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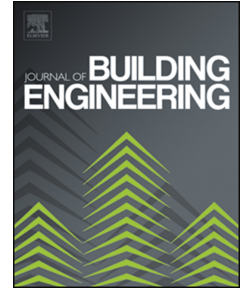
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# Journal Pre-proof

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# Limited-Damage 3D-Printed Interlocking Connection for Timber Volumetric Structures: Experimental Validation and Computational Modelling

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## Abstract

Cross laminated timber volumetric construction (CLTVC) is an innovative construction technology that combines the sustainability of timber and the efficiency of modular construction, as opposed to conventional construction. However, the connection installing methods of CLTVC, such as fastening, are laborious with limited accessibility for connection installations, thus hindering the application of CLTVC in mid- and high-rise structures. Therefore, a new way of connecting CLT modules by sliding and stacking is explored herein with a proposed damage-control interlocking connection system, aiming to provide a more efficient assembly solution to CLTVC that does not require onsite screwing. Quasi-static monotonic and cyclic test, and numerical analyses were conducted to assess the mechanical performance of the proposed connections, which possessed adequate translational stiffness and strength of the proposed connections. The connections' ability to control deformation – damage is moved away from timber and the embedded fasteners, was also well demonstrated in the test, as both screws and timber remained mostly intact after testing. The proposed connection design showcases a new concept of modules' assembly in volumetric construction with higher efficiency and flexibility; meanwhile demonstrates the potential in reducing the permanent damage to structural materials during service life and enabling reuse.

## 1. Introduction (more recent studies, new interlocking connections and their limitations)

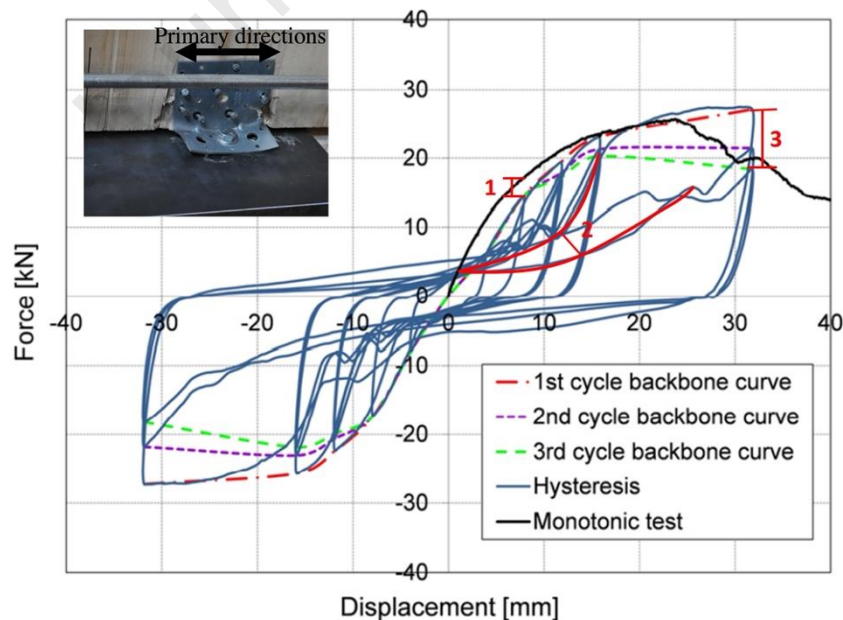
### 1.1 Multi-storey timber volumetric construction

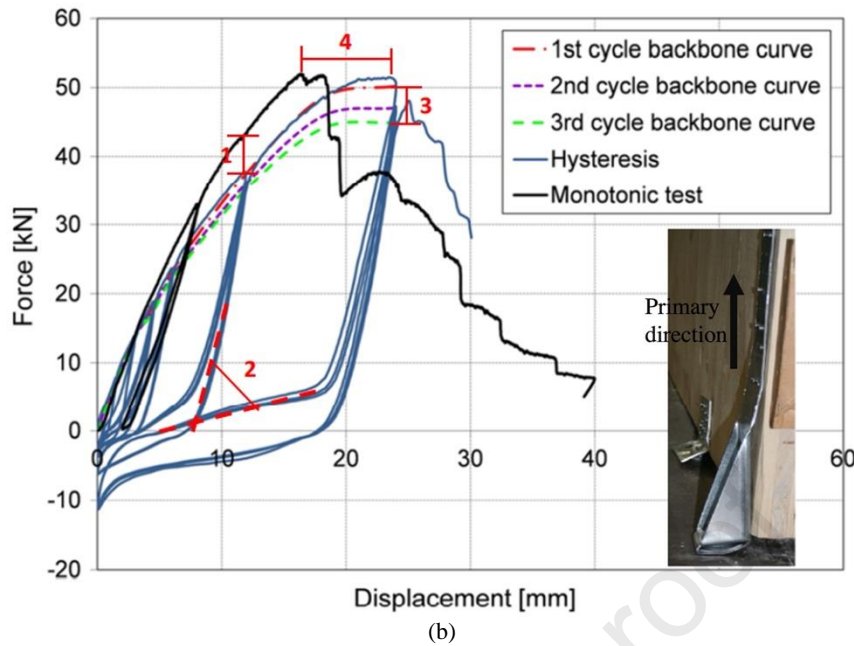
Volumetric construction is an emerging modern method of construction (MMC) that has the highest degree of prefabrication (~95%) [1] with the main structure and other building accessories such as cladding, internal finishes and MEP services all being manufactured and assembled into flat modules in factories (offsite manufacturing - OSM) before being transported to sites for the final assembly. With the advent of automation and the future factories (the fourth industrial revolution), better quality and higher precision of assembly work compared to the on-site manual operation can be ensured in volumetric construction. Being able to combine the sustainability of timber materials and the efficiency

38 of the OSM, volumetric timber construction is considered as one of the direct solutions to the growing  
 39 demand in affordable urban accommodation as well as the high carbon footprint problem own to the  
 40 construction industry [2]. Yet, this construction technology is limited to mid- and low-rise structures  
 41 [2], and the current trend of building tall volumetric timber structures imposes new challenges on the  
 42 wind and seismic resistances of such structural systems, requiring further investigations on varied  
 43 aspects of VTC; for example updates on design strategy in standards, high-performing connection  
 44 systems and assembly tools, as well as the development of more accurate numerical modelling methods  
 45 [3] for the confident delivery of secured high-rise volumetric timber structures.

