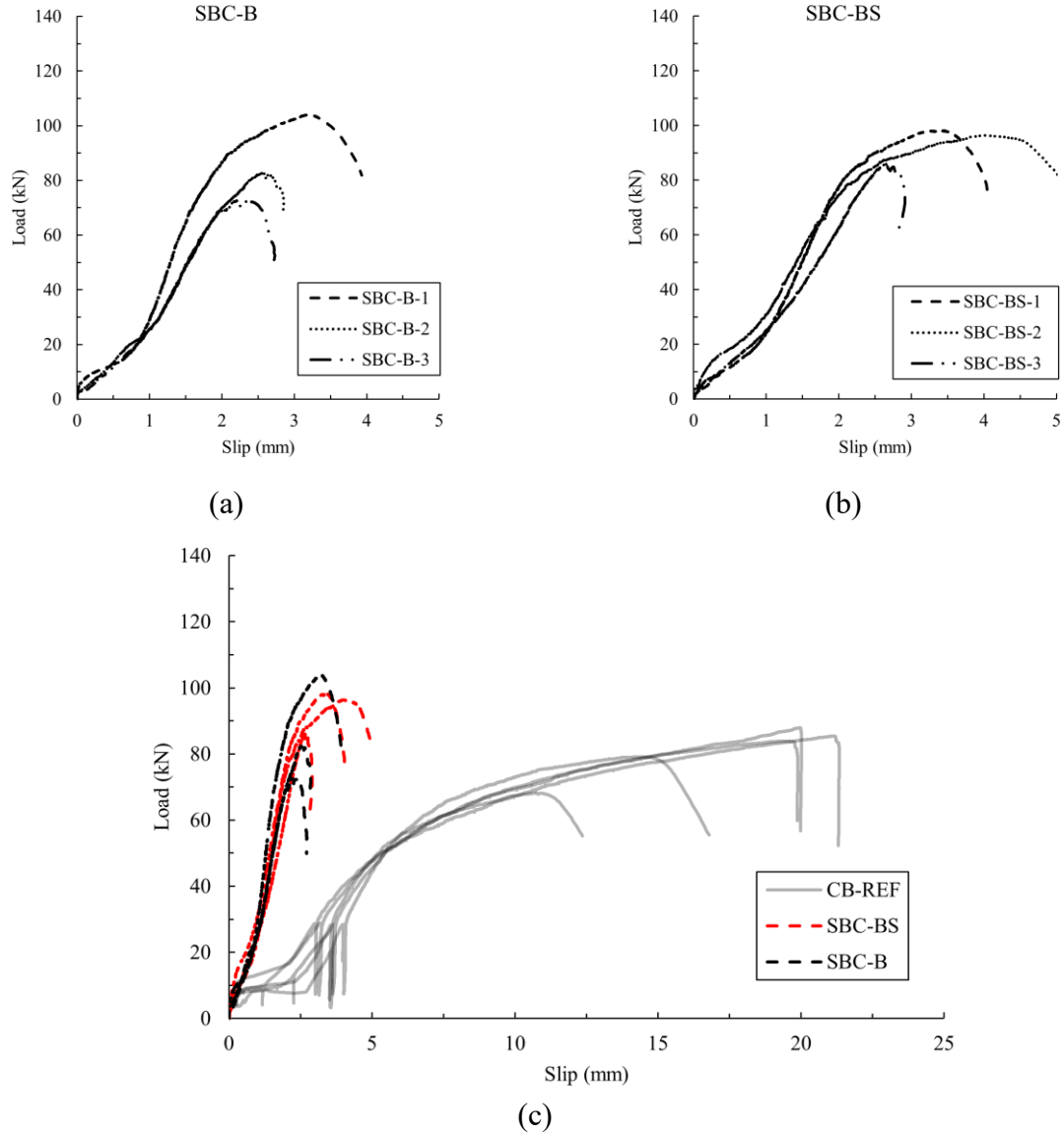


Figure 9 Load–slip responses in the connection tests: (a) without screws (SBC-B), (b) with screws (SBC-BS), (c) comparison of proposed steel block connections with the conventional bolted connection (CB-REF).

Although only minor eccentricity and rotation were observed during loading, all proposed connection specimens exhibited rotation at failure, and the governing failure mode was uneven timber crushing (Figure 10a). No damage was observed in the steel components; however, in all screwed configurations, slight plastic hinging developed in the screws, leading to local embedment damage in the timber (Figure 10b).

According to ISO6891_1983 [31], a maximum slip displacement of 15 mm is defined as the limit state for timber connections, and the corresponding load is taken as the maximum resistance (F_{max}). In the conventional bolted connection tests conducted in this study, the tests were slightly extended beyond this displacement in order to observe the post-embedment response; however, for consistency with the standard, the load at 15 mm slip was adopted as the maximum resistance (F_{max}) in the evaluation. All connection tests failed by bolt embedment (Figure 10c), accompanied by plastic hinge formation in the bolts (Figure 10d) and the activation of the rope effect (Figure 10e).

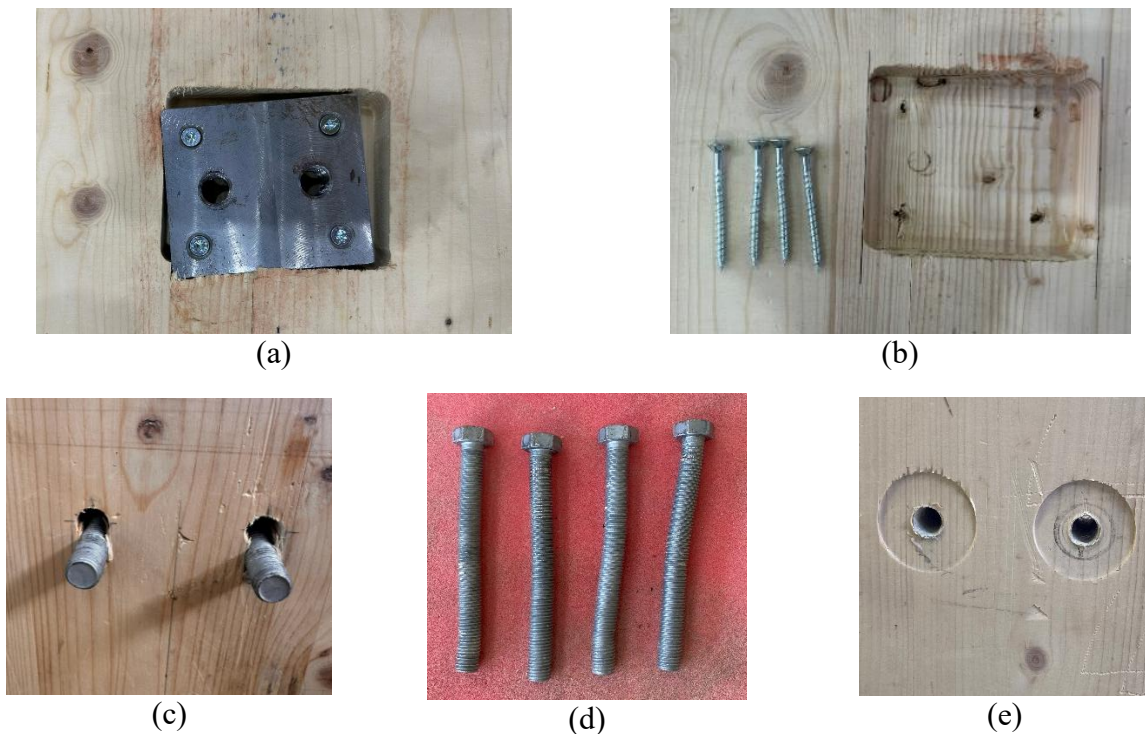


Figure 10 Observed failure modes in the connection tests: (a) uneven crushing of the timber, (b) screw yielding and local crushing, (c) bolt embedment in timber, (d) plastic hinges of the bolts, (e) perpendicular-to-grain crushing associated with rope effect.

*A patent application related to this connector is currently pending.

The maximum resistance, stiffness, and failure mode results of all specimens are summarized in Table 4.

Table 4 Summary of maximum resistance, stiffness, and failure modes in the connection tests.

Specimen	Maximum resistance (kN)	Mean maximum resistance (kN) (%CoV)	Stiffness (kN/mm)	Mean stiffness (kN/mm) (%CoV)	Failure mode
SBC-B-1	104.0		73.2		Uneven crushing + rotation
SBC-B-2	82.8	86.5 (13)	52.8	58.5 (17.9)	Uneven crushing + rotation
SBC-B-3	72.7		49.5		Uneven crushing + rotation
SBC-BS-1	98.2		59.1		Uneven crushing + rotation
SBC-BS-2	96.3	93.5 (5.4)	48.7	50.3 (13.1)	Uneven crushing + rotation
SBC-BS-3	85.9		43.1		Uneven crushing + rotation
CB-REF-1	79.1		17.1		Embedment + rope
CB-REF-2	77.5		12.8		Embedment + rope
CB-REF-3	79.5	78.9 (1.1)	14.4	14.6 (15.6)	Embedment + rope
CB-REF-4	68.1		17.1		Embedment + rope
CB-REF-5	79.6		11.4		Embedment + rope

The proposed steel block connections exhibited slightly higher maximum resistance than the conventional bolted connection. When comparing the two variants of the proposed connector, the configuration with screws achieved on average an 8 percent increase in maximum resistance relative to the screw free configuration. In terms of stiffness, the screw free configuration showed an elastic stiffness almost four times higher than the bolted connection, while the configuration with screws reached a stiffness approximately 3.5 times higher.