## 46 1.2 Conventional timber connection systems for multi-story VTC

47 Among all the currently available forms of timber volumetric structures, the Cross Laminated Timber  
 48 (CLT) volumetric structure that employs load-bearing CLT panels, has somewhat better potential over  
 49 others (e.g., Framed Volumetric Structures and Structural Insulated Panels, SIPs) to be used for tall  
 50 buildings due to the high stiffness of CLT. However, suitable connections for this structural system still  
 51 lack sufficient investigation, as it is a relatively new construction technology. In volumetric  
 52 construction, the connections between modular units, also known as inter-module connections, are  
 53 critical to both the project efficiency and the overall structural performance of the buildings, as they  
 54 link prefabricated modular units together on-site to form the entire structure and are designed to transfer  
 55 horizontal and uplift forces from the wind and seismic actions. Previous quasi-static tests on CLT  
 56 panelised systems [4] [5] and shake table tests on full-scale low-rise CLT buildings [6] [7] indicated  
 57 that structures made of CLT panels generally demonstrated high strength and stiffness with most of the  
 58 deformation and energy dissipation being processed in connections and friction between timber panels.  
 59 Therefore, the proposal of high-performing connections is crucial to the realisation of high-rise  
 60 volumetric CLT structures with improved wind and seismic performances.





61 Fig. 1. Typical CLT connections (a) angle bracket and (b) hold-down and their representative monotonic and cyclic  
 62 responses and failure modes on the primary working directions [8]

63 Angle brackets and hold-downs (Fig. 1) are the commonly used shear and tensile connections in timber  
 64 construction, which are originally designed for post-and-beam timber structures, so they are not fully  
 65 applicable in CLTVC. Some well-known disadvantages of these connection systems in both  
 66 construction and structure aspects currently limit their application in high-rise CLT volumetric  
 67 structures. To name a few:

#### 68 1.2.1 Incompatibility with CLTVC and low construction efficiency

69 The limited accessibility is a widely recognised challenge in volumetric construction. The installation  
 70 of conventional timber connections requires external access, while accessibility for inter-module  
 71 connections has long been a key challenge in volumetric construction [9], especially when CLT  
 72 modules become enclosed as panelised structures, which makes the inter-module connections in  
 73 CLTVC entirely inaccessible (Fig. 2). Consequently, in some CLT volumetric structures [2, 10], the  
 74 inter-module connections are absent and the lateral resistance of the structure is provided by the friction  
 75 between modules or the additional structural reinforcing systems, such as steel frames and concrete  
 76 cores. This method is commonly adopted in low-rise CLTVC and the inaccessibility of inter-module  
 77 connections is treated as one of the main obstacles to achieving tall CLT volumetric structures with  
 78 sufficient lateral stiffness, considering the importance of inter-module connections in defining the  
 79 integrity and stability of volumetric structures.

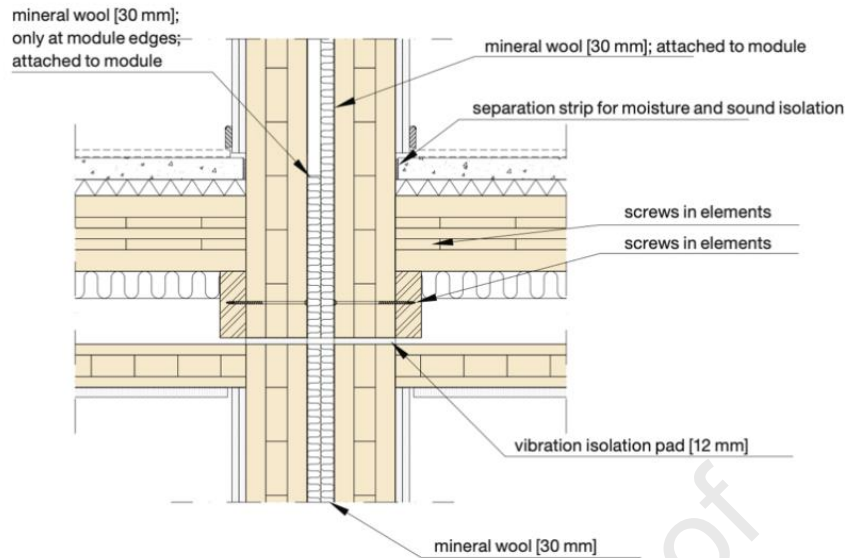


Fig. 2. Illustration of inaccessible inter-module connection between CLT modules [11]

### 1.2.2 Insufficient mechanical properties

According to the published experimental studies [8, 12], both angle brackets and hold-downs are characterised by high stiffness and strength but insufficient ductility (belonging to L or M ductility class as prescribed in EC8 [13]) in their primary directions (Fig. 1(a)&(b)). In the cyclic tests on conventional connections, several hallmarks can be observed (Fig. 1): (1) the reduced maximum strength at each step than that in monotonic test at the same displacement amplitude, (2) the stiffness degradation happens at load reversals, (3) the strength degradation occurs during the repeated cycles at the same displacement, (4) the delayed attainment of maximum strength in cyclic test compared to monotonic test.

The above-mentioned features are attributed to the cavities around fasteners formed by the crushing of timber and the deformation of fasteners during loading, which damage the capacity of surrounding timber and are highly unfavourable to the seismic design of high-rise structures, as they make the connection cyclic behaviours unpredictable and reduce the structure's capacity of energy dissipation during seismic events [14].

In addition, as the connection is the governing factor of the timber structures' ductility, the common concept of designing ductile timber structures is achieved by increasing the number of connections while using small-diameter fasteners. Large number of conventional connections with slender fasteners are therefore commonly adopted in large timber structures to achieve desired structural capacity (Fig. 3), which leads to time-consuming on-site fastening work that negatively impacts the project progress. This also limits the maximum reclamation and the reuse of timber components at the end-of-life of buildings, as the removal of nails and screws is labour-intensive and can damage the structural material [15]. Another concern of using small-diameter fasteners is the high stress they introduce on timber after experiencing significant deformation at the later stage of loading, which can pinch through the fibres before the capacity in timber is fully developed (Fig. 4(a)), causing sudden drop in connection strength and great residual displacement even after the removal of the external loading. This can be considered as brittle failure and cause irreversible damage to the structural elements, which results in difficulties in structural maintenance and should be avoided in connection design.



Fig. 3. Numerous angle brackets used on a single CLT shear wall to achieve sufficient capacity and ductility

### 1.2.3 Insufficient accuracy in analytical models for connection capacity calculation

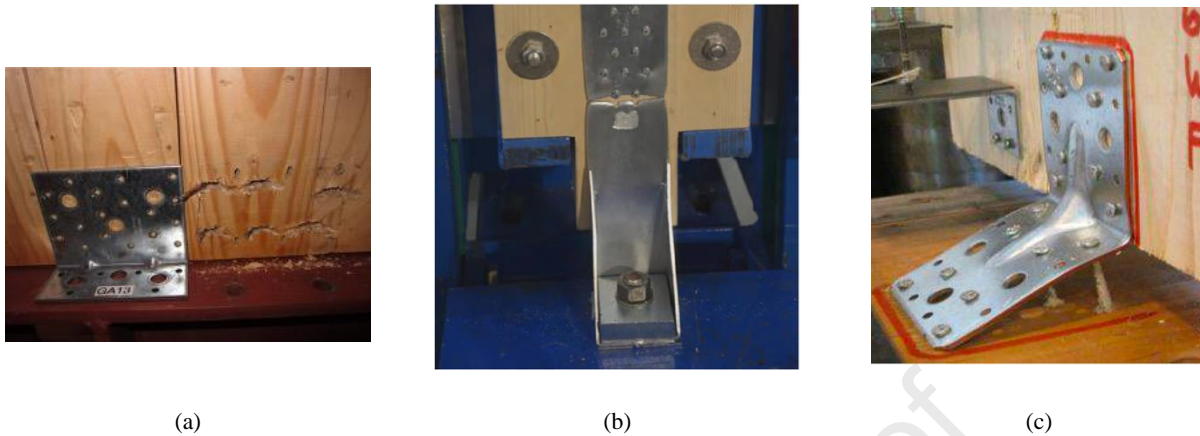
Comparison studies between analytical models and experiment results were conducted in previous research [8, 12], which indicated that the existing analytical models for timber connections provide conservative predictions on connection strength but seems to significantly overestimate the stiffness (up to 9 times higher according to the study of Gavric and Fragiaco [8]). One potential source of errors in the analytical models can be the assumption of brittle elements (timber) and ductile elements (fasteners and steel plate) working independently, with the embedding strength of timber and the yield moment of fastener being considered separately in calculations. This is because typical timber connections are composite structures, in which deformation happens simultaneously in ductile and brittle elements when loaded (Fig. 4(b)&(c)). Hence, considering the strength of different components independently and ignoring the composite effect when estimating the overall capacity of the connection could lead to significant differences between the actual strength and the calculated value. The much bigger actual strength of the embedded fasteners than the design value that was estimated with Johansen's theory in EC5 [16], and the overestimations of the stiffness and strength of the brittle elements due to the much greater scattering of timber material properties than steel material [17], can also lead to inaccurate estimations in connection design, which may cause non-ductile failure modes in connections (e.g., splitting of timber) [8, 12].

The afore-mentioned factors also affect the overstrength method that widely adopted in the research and practice of timber structures, which is developed by Jorissen and Fragiaco (Eq.1) [18] based on capacity-based design method to ensure the full activation of all ductile elements before the non-ductile area enters the plastic stage [19]. In this method, an overstrength factor  $\gamma_{Rd}$  accounting for all possible factors that may lead to unexpected high strength in ductile elements ( $R_{d,ductile}$ ) (underestimated material strength, potential strain hardening and contribution of steel plate at large deformations) is introduced in the design strength of brittle elements ( $R_{d,brittle}$ ) (Eq.1).

$$\gamma_{Rd} R_{d,ductile} \leq R_{d,brittle} \quad (1)$$

This factor is defined as the ratio of the 95th-percentile strength distribution to the analytical design strength capacity of the connection [18]. The inaccurate estimations of analytical models can lead to the

138 adoption of inappropriate overstrength factors, which may further result in the overdesign of structural  
 139 material or the unfavourable connection behaviours.