In the conventional bolted configuration, a lower yield resistance is observed, primarily due to the early onset of timber embedment and the formation of plastic hinges in the bolts. As loading progresses, the timber begins to crush at relatively low load levels, which limits the initial stiffness and yield capacity of the connection.

In contrast, the conventional configuration exhibits higher ductility, primarily due to the formation of plastic hinges in the bolts. Since the bolts pass entirely through the timber panel and are anchored with washers at the rear face, the rope effect is also activated at larger deformations. This

mechanism contributes mainly to the post-yield increase in load-carrying capacity, while the pronounced deformation capacity itself is governed by the yielding behaviour of the fasteners.

However, the maximum resistance in the conventional bolted configuration is reached at deformation levels associated with significant timber damage and fastener yielding. The resulting local damage around the bolts was observed to make the disassembly and removal of the fasteners considerably more difficult, which may limit the disassembly and reuse potential of the connection components.

The comparatively low stiffness observed in the bolted connection is mainly attributed to the limited contact area involved in its load path, while the early onset of timber crushing leads to a rapid reduction in stiffness as loading progresses. In the proposed steel block connection, the higher stiffness and resistance arise from the more favourable load transfer mechanism. However, the stiffness reduction observed when screws were added to the proposed steel block connection, although not expected theoretically, it is hypothesized that the screws penetrated the interlayer and redistributed the load into the second layer of the CLT panel, thereby reducing the overall stiffness. Further detailed testing is required to confirm this mechanism.

For designs that prioritise disassembly and reuse, the yield resistance is more relevant than the maximum resistance, as it marks the onset of irreversible timber damage. Although bolted connections can maintain load-carrying capacity well beyond the peak due to pronounced post-peak ductile behaviour, this portion of the response is not desirable or design-relevant for reversible systems. For this reason, the yield resistance of all connection types was evaluated and compared separately, and summarized in Table 5 together with ductility parameters. The tests on the conventional bolted specimens were terminated due to instability or manually, so the ductility values reported represent lower-bound estimates.

Table 5 Yield resistance and ductility characteristics of the connection specimens

Specimen	Yield resistance (kN)	Mean yield resistance (kN) (% CoV)	Slip capacity (mm)	Mean slip capacity (mm) (% CoV)	Ductility index	Mean ductility index (% CoV)	Ductility classification
SBC-B-1	67		3.3		0.62		Low
SBC-B-2	81.7	71.7	2.2	2.6	0.41	0.46	Brittle
SBC-B-3	66.4	(9.8)	2.2	(19.7)	0.35	(25.1)	Brittle
SBC-BS-1	83.3		3.4		0.55		Low
SBC-BS-2	79.9	80.6	4.8	3.5	0.63	0.46	Low
SBC-BS-3	78.6	(2.5)	2.3	(28.3)	0.21	(39.5)	Brittle
CB-REF-1	61.9		> 16.3		> 0.67		Low
CB-REF-2	51.2		> 21.3		> 0.76		Moderate
CB-REF-3	54.6	55.5	> 19.9	> 17.9	> 0.72	> 0.7	Low
CB-REF-4	54.9	(6.2)	> 12.4	(18.0)	> 0.62	(6.8)	Low
CB-REF-5	55		> 19.9		> 0.71		Low

The proposed steel block connection exhibited higher yield resistance than the conventional bolted joint, with the configurations with and without screws reaching approximately 45 percent and 30 percent higher yield values, respectively. The bolted system entered inelastic response at relatively low load levels due to early embedment deformation and bolt slip. The steel block connector remained elastic over a larger portion of the load displacement curve. For the proposed configurations, the measured slip capacity was relatively limited, which reduced the deformation capacity.

The higher yield resistance obtained with the steel block connector is particularly relevant for reuse oriented applications, since a larger elastic range reduces damage accumulation during service and improves the condition of the components after disassembly. This allows the connection to be reinstalled with more predictable mechanical performance, which is a key requirement in design approaches that prioritise repeated use of structural elements [14, 31]. However, the current detailing of the proposed steel block connection results in a limited slip capacity, and therefore the ductility is lower than desirable for applications requiring large deformations. This limitation stems

from the adopted hierarchy of components in the tested configuration rather than from the connection concept itself. Higher ductility could be achieved by shifting the governing failure mode toward more ductile steel-controlled mechanisms, such as yielding through beam-flange bearing or the introduction of dedicated steel fuse elements, before timber crushing becomes active. This provides a clear pathway for future refinements in which ductile behaviour is intentionally incorporated without compromising the integrity of the timber panel.

4 Modelling and assessment

In this section, six specimens are modelled numerically and analytically, namely two configurations of the proposed steel block connection and four local timber crushing tests, and the results are compared against the corresponding experiments. The objective is to examine the structural response and associated failure mechanisms in greater detail and to assess the capability and limitations of the numerical and analytical tools in reproducing the observed behavior. On this basis, the suitability of these models for use in global design applications and in subsequent parametric studies is evaluated.

4.1 Numerical modelling

4.1.1 Modelling techniques

All simulations were carried out using the *Abaqus/Explicit* [34] solver under quasi-static conditions, using C3D8R eight-node reduced-integration brick elements, with kinetic energy remaining negligible throughout the analyses (around 1% of the internal energy). Displacement was applied at the hydraulic jack location from the tests, corresponding to the mid-span of the steel bar and the top surface of the steel profile for the component and connection tests, respectively. To reduce computational time, only one-quarter of each test specimen was modeled by applying

symmetry conditions. Timber specimens were fully restrained at their bottom surface, and lateral stabilizers were added to configurations without the screws to match the experimental boundary conditions. In accordance with the experimental procedure, local displacement was calculated by subtracting the displacement of a point on the adjacent timber edge from the displacement of the steel bar or profile at the loading area. (Figure 11).

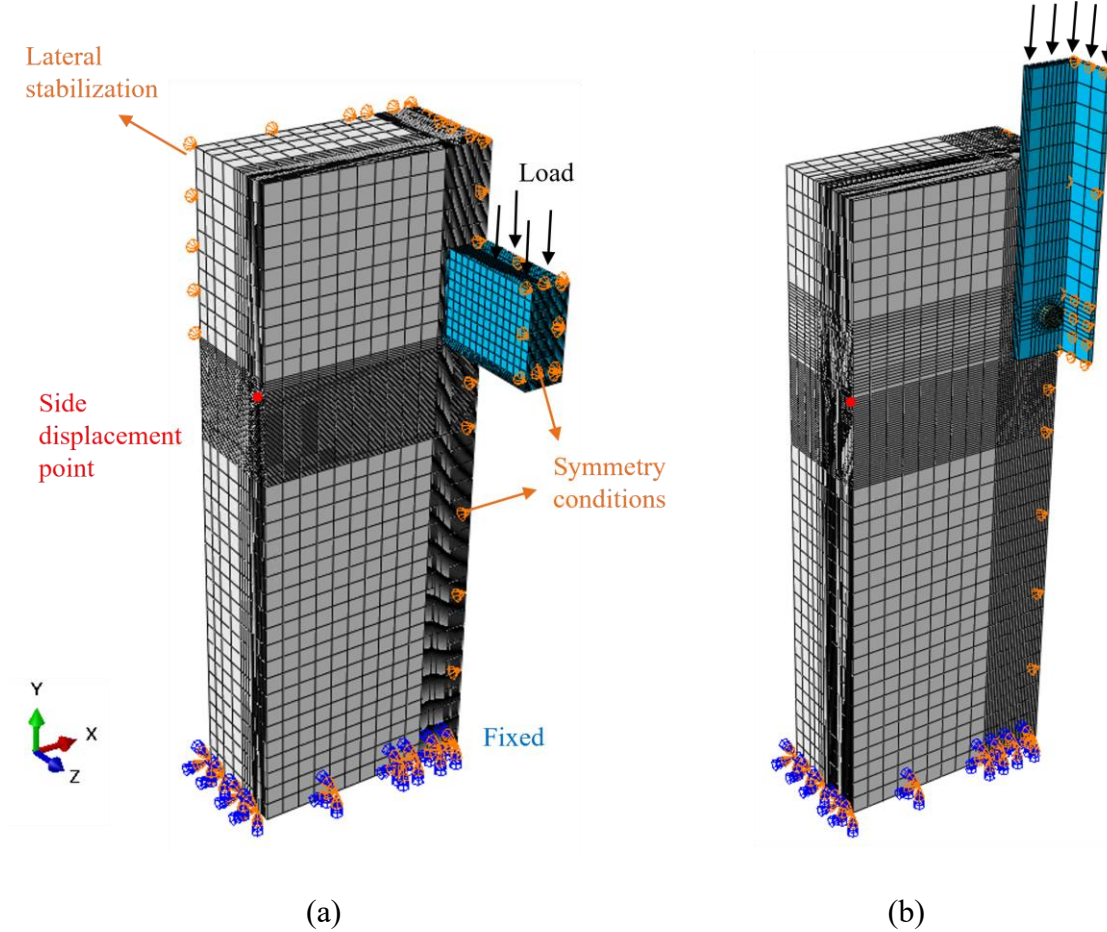


Figure 11 Finite element model for the (a) crushing and (b) connection tests.

Contact interactions were defined according to the mechanics of each interface. Steel–steel contacts were modeled with hard contact in the normal direction and a tangential penalty friction coefficient of 0.4, while steel–timber contacts were assigned a coefficient of 0.25 for tangential friction [29]. The normal behavior of the steel–timber interface depended on the fiber orientation. For

perpendicular-to-grain compression, hard contact was used. For longitudinal compression, however, a standard hard-contact formulation would overestimate the stiffness because drilling or cutting does not sever all fibers uniformly, resulting in an initial soft phase in the bearing response. To capture this effect, the longitudinal normal contact behavior was implemented following the approach of Dorn [33, 34], which employs the Lamé function [29] to represent the compressive response and incorporate localized fiber degradation. The corresponding contact law is given in the following equation.

$$\left(\frac{u_{inter}}{u_0}\right)^{3.9} + \left(\frac{\sigma_0 - \sigma_L}{\sigma_0}\right)^{1.1} = 1 \quad (2)$$

Here, σ_0 represents the compressive strength and was taken as 36 MPa. The parameter u_{inter} denotes the displacement increment at which the contact behavior is defined, and a step size of 0.05 mm was used. The parameter u_0 has been recommended in Hassanieh et al. (2017) [29] for dowel-type connections, with values of 0.45 mm for bolts and 0.35 mm for screws. However, the contact surface in the present study is smooth and does not involve a dowel-type fastener. Therefore, the bolt-based value of 0.45 mm was adopted, as it better reflects the behavior of a smooth steel surface. Further investigation is required to establish a more accurate representation specifically for this type of contact interface. Finally, σ_L denotes the contact pressure corresponding to the displacement increment u_{inter} . This formulation yields a tabular pressure–overclosure curve for use in the soft contact definition.

Threaded engagement between the bolt and the connector was modeled as rough in the tangential direction to represent thread interlock. For screws embedded in timber, a tangential friction coefficient of 0.4 was applied [37]. As the M16 bolts used in the tests were fully threaded, the pitch diameter was assigned in the finite element model to more accurately represent their stiffness contribution [36, 37].

Figure 12 shows the 3D model of the components used in this study.

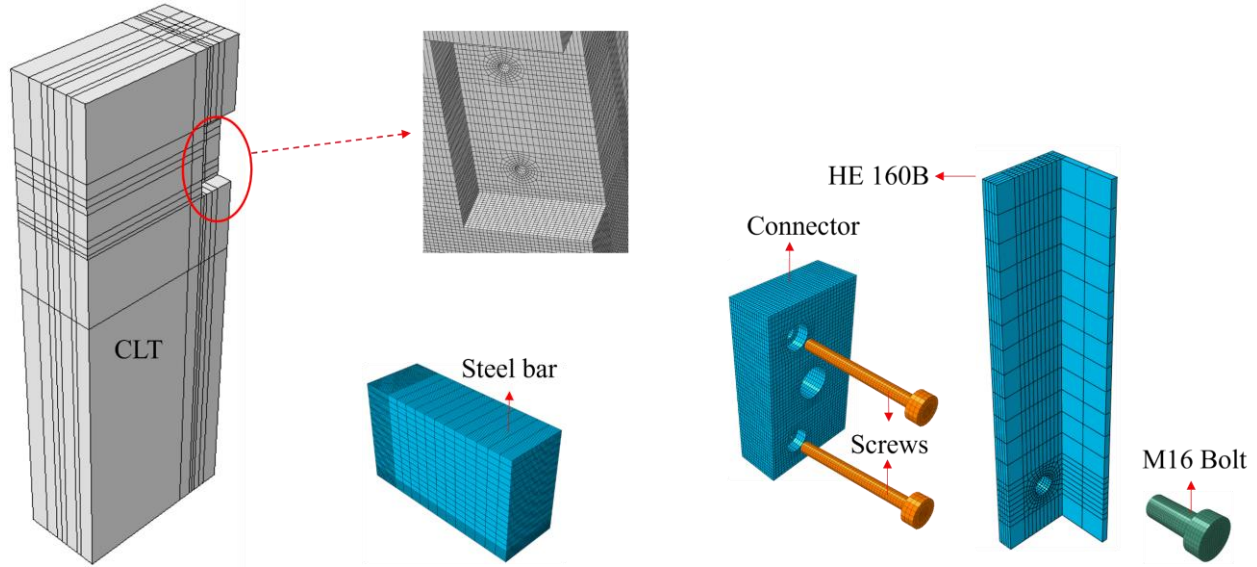


Figure 12 3D FEM models of the components.

*A patent application related to this connector is currently pending.

Steel parts were modeled using isotropic elastic–plastic definitions available in Abaqus, with material parameters consistent with the steel grades reported in Section 3.3.2. Steel components were modelled using the isotropic elastic–plastic material formulation in Abaqus, with material parameters consistent with the steel grades listed in Section 3.3.2. The material properties were defined using the nominal values of the specified grades; however, the use of mean values is not expected to significantly influence the results, as steel is not a governing component in the present configuration. The yield stresses adopted for S235, S355, grades 4.6, 8.8 and 12.9 were 235, 355, 240, 660 and 1120 MPa, respectively, while the corresponding ultimate stresses were 400, 500, 400, 800 and 1200 MPa. Plastic strain was set to zero at the yield stress for all grades, and the plastic strain at the ultimate stress was taken as 0.2 for S235, S355 and grade 4.6, and 0.1 for grades 8.8 and 12.9.

For timber, a VUMAT subroutine was developed to simulate failure mechanisms under tension and shear. The implementation follows the constitutive model proposed by Sandhaas [40],

originally formulated as a UMAT for implicit analyses and reformulated here for the explicit framework. However, in regions predominantly subjected to compressive loading, the damage formulation was deliberately deactivated and a bilinear elastic–plastic response was assigned in all material directions. This modeling choice is physically motivated by the fact that, under compressive contact conditions, timber is able to continue transferring load even if local shear stress levels exceed the nominal shear damage threshold. Activating shear-driven damage in such regions would lead to an artificial loss of stiffness or premature element deletion, resulting in unrealistic post-peak load drops once contact-induced crushing governs the response. To avoid this numerical artifact and to preserve a realistic load transfer mechanism beneath contact interfaces, brittle tensile and shear damage was therefore deactivated beneath compressive contact surfaces (Figure 13). In practice, this was implemented by replacing the brittle damage formulation with an anisotropic bilinear constitutive response based on the Hill48 yield criterion [41], defined using the POTENTIAL option available in Abaqus.

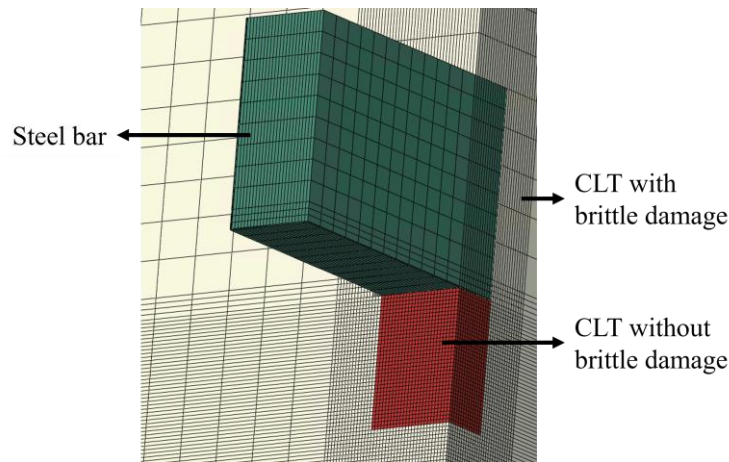


Figure 13 Regional damage modelling in the CLT model.

Damage initiation was governed by the Separated Damage Modeling (SDM) approach [42], in which each failure mode (e.g., tension, shear) is treated independently and triggered according to

the corresponding maximum stress criterion. Once the stress threshold is exceeded in a given direction, the damage variable evolves according to the appropriate damage law for that mode.

The formulation uses a scalar damage parameter $d(\kappa)$, which progressively reduces the material stiffness. Two distinct evolution laws were employed. For ductile failure, the simplified Sandhaas law was adopted:

$$d(\kappa) = 1 - \frac{1}{\kappa} \quad (3)$$

For brittle failure, an exponential evolution law proposed by Benvenuti et al. (2024) was used [43].

The formulation is based on fracture energy concepts and was regularized through the crack band theory, in which the mode-specific fracture energy ($G_{f,\theta}$) is divided by the characteristic element length to ensure mesh-independent energy dissipation:

$$d(\kappa) = 1 - k_{\theta}^{-1} e^{-2Z_{\theta}(k_{\theta}-1)}, \theta = L, R, T, LR, RT, LT \quad (4)$$

$$Z_{\theta} = \frac{f_{\theta}^2}{2E_{\theta} \frac{G_{f,\theta}}{L_k} - f_{\theta}^2}, \theta = L, R, T \quad (5)$$

$$Z_{\theta} = \frac{f_{\theta}^2}{2G_{\theta} \frac{G_{f,\theta}}{L_k} - f_{\theta}^2}, \theta = L, R, T \quad (6)$$

Here, f_{θ} denotes the strength, $G_{f,\theta}$ the fracture energy, E_{θ} the elastic modulus, G_{θ} the shear modulus, L_k the characteristic element length, and κ a history variable in the corresponding direction. Before damage initiates, $\kappa=1$; after initiation, it evolves as the ratio between the trial stress (from elastic behavior) and the actual stress. As κ increases, the damage parameter d approaches to 1, representing stiffness degradation. Element deletion is triggered when d reaches 0.999999, which corresponds to a complete loss of load-carrying capacity in the failed direction. The material parameters for timber were obtained by converting characteristic values into mean values using the Gaussian normal distribution, following the procedure described by Sandhaas [40].