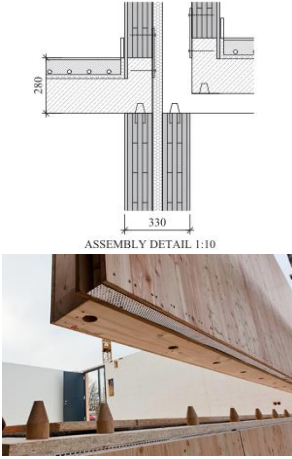
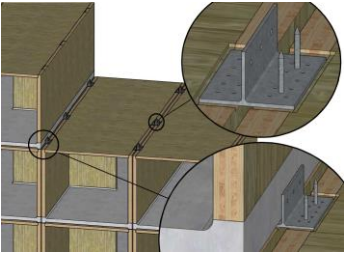

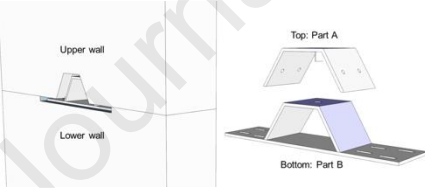
140 Fig. 4. Possible deformation modes in timber connections: (a) localised deformation on fasteners with timber fibres being cut  
 141 through [8] (b) brittle failure on metal flange (c) combination of bending of plate, splitting of timber and withdrawal of  
 142 screws [20]

### 143 1.3 Innovative connection systems


144 To address the limited accessibility of inter-module connections in volumetric construction, a new way  
 145 of assembling flat modules using interlocking techniques was explored in previous research [21, 22].  
 146 These conceptual studies demonstrate the potential of interlocking technique in achieving  
 147 replaceability, adaptability and dismantlability in volumetric structures. This technique was also  
 148 employed in the design of some recently proposed connection prototypes for CLT volumetric structures,  
 149 as summarised in Table 1. These new connections eliminate the onsite manual operation in CLTVC  
 150 needed for screwing, so that the access to inter-module connections is not required, and better  
 151 construction efficiency and accuracy can be achieved. Similar interlocking design can also be observed  
 152 in some new connections proposed for CLT panelised structure (Table.1). These recent developments  
 153 enrich the importance of interlocking technique in CLT connection systems for efficient construction  
 154 and upgraded structural performance. However, the listed interlocking connections for CLTVC are all  
 155 bespoke connection solutions proposed for specific projects, while a universal interlocking connection  
 156 solution for CLTVC has not yet been proposed. And these bespoke connections all require special  
 157 modifications on CLT panels for the fitting of connections, which limit their applications in other CLT  
 158 volumetric structures.

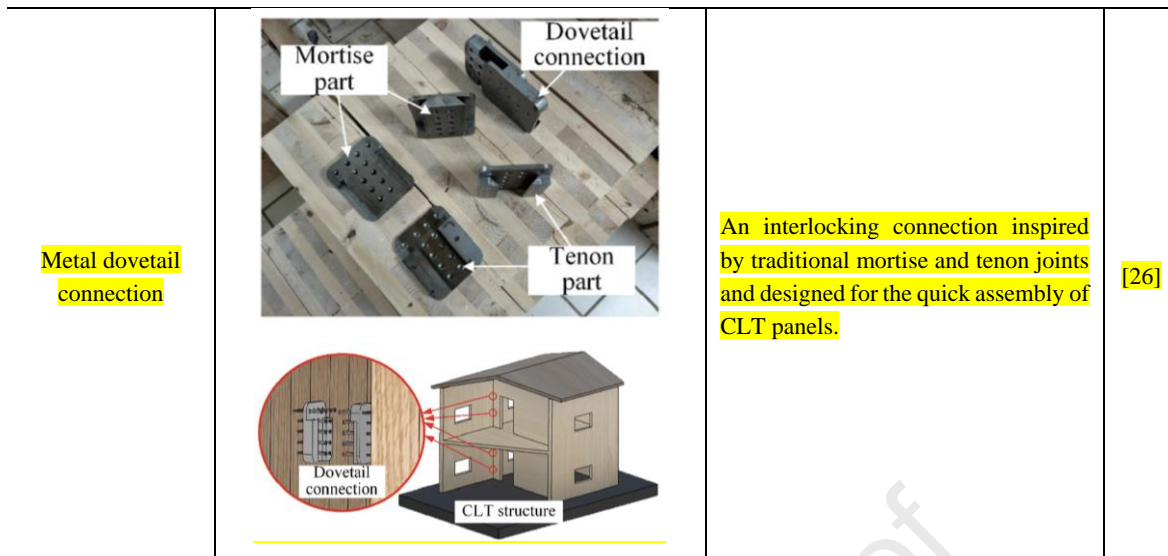
159 **Table 1.** List of innovative interlocking connection systems for CLT structures

Name	Connection Fig.s	Comments	Ref
<b>Novel connections for CLT volumetric structures</b>			

<p>Heidelberg Student Accommodation</p>	 <p>ASSEMBLY DETAIL 1:10</p>	<p>A bespoke interlocking inter-module connection for the easy installation of CLT flat modules.</p>	<p>[2]</p>
<p>Jakarta Hotel</p>		<p>A steel T-shaped angle plate with pins for the accurate alignment and translational constraints of modules</p>	<p>[23]</p>
<p>Moxy Modular Hotels (Marriott)</p>		<p>A specially designed notched CLT panel used as the side wall of modules and the inter-module connection for the easy on-site modules assembly.</p>	<p>[2]</p>
<p>A novel inter-module horizontal connection proposed by the University of British Columbia</p>		<p>A replaceable inter-module connection to achieve good energy dissipation and limit plastic deformation in timber.</p>	<p>[24]</p>

#### Interlocking connections for CLT panelised structures

<p>LOCK Connection from Rothoblaas Ltd.</p>		<p>A concealed, interlocking wall-to-floor connection for CLT structures. The Lock Connection is attached on the surfaces or edges of wall and floor CLT panels, so the floor panels can be slid into the right positions to connect with wall panels, eliminating on-site fastening work.</p>	<p>[25]</p>
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160

161 In some novel connections [27-31] designed for CLT panelised structures, another commonly adopted  
 162 design strategy is to introduce an individual ‘weak component’ in the connections to control the plastic  
 163 deformation. By applying capacity-based design to strengthen timber and fasteners, this design strategy  
 164 can efficiently isolate timber from deformation and localise the deformation within the additional metal  
 165 connectors. In this way, much improved strength and ductility can be achieved in connections with  
 166 reduced risks of brittle failure, considering the limited ability of timber to deform plastically. A similar  
 167 approach is also widely adopted in the recent proposals of dissipator [32-35] for CLT structures.