Fracture energy values were also adopted from Sandhaas [40]. All timber properties used in the simulations are summarized in Table 6.

Table 6 VUMAT input parameters for CLT panels.

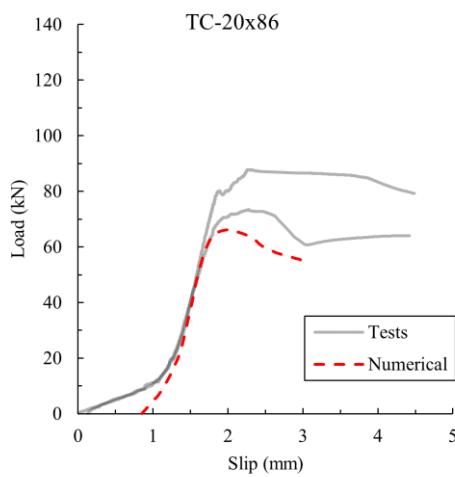
Modulus of Elasticity (MPa)					
E_{11}	$E_{22}=E_{33}$		$G_{12}=G_{23}$	G_{23}	
11000	370		690	50	
Strength components (MPa)					
$f_{c,0}$	$f_{t,0}$	$f_{c,90}$	$f_{t,90}$	f_v	f_r
36	24	4.3	0.7	6.9	0.5
Fracture Energies (N/mm)					
$G_{f,0}$	$G_{f,90}$		$G_{f,v}$	$G_{f,r}$	
60	0.5		1.2	0.6	

4.1.2 Comparison with experimental results

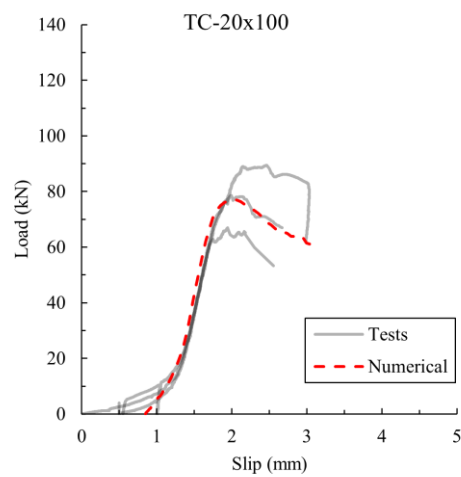
Figure 14 compares the load–slip curves obtained experimentally and numerically. To ensure a consistent visual comparison, the numerical curve was shifted along the displacement axis so that its linear elastic region aligns with that of the experimental curve. This adjustment was applied to remove the influence of initial compliance and seating effects associated with the test setup. Overall, the component simulations show good agreement with the experiments in terms of both resistance and stiffness. However, for the TC-25×86 specimens, the numerical stiffness was around 51% higher than the experimental values. These specimens also showed lower stiffness than the TC-20×86 series, despite their larger loaded area. As discussed in Section 3.5.2, this unexpected behavior is attributed to the proximity of the groove to the CLT interlayer, which likely activated rolling-shear effects and reduced the measured stiffness. The simulations support this

interpretation: in the 25 mm groove configuration, rolling-shear damage progressed almost to the point of full loss of load-carrying capacity, whereas in the 20 mm case it remained at an early stage. This is illustrated in Figure 15a, where SDV32 represents the rolling-shear damage parameter and $SDV32 \approx 1$ corresponds to complete loss of load-bearing ability followed by element deletion. Although the models did not directly predict a stiffness reduction with increased groove depth, the lower stiffness observed in the 25 mm tests is consistent with rolling-shear mobilization when considering specimen imperfections.

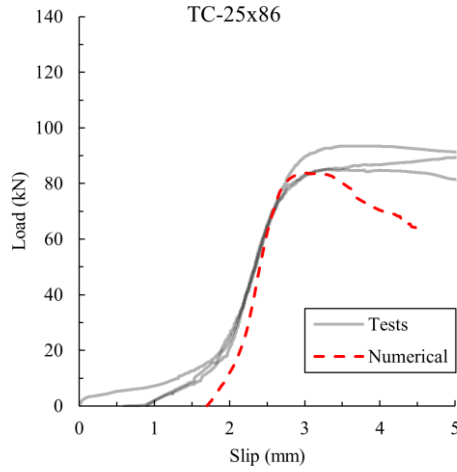
For the connection tests, the numerical model showed very good agreement with the experiments in terms of resistance. Regarding stiffness, the predictions closely matched the average response of the SBC-B specimens. In contrast, although the model reproduced the stiffness of the stiffest SBC-BS test, the comparatively lower stiffness observed in the remaining SBC-BS specimens resulted in an overall overestimation of about 12%. Given that the stiffness in the connection tests arises from the interaction of several components, a certain degree of variability is expected. Furthermore, because the numerical simulations represent an idealized system, it is reasonable that they correlate most strongly with the experimental specimens exhibiting the highest stiffness.



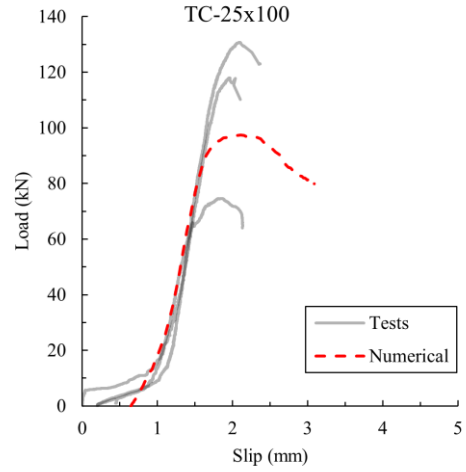
(a)



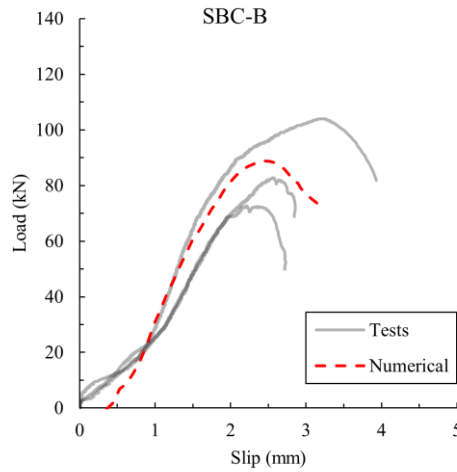
(b)



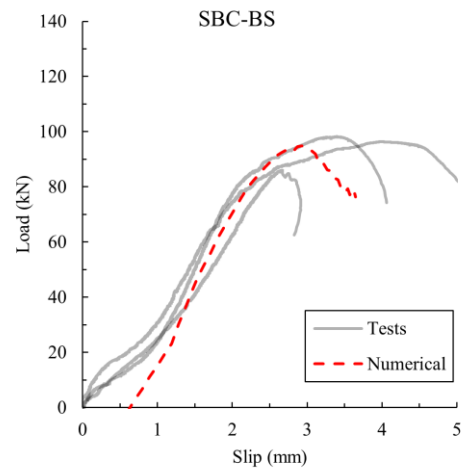
(c)



(d)



(e)

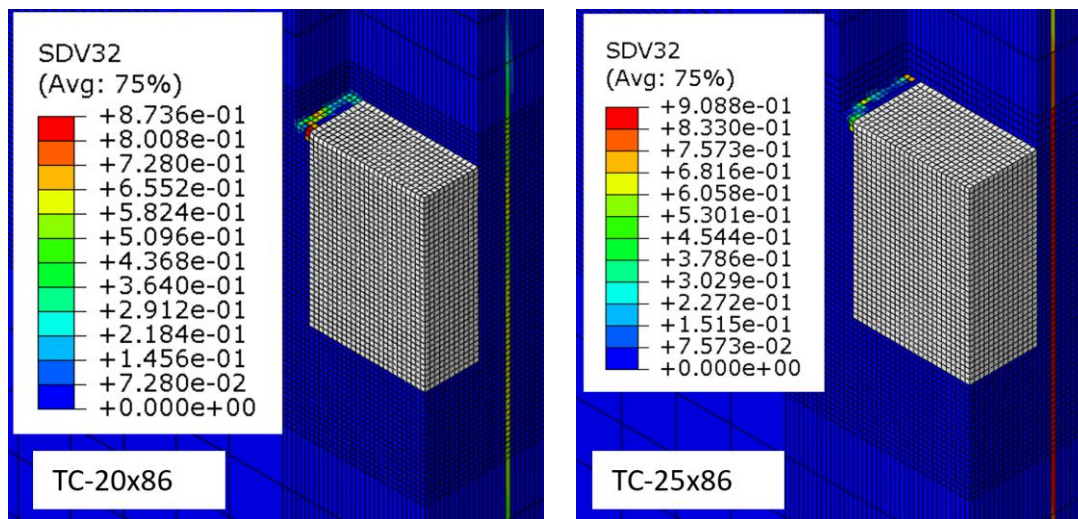


(f)

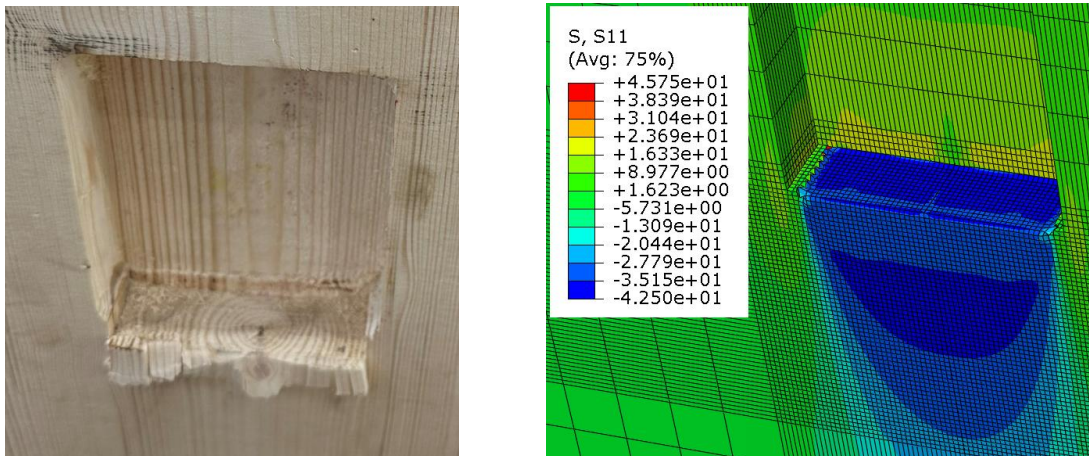
Figure 14 Comparison of experimental and numerical load-slip responses: (a) TC-20x86, (b) TC-20x100, (c) TC-25x86, (d) TC-25x100, (e) SBC-B, (f) SBC-BS.