## 168 2. Novel Interlocking Connection System for CLT Modules

### 169 2.1 Concept of the interlocking connection

170 In order to achieve enhanced structural performance in multi-storey CLT volumetric structures and  
 171 simplify the module assembly process, a novel interlocking inter-module connection system is herein  
 172 proposed. This connection system consists of both vertical and horizontal connections that can be pre-  
 173 installed onto CLT modules in the factory (off-site) (Fig. 5(a)). It is suitable for most of the module  
 174 specifications, as it requires no further modification on panels for the fitting of connection, which can  
 175 greatly simplify and standardise the structural design process and become available to the entire market.  
 176 Owing to its interlocking feature, the modules can be accurately assembled on-site by sliding and  
 177 stacking without the need for additional tooling and special operations (Fig. 5(b)&(c)), so the on-site  
 178 assembly of modules will no longer be constrained by the access of inter-module connections and the  
 179 erection process of CLT volumetric structures can be significantly sped up.

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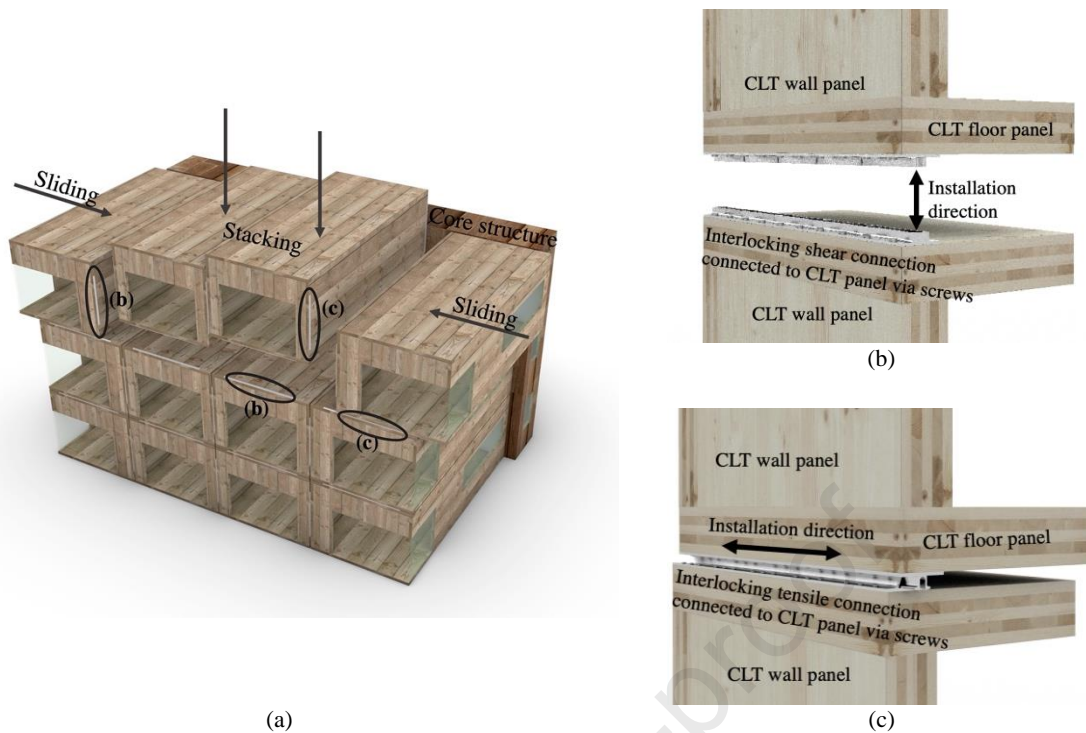
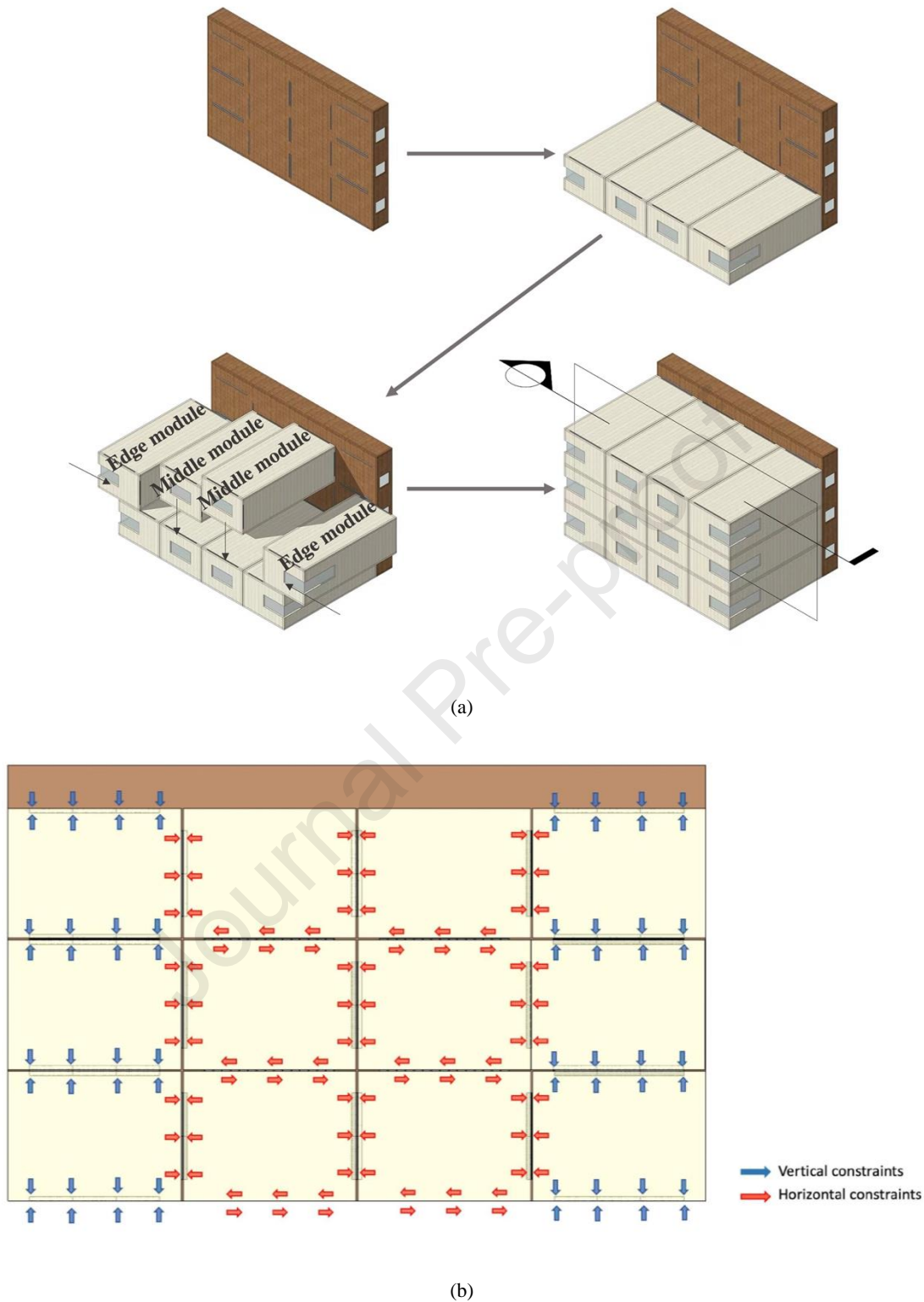