Figure 15 presents failure modes observed in the simulations. SDV34 represents the maximum value among the tensile and shear damage variables in all directions. A value of 1 for the damage parameters indicates that the element can no longer sustain load, resulting in collapse of the elements.

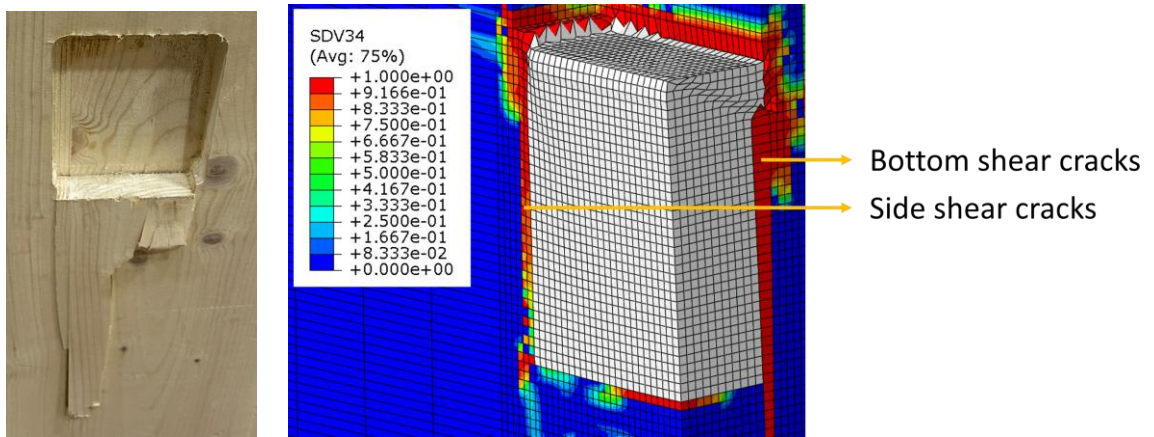
In the component tests, timber crushing was the predominant failure mode (Figure 15b). However, for the post peak stage, shear-tensile cracks were also captured in the simulations (Figure 15c). This behavior highlights the potential of the timber crushing component to induce shear-tensile combined failures, and it also explains the local plug shear failure observed in the TC-20×86 specimens. For the connection tests, the dominant crushing and embedment failure observed experimentally was consistently reproduced numerically (Figure 15d). Lastly, the numerical model predicted the formation of a single plastic hinge in the screws, which is in agreement with the majority of experimental observations. (Figure 15e).



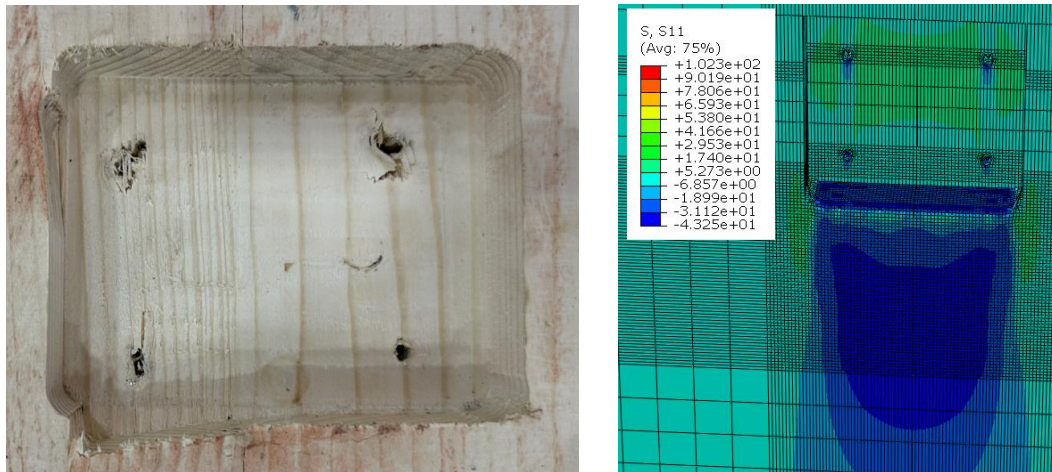
(a)



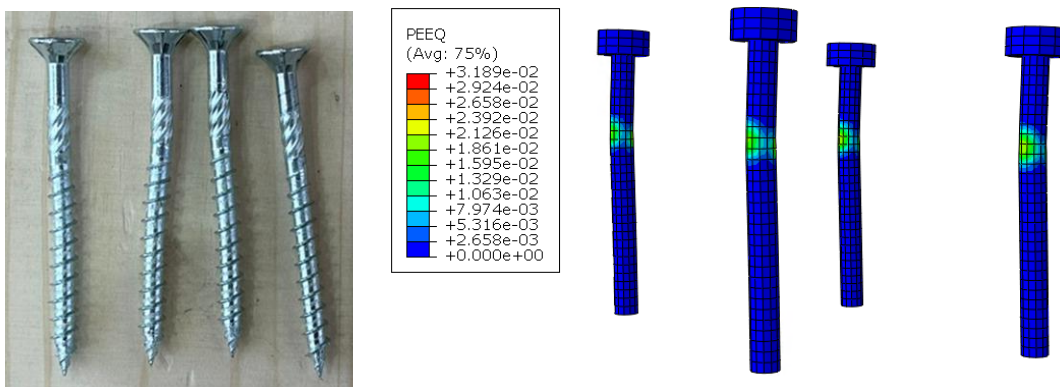
(b)



(c)



(d)



(e)

Figure 15 Observed failure modes; (a) rolling shear in component tests (cut view); (b) timber crushing in component tests; (c) local plug shear failure (cut view); (d) timber crushing and embedment in connection tests; (e) plastic hinge of the screws.

*A patent application related to this connector is currently pending.

4.2 Analytical approach

The component method was initially developed to model the behavior of steel joints and connections by characterizing the mechanical response of the activated components, thereby enabling the derivation of the global moment–rotation ($M-\theta$) or force–deformation ($F-\delta$) relationship [44]. This approach has been incorporated in design standards such as Eurocode 3 Part 1-8 [32], which provides the basis for deriving the global joint behavior. The method is typically applied in three steps: (i) identifying the active components, (ii) determining their mechanical properties, and (iii) assembling them to obtain the joint response. Although originally developed for steel joints, the same analytical framework has also been applied to composite systems, including TCC and STC structures [42-44].

In the present work, a similar philosophy is adopted to characterize the mechanical response of the proposed steel block connection. The load-transfer mechanism is decomposed into individual components, each represented by simplified mechanical models. The assembly of these components allows the global stiffness and strength of the connection to be derived analytically, providing a complementary interpretation to the experimental and numerical investigations.

4.2.1 Analytical model of the connection

The load transfer in the proposed steel block connection can be described through a series of connection components, each characterized by stiffness and resistance. The load is first transmitted

from the steel part (e.g., beam flange) to the bolts (1), mobilizing bolts in shear and bearing of the steel plate (2), with the possibility of localized brittle failure in the steel part. Then, it reaches the steel block connector, where compression and bearing stresses develop (3). The connector subsequently distributes the load to the timber panel through connector crushing and screw embedment (4) (Figure 16a). Within the timber, the main activated components are the crushing resistance and potential brittle mechanisms such as plug shear, row shear, and splitting.

The screws act as steel-to-timber dowel-type connectors, transferring load primarily through shear and embedding actions in the surrounding timber. Therefore, their mechanical behavior is represented using the analytical formulations originally proposed by Johansen (1949) [48] for dowel-type connections and later adapted for timber structures in Eurocode 5 [49]. However, since the steel block connector behaves as a thick plate, the failure modes involving screw rotation or bearing within the steel plate region, are not expected to occur. These modes are consequently excluded from the analytical model, in accordance with the recommendations of Eurocode 5. As a result, in the mechanical idealization, the resistance of the timber through the steel block connector and the resistance of the timber through the screws are modeled in parallel, as shown in Figure 16b. The list of activated components shown in Figure 16b is provided in Table 7.

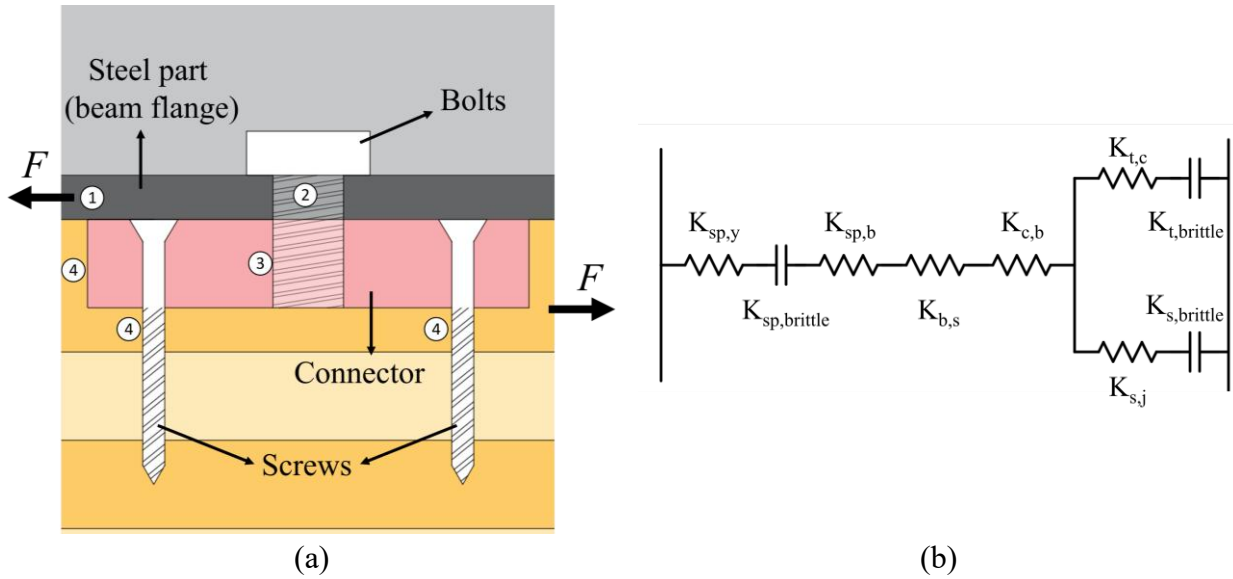


Figure 16 (a) Proposed steel block connection under shear loading, (b) mechanical spring model.

*A patent application related to this connector is currently pending.

Table 7 Description of the active components.

Component	Description
$K_{sp,y}$	Steel part (beam flange) yielding
$K_{sp,brittle}$	Steel part (beam flange) brittle failure
$K_{sp,b}$	Steel part (beam flange) in bearing
$K_{b,s}$	Bolt in shear
$K_{c,b}$	Steel block connector in bearing
$K_{t,c}$	Timber crushing
$K_{t,brittle}$	Timber brittle
$K_{s,j}$	Screws Johansen
$K_{s,s}$	Screws in shear

4.2.2 Characterization of the active components

The components associated with the steel part of the connection have been extensively characterized in design standards, and their stiffness and resistance properties are well established. Typical examples include bolt in tension, bearing in steel plate, and plate in bending, which are covered in detail in Eurocode 3.

In contrast, although the dowel type ductile embedment mechanism (Johansen component) (Figure 17a) is well established in current design standards [49], two timber related components remain insufficiently defined: the local timber crushing component and the brittle failure components. CEN/TS 19103:2021 [23] integrates a crushing component originally developed and experimentally substantiated for timber–concrete composite (TCC) systems [47-49] (Figure 17b). However, this model is based on notches extending over the full timber width and determines the resistance simply by multiplying the contact area by the compressive strength of timber. Considering the non-uniform stress distribution that develops within a local groove, this approach is likely to provide conservative estimations. Furthermore, the associated stiffness is defined through a purely geometric expression, independent of the material properties of timber. For this reason, the present study includes dedicated tests on the timber crushing component to evaluate the accuracy and applicability of these existing formulations. The resistance ($F_{t,c}$) and stiffness ($K_{t,c}$) model for the timber crushing component proposed in CEN/TS 19103:2021 [23] is given by:

$$F_{t,c} = f_{c,0,d} * b_c * d_c \quad (7)$$

$$K_{t,c} = \begin{cases} 1000 * b_c \text{ N/mm} & \text{for } d_c = 20\text{mm} \\ 1500 * b_c \text{ N/mm} & \text{for } d_c \geq 30\text{mm} \end{cases} \quad (8)$$

Where $f_{c,0,d}$ denotes the design compressive strength of timber parallel to the grain, while b_c and d_c represent the width and depth of the contact surface, respectively. For stiffness, Linear interpolation is recommended for groove depths d_c between 20 mm and 30 mm.

With respect to brittle failure modes (Figure 17c), the available analytical formulations are relatively recent and remain insufficiently validated. However, the present study does not further investigate these mechanisms, as the risk of such failures is not relevant for the current configuration. The connection examined herein mainly focuses on the local transfer of load through contact and compression between the steel block connector and the timber, where brittle

mechanisms such as plug shear or splitting are unlikely to be activated. Nevertheless, for applications involving free edge configurations, such failure modes should be considered in future design or analytical evaluations.

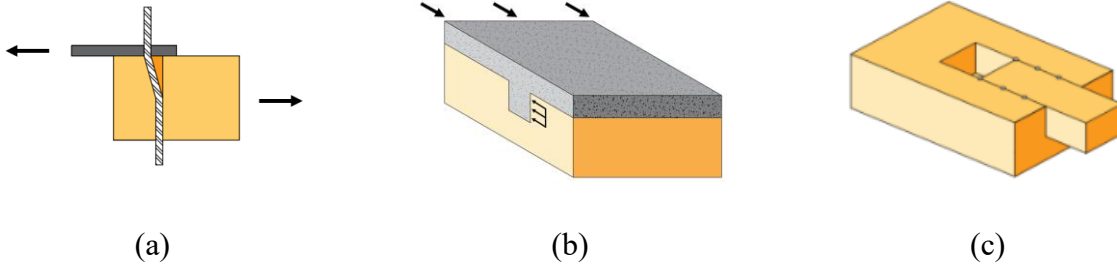


Figure 17 Activated timber components of the proposed steel block connection; (a) Johanssen, (b) timber crushing, (c) brittle failure of timber.

4.2.3 Assembly of the connection

After the components were characterized and the mechanical model was established in Sections 4.2.1 and 4.2.2, the global resistance and stiffness response of the connection must be determined in accordance with the principles of the component method. In the system shown in Figure 16b, the component springs connected in series are governed by the element with the lowest resistance, which controls the maximum resistance. For the evaluation of the stiffness, both the parallel and series arrangements of the springs are considered based on mechanical combination rules. The maximum resistance (F_{con}) and elastic stiffness (K_{con}) of the connection is determined as follows:

$$K_{con} = \left(\frac{1}{K_{sp,y}} + \frac{1}{K_{sp,b}} + \frac{1}{K_{b,s}} + \frac{1}{K_{c,b}} + \frac{1}{K_{t,c} + K_{s,j}} \right)^{-1} \quad (9)$$

$$F_{con} = \min (F_{sp,y}, F_{sp,brittle}, F_{sp,b}, F_{b,s}, F_{c,b}, (\min(F_{t,c}, F_{t,brittle}) + \min(F_{s,j}, F_{s,brittle}))) \quad (10)$$

4.3 Assessment of the models

In this section, the available analytical models are evaluated by comparing their predictions with the experimental and numerical results. For the timber crushing tests, the response is governed by

a single component, namely the local timber crushing component, whose resistance and stiffness are taken directly from the relevant design standard (Section 4.2.2).

For the proposed steel block connection, the analytical characterization follows the steps outlined in Section 4.2.1. The timber crushing component was characterized by using the same standard adopted for the isolated crushing tests, while the components associated with the bolted steel profile were defined in accordance with Eurocode 3, Part 1-8 [32]. The connector–CLT screw connection, which represents a steel-to-timber interface, was characterized following the provisions of Eurocode 5.

In all analytical evaluations, the mean material properties of timber were used (Table 8), and a safety factor of 1.0 was consistently adopted in order to enable a direct comparison with the experimental and numerical results.

The ratios between the analytical, numerical, and experimental maximum resistance and stiffness values obtained from these calculations are presented in Table 8.

Table 8 Comparison of experimental, analytical, and numerical results.

Specimen	F_{test}/F_{ana}	F_{test}/F_{num}	F_{ana}/F_{num}	K_{test}/K_{ana}	K_{test}/K_{num}	K_{ana}/K_{num}
TC-20x86	1.30	1.22	0.94	1.21	0.96	0.80
TC-20x100	1.09	1.02	0.93	1.28	0.98	0.76
TC-25x86	1.16	1.07	0.92	0.81	0.66	0.82
TC-25x100	1.20	1.11	0.92	1.27	1.12	0.89
SBC-B	1.09	0.97	0.89	1.22	1.01	0.83
SBC-BS	1.11	0.99	0.89	0.98	0.88	0.90
Mean	1.16	1.06	0.92	1.13	0.94	0.83
(%CoV)	(6.5)	(7.8)	(2)	(15)	(15)	(5.6)

The numerical model showed a strong correlation with the experimental results, with the simulations predicting on average 6% lower maximum resistance than the tests across the six specimens, which represents a very good level of agreement for this type of analysis. The stiffness values obtained numerically were about 10% higher than the experimental averages. However, excluding the TC-25×86 specimen with its unexpectedly low stiffness, the difference decreases to about 1%. With good agreement in both failure modes and global response parameters, the numerical model provides a reliable reference for evaluating the analytical formulations.