Fig. 5. Overview of the novel interlocking connection system for VTC: (a) Locations of connections in a CLT volumetric structure and the assembly of CLT modules with the pre-installed connections (b) the stacking of interlocking shear connection (c) the sliding of interlocking tensile connection

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185

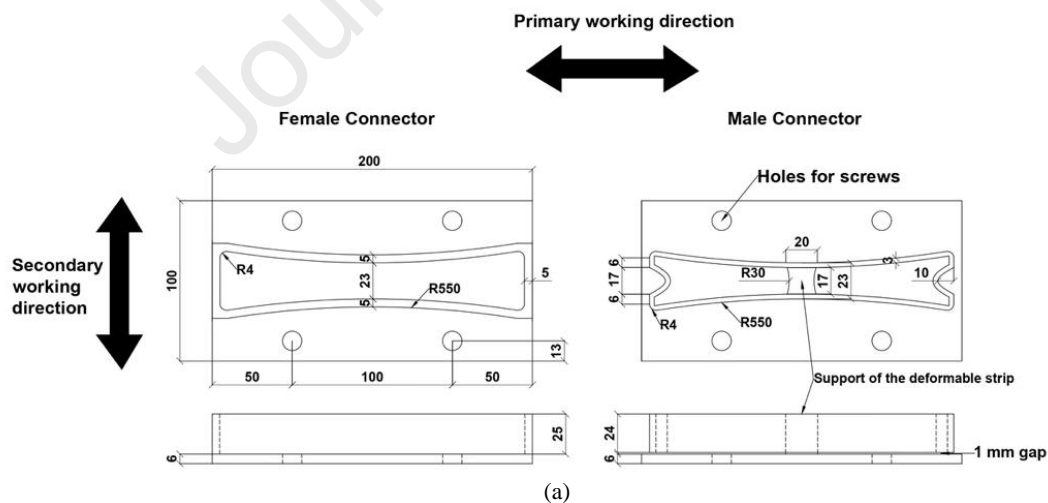
186 Due to the interlocking technique, both shear and tensile connections of this new system are not fully  
187 rigid in the installation direction and are designed to transmit shear or tensile forces only (Fig. 6(b)).  
188 After assembly, the movement of connectors can be locked by simple end-locking systems (i.e.  
189 extended plates with holes welded on the ends of the connectors, through which fasteners can fix two  
190 connectors). The full integration of constraints on the structure in both horizontal and vertical directions  
191 is achieved by the strategic placement of modules. In the CLTVC with interlocking connections as  
192 illustrated in Fig. 6, the core structure used as lift shaft or staircase is first constructed with preinstalled  
193 connections. Following the arrangement of the connections on the core structure, the flat modules with  
194 pre-installed connections can simply slide over or be stacked into the correct positions (Fig. 6(a)). The  
195 illustrated strategic assembly can form a continuous vertical reinforcement along the edge modules and  
196 a horizontal reinforcement along the middle modules (Fig. 6(b)), resisting the uplifting and inter-storey  
197 drift caused by external forces. The continuous reinforcement provided by this connection strip design  
198 can also minimise load concentration on CLT panels as well as provide a certain amount of bending  
199 resistance, though bending moment is often neglected in CLT building design due to the high in-plane  
200 stiffness of CLT panels. This kind of edge-supporting connection system, as proved by the previous  
201 numerical analysis on similar interlocking connection [21], can help achieve enhanced integrity in  
202 volumetric structures. When damage happens in the connection units of one of the modules, the entire  
203 structures can remain stable with little movement and require no need for immediate replacement,  
204 because alternative load path can form along the edges of the remained modules, reassuring that the  
205 performance requirements is still satisfied. In addition, located at the exterior surface of modules, the  
206 interlocking connections are invisible from the interior of flats and can be protected from the external  
207 environment by the façade after the completion of the project.



208 Fig. 6. The Illustrations of the working mechanism of the interlocking connection system (a) the modules assembly process  
 209 with the interlocking connection system (b) the sectional elevation of the structure showing the overall constraints provided  
 210 by the integrated interlocking connection system

211 **2.2 Description of the unit elements of the interlocking connectors**

212 Both tensile and shear connections have female connectors and male connectors, and each of them  
 213 consists of repetitive unit patterns as demonstrated in Fig. 7. Their width is being determined according  
 214 to the dimensions of conventional plate connections. The thickness of the bottom plate in all connectors  
 215 was chosen based on the common practice of having steel plate thickness as  $0.5d$  in double shear timber  
 216 joints, to achieve adequate strength and avoid local deformation in the vicinity of screws [36]. The  
 217 female connectors of both connections are groove-like devices, which are designed for accommodating  
 218 the male connectors. The male connector in shear connection is formed by a cantilevering thin-walled  
 219 curved steel band connected to a bottom steel plate via a cubic support at the middle, which is designed  
 220 to deform during movement. In the unit element of the tensile male connector, two symmetric L-shaped  
 221 steel components are connected to the steel plate with a 12mm gap in between for their free inward  
 222 movement when sliding along the sloping walls of the female connector. The width of the gap was  
 223 chosen corresponding to the maximum horizontal movement of the L-shaped components within the  
 224 female connector before reaching the top. Both male connectors are designed to act as the damage  
 225 control devices of the system and to yield first to isolate most of the deformation (without damaging  
 226 other integral parts of the connections) before reaching the target displacement, so the female  
 227 connectors, the timber and the fasteners can remain mostly intact (undeformed). In this way, the male  
 228 connectors become the critical components that determine the strength and ductility in the interlocking  
 229 connection, which ensure more predictable behaviours due to the less scattering properties of steel than  
 230 timber. This can also contribute to a more reliable (finite element) simulation with only basic material  
 231 properties such as compressive strength and tensile strength of timber and steel being used. Moreover,  
 232 as the ductility in this interlocking connection is designed to be achieved in the steel connectors instead  
 233 of the fasteners, large diameter screws ( $\text{Ø}7\text{mm}$ - $\text{Ø}14\text{mm}$ ) are used in this connection system, which as  
 234 suggested in the literature [8] [14] can improve load distribution and help to reduce the risk of in-service  
 235 damage or brittle failure in timber. It is worth noting that in case of changing the damaged parts (units)  
 236 of the connections, only the panels with the male connectors need removing.



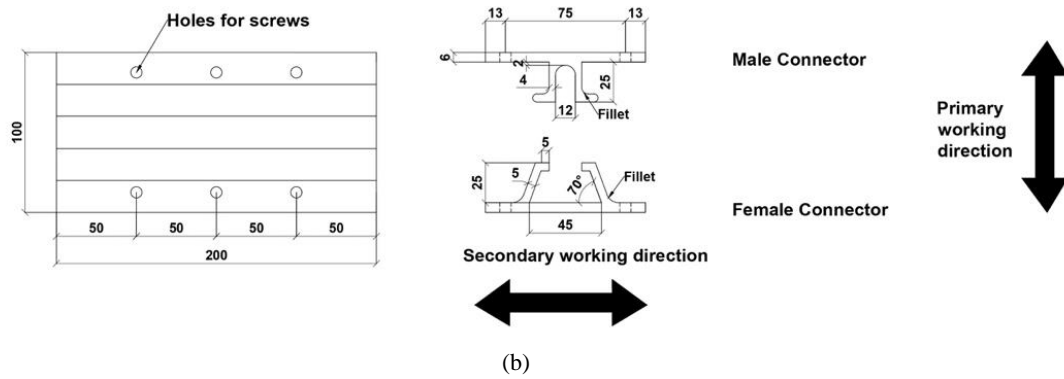


Fig. 7. Schematic the basic units of interlocking connection (in mm): (a) shear connection (b) tensile connection

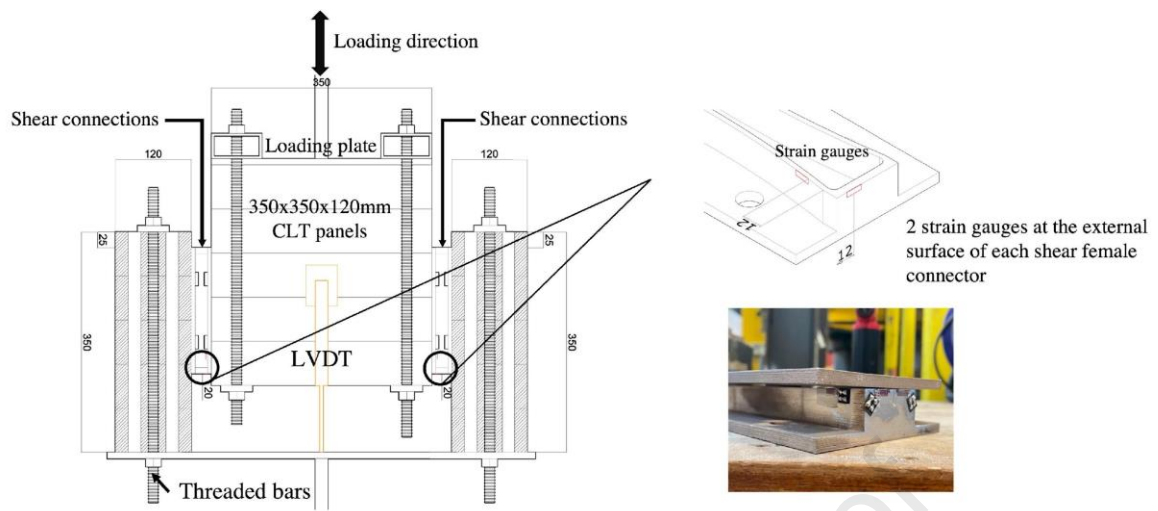
### 3. Experimental Testing

To assess the realistic performance of the proposed connection design and to develop reliable numerical models that can accurately simulate the connection performance, monotonic and cyclic tests were conducted on full-scale shear and tensile connection unit elements.

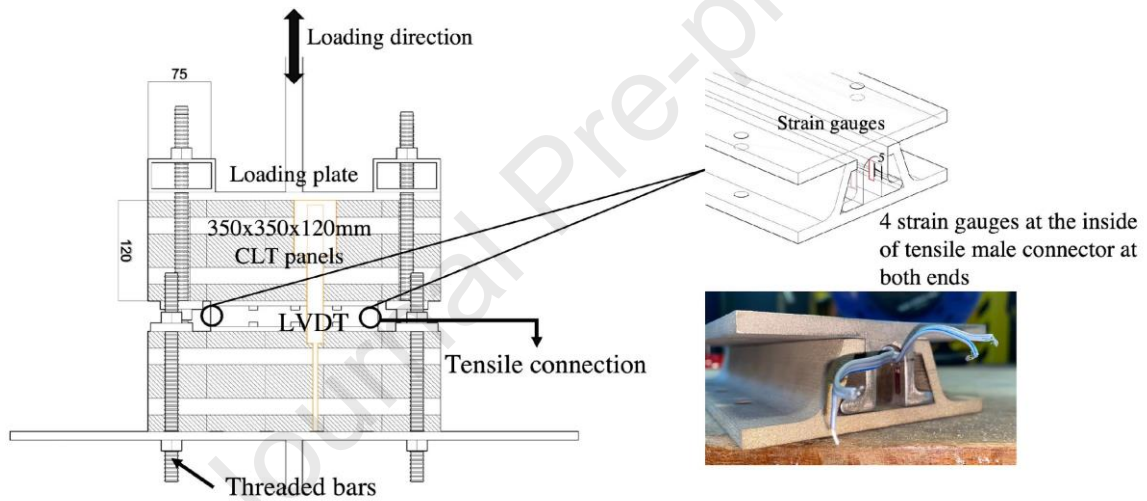
#### 3.1 Test configuration and testing material preparation

Experimental set-ups for shear and tensile connections are shown in Fig. 8. To achieve symmetric test set-up for avoiding a moment being applied to the testing apparatus that may cause instability, two shear connections were tested together with three panels, while each tensile connection was tested individually. Overall, 3 monotonic tests (3 specimens) were conducted on the tensile connection and 1 (2 specimens) on the shear connection under the rate of 0.05mm/s, and 2 cyclic tests (2 specimens) were performed on tensile connection and 1 (2 specimens) on shear connection under the speed of 0.02mm/s. In the quasi-static (cyclic) test, the loading protocol as prescribed in EN12512 [37] was adopted, with the estimated yield point  $V_{est}$  of 4mm for tensile connections and 2mm for shear connections according to the preliminary FE simulations. 4 strain gauges were attached to the specimens in each test with the locations of which being decided considering the preliminary numerical simulations and the accessibility of the assembled connections (Fig. 8).

All connection specimens were connected to 5-ply GL24h 350mmx350mmx120mm CLT panels with density varied between 435-470kg/m<sup>3</sup> provided by Stora Enso using LBS7100 for the tensile connections and HBSP12120 for the shear connections. The locations of specimens on timber panels were all marked before the testing to measure the relative movement between timber and the steel connector. After being screwed to CLT panels, connection specimens were assembled by stacking (shear connection) or sliding (tensile connection) to form the experimental set-up and were tested without additional reinforcement. Mechanical and geometric properties of panels and screws are listed in Table 2 and Table 3. All CLT panels were conditioned in a controlled environment of 20°C with 65% humidity for the week before testing in accordance with EN 1380 [38], and all achieved the moisture content of 10%-10.8%, which were within the required range in the standard (10%-14%).



(a)



(b)



(c)



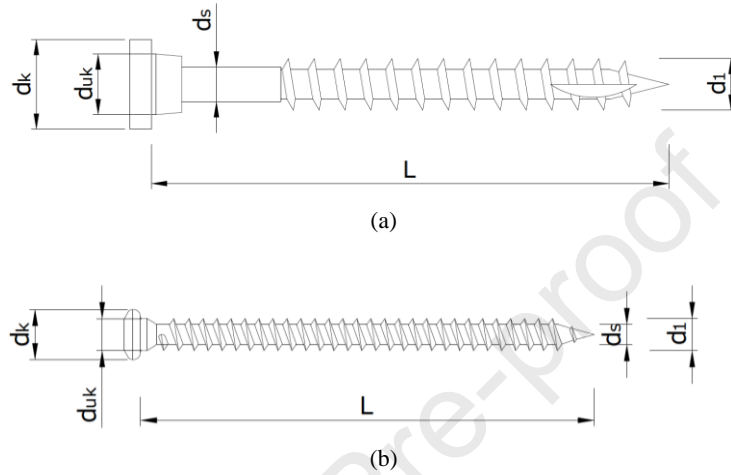
(d)

264 Fig. 8. Experimental set-up for (a)&(c) shear connection