When examining the analytical resistance model, the predicted values were on average 16% lower than the experimental results and approximately 9% lower than the numerical predictions. This underestimation primarily arises from the model's neglect of stress-spread effects. Regarding stiffness, even considering the unexpectedly low stiffness measured in the TC-25×86 specimen, the analytical formulation underestimated the experimental stiffness by 12% and the numerical stiffness by 20%. This discrepancy stems from the stiffness expression being solely geometry-based and independent of timber material properties.

Overall, although the existing formulations provide reasonably good predictions, the resistance model could benefit from a modification to account for the stress-spread effect, which is currently not captured. In terms of stiffness, relying solely on geometric parameters appears insufficient, indicating the need for a more substantial improvement in the formulation.

Based on the findings obtained from the analytical and numerical investigations, a parametric and sensitivity study was conducted to further examine the influence of key geometric and material parameters on the connection behaviour.

5 Parametric and sensitivity study

5.1 Mechanical contribution of the active components

This section presents a parametric and sensitivity study aimed at identifying the key components governing the behaviour of the proposed steel block connection. As an initial step, the relative contributions of the individual components to the overall resistance and initial stiffness are quantified for the reference configuration. For this purpose, an analytical component-based assessment is performed (see Section 4.2), and the corresponding resistance and stiffness contributions are reported in Figure 18a and Figure 18b, respectively.

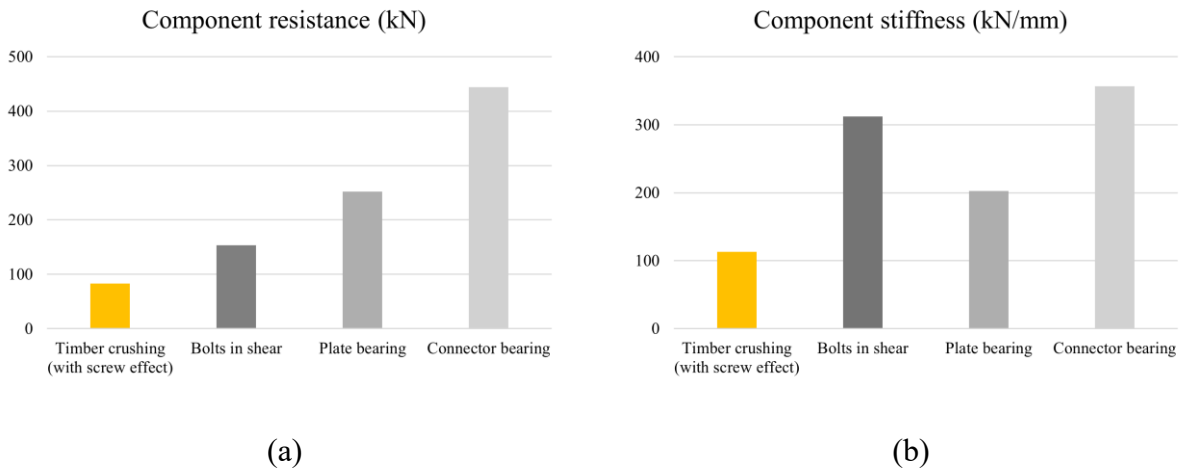


Figure 18 Contribution of individual components to the connection response: (a) resistance and (b) stiffness.

As illustrated in Figure 18, the screw yielding component is not treated independently but is incorporated into the timber crushing component, as both contributions are combined within the analytical formulation (Figure 16b). In the current configuration, the influence of the screws remains marginal, accounting for approximately 4.5% of the total resistance and less than 1% of the corresponding stiffness. For visual clarity, this contribution is therefore not reported separately.

Regarding the steel plate, yielding is not activated in the present loading configuration due to the absence of tensile forces. Under the current conditions, the plate response is governed by compression, for which the buckling resistance corresponding to half of the section is approximately 650 kN. In contrast, if tensile forces were present, the corresponding section yielding capacity for half of the section would be approximately 640 kN.. Both values remain significantly higher than the resistance levels observed in the connection. The associated axial stiffness is approximately 3500 kN/mm. Consequently, the steel plate yielding component does not influence the global response within the investigated range and is therefore excluded from the sensitivity analysis to maintain clarity in the visual comparison.

Based on this analytical assessment, five parameters are selected for further investigation: connector width, bolt diameter, bolt class, screw diameter, and steel profile geometry. The connector width is directly related to the timber crushing mechanism, while the remaining parameters are associated with the steel fasteners and their contribution to the overall connection behavior.

5.2 Analytical parametric study of the active components

This section presents an analytical parametric study based on the methodology and spring-based representation introduced in Section 4.2. The individual components are systematically varied to quantify their influence on the global resistance and stiffness of the connection. The resulting trends provide a basis for identifying the critical components to be further investigated in the subsequent numerical analysis. The analytical parametric study of the active components is shown in Table 9.

Table 9 Analytical parametric study of the active components

Parameter	Value	Resistance (kN)	Failure mode	Stiffness (kN/mm)
Bolt diameter (mm)	12	84.6	Timber crushing	41.0
	16	84.6	Timber crushing	51.5

	20	84.6	Timber crushing	59.6
Bolt class	4.6	62.8	Bolts in shear	38.5
	8.8	84.6	Timber crushing	47.4
	12.9	84.6	Timber crushing	51.5
Screw diameter (mm)	3.5	81.9	Timber crushing	51.2
	5	84.6	Timber crushing	51.5
	6	88.3	Timber crushing	51.8
Steel profile	IPE 300	84.6	Timber crushing	49.6
	HE 160B	84.6	Timber crushing	51.5
	HE 180B	84.6	Timber crushing	52.8
Connector width (mm)	80	68.8	Timber crushing	46.4
	100	84.6	Timber crushing	51.5
	120	100.5	Timber crushing	55.6

The influence of bolt diameter was assessed by considering M12, M16, and M20 configurations. While the connection resistance remains constant at 84.6 kN due to an unchanged governing failure mode, the initial stiffness shows a clear positive correlation with bolt diameter.

The effect of bolt class was evaluated by considering classes 4.6, 8.8, and 12.9. For the 4.6 class, the bolt shear resistance becomes lower than the timber crushing resistance, leading to a shift in the governing failure mode and a reduction in the connection resistance of approximately 32%. For higher classes (8.8 and 12.9), the governing failure mode remains timber crushing, and the resistance stabilizes. Nevertheless, an increase in bolt class results in a moderate increase in initial stiffness.

The influence of screw diameter, investigated for typical sizes of 3.5, 5, and 6 mm, was found to be negligible, with only minor variations in both resistance and stiffness.

Similarly, variations in the steel profile (IPE300, HE160B, and HE180B), corresponding to flange thicknesses between 10.7 mm and 14 mm, do not significantly affect the resistance, stiffness, or governing failure mode of the connection.

In contrast, the connector width has a direct influence on both resistance and stiffness. Increasing the width from 80 mm to 120 mm leads to a proportional increase in the timber crushing area, which is reflected in a comparable increase in connection resistance. A similar, though less pronounced, trend is observed for the initial stiffness.

The analytical results show that bolt diameter, bolt class, and connector width are the primary parameters influencing the mechanical response of the connection. Bolt diameter and bolt class mainly affect the initial stiffness, whereas their influence on resistance becomes relevant when they alter the governing failure mode. Connector width has a more direct effect on both resistance and stiffness, owing to its direct contribution to the timber crushing area.

In contrast, variations in screw diameter and steel profile have a negligible influence on both resistance and stiffness within the investigated range. Based on these findings, connector geometry, bolt diameter, and bolt class are identified as the governing parameters and are therefore selected for further numerical investigation.

5.3 Numerical investigation of the governing components

This section presents a numerical parametric study focusing on the governing components identified in Section 5.1. The same numerical modelling framework described in Section 4.1.1 is retained to ensure consistency and comparability of the results. In addition, the simplified damage model proposed in [53] is implemented in Abaqus to capture the behaviour of the bolt shear component. The resulting load–slip responses obtained from the simulations are presented in Figure 19.

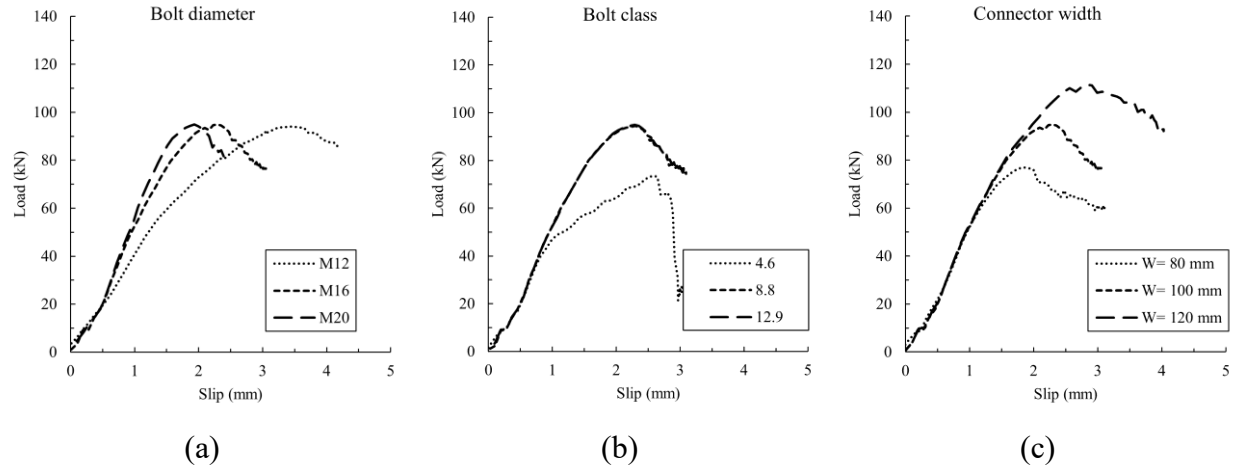


Figure 19 Numerical load-slip responses for the parametric variations of (a) bolt diameter, (b) bolt class, (c) connector width.

As shown in Figure 19, the numerical load-slip responses generally follow the trends predicted by the analytical model in terms of connection resistance. The influence of the investigated parameters on the peak load is consistently captured. In addition, the governing failure modes are correctly predicted, with the bolts-in-shear component governing for the 4.6 bolt class, while timber crushing governs for the higher bolt classes, in agreement with the analytical assessment. In contrast, differences become more apparent in the initial stiffness, where the numerical responses exhibit variations that are not fully reflected by the analytical formulation. These discrepancies indicate that a more detailed quantitative comparison is required to assess the stiffness prediction. To this end, the ratios of resistance and stiffness obtained from the analytical and numerical approaches are summarized in Table 10.

Table 10 Comparison of the numerical and analytical results

Parameter	Value	F_{ana}/F_{num}	K_{ana}/K_{num}
Bolt diameter (mm)	12	0.89	0.97
	16	0.89	0.90
	20	0.89	0.96

	4.6	0.85	0.67
Bolt class	8.8	0.89	0.84
	12.9	0.89	0.90
Connector width (mm)	80	0.89	0.84
	100	0.89	0.90
	120	0.89	0.94

As shown in Table 10, a consistent difference of approximately 10% is observed between the analytical and numerical predictions in terms of connection resistance. As discussed in Section 4.3, this discrepancy primarily arises from the analytical timber crushing model, which is based solely on the contact area, whereas in reality an additional contribution develops due to stress spreading effects within the timber.

In contrast, the stiffness predictions do not exhibit a systematic mismatch, but noticeable differences are observed in the extent to which the investigated parameters influence the initial stiffness. Although the analytical and numerical results remain of the same order of magnitude, variations in the relative sensitivity to parameter changes are evident.

To further assess these differences, the relative variations in resistance and stiffness with respect to the investigated parameters are compared in Figure 20. This comparison enables a more detailed evaluation of both the influence of each parameter and the extent to which the analytical approach overestimates or underestimates these effects.

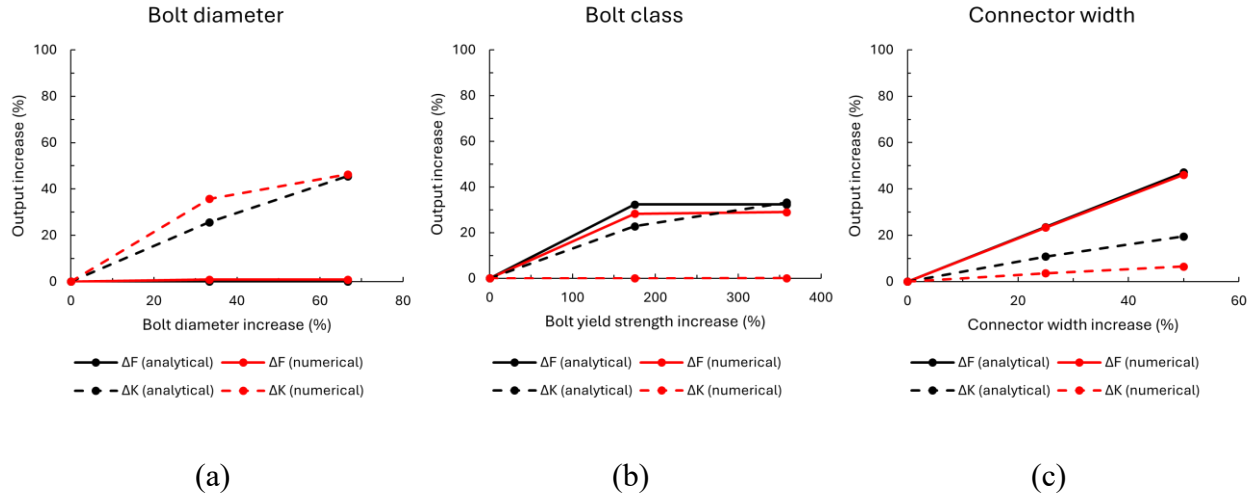


Figure 20 Predicted output increases with (a) bolt diameter, (b) bolt yield strength, (c) connector width.

For the bolt diameter, both the analytical and numerical models exhibit very similar trends in terms of resistance and stiffness. While the increase in bolt diameter results in a clear increase in stiffness, no variation in connection resistance is observed as long as the governing failure mode remains unchanged.

For the bolt class, both approaches successfully capture the change in resistance associated with the transition of the governing failure mode from bolts in shear to timber crushing when increasing the bolt class from 4.6 to 8.8. In contrast, no additional increase in resistance is observed from class 8.8 to 12.9, since the governing failure mode remains unchanged. In contrast, the most pronounced discrepancy in this parametric study is observed in the stiffness response. While the analytical model predicts a moderate increase in stiffness with increasing bolt class, the numerical results indicate no significant variation in the initial elastic stiffness. This difference arises from the formulation of the analytical bolts-in-shear stiffness component, which is directly correlated with the ultimate strength of the bolt material. In the numerical model, however, the connection response

remains predominantly elastic in the initial regime, with limited local plasticity in the bolts, resulting in a negligible effect on the initial stiffness. Nevertheless, as observed in Figure 19, earlier onset of plasticity in lower bolt classes leads to a more rapid reduction in stiffness and a significantly lower post-yield stiffness. Despite this behaviour, the initial elastic slopes of the load–slip curves remain largely similar across the different bolt classes.

The connector width emerges as a governing parameter in this configuration due to its direct control over the timber crushing component. Accordingly, its influence on the connection resistance is clearly reflected in both analytical and numerical results, with a consistent increase as the width increases. Although minor differences exist between the two approaches, both capture this trend in a coherent manner. A different behaviour is observed in the stiffness response. While the analytical model suggests a stronger sensitivity of stiffness to connector width, the numerical results indicate a more limited increase. This discrepancy implies that the analytical formulation tends to overestimate the contribution of connector width to the initial stiffness of the connection.

6 Conclusions

In this study, a reversible steel–timber shear connection was proposed and examined through experimental, numerical, and analytical investigations. The performance of the connector was also experimentally compared with one of the most commonly used solutions, the conventional bolted connection. In addition, the sensitivity of the connection to key parameters was evaluated through a parametric and sensitivity analysis. The key findings can be summarized as follows:

- The proposed steel block connection mechanically outperformed the conventional bolted connection, achieving approximately 45% higher yield resistance and more than three times higher elastic stiffness.

- Unlike the conventional bolted connection, which exhibited irreversible timber embedment from the early stages of loading, the proposed steel block connection remained elastic up to higher load levels, preserving both the timber and steel components and thereby demonstrating clear potential for disassembly and reuse.
- When local timber crushing component is activated, deformations occurring particularly in the post-peak region may lead to sudden shear failure. Therefore, this phenomenon should be taken into account in the design of the connector.
- Although the proposed steel block connection specimens were tested until complete failure, no damage was detected in any of the steel components, besides screws. This finding highlights the strong potential for reusing the steel parts.
- In configurations governed by local timber crushing, increasing the crushing area typically results in a more brittle structural response
- For the proposed steel block connection, the results indicate that maintaining a groove–interlayer distance of around 10 mm leads to a more predictable and stable stiffness response, whereas smaller distances (e.g., 5 mm) showed increased variability. However, this threshold is specific to the present study, and further investigation is required to more precisely characterize the influence of groove–interlayer separation on stiffness.
- The numerical simulations showed good correlation with the experimental results in terms of resistance, stiffness, and failure modes.
- The analytical model for timber crushing underestimated the maximum resistance by an average of 14% when compared with the six experimental tests, and by approximately 9% relative to the numerical simulations. This underestimation is attributed to the

simplifications in the analytical formulation, which neglects stress-spread effects and assumes that only the nominal crushing area contributes to the compressive resistance.

- The analytical model for the stiffness of the timber crushing component, despite the unexpectedly low stiffness observed in the TC-25×86 specimen, underestimated the experimental stiffness by an average of 9% across all six tests and by approximately 17% relative to the numerical simulations. This discrepancy primarily stems from the fact that the stiffness formulation is purely geometric and does not incorporate any material parameters of timber.
- The main limitation of the proposed steel block connection is its relatively low ductility. Its high stiffness, while advantageous, inherently results in limited slip capacity. Therefore, during beam design, sufficient slip must be provided through other components or dedicated fuse elements.
- The parametric study showed that the connection response is primarily governed by timber crushing, resulting in a direct influence on both stiffness and resistance. The effect of bolt diameter and bolt class on resistance is dependent on the governing failure mode: these parameters influence the resistance only when they constitute the weakest component, while no significant effect is observed when they remain stronger than the governing mechanism. In contrast, the connection stiffness is directly correlated with bolt diameter, whereas variations in bolt class do not significantly affect stiffness as long as bolt yielding does not occur.
- Although this study demonstrated that the proposed steel block connector remains elastic up to significantly higher load levels, and thus indicates clear potential for disassembly and reuse, further research is required to substantiate these conclusions through dedicated reuse

testing and comprehensive disassembly metrics, particularly under repeated assembly and disassembly cycles.

Conflict of Interest

A patent application (EP25180501 on 03/06/2025) has been filed, covering the design and application of the described connector.

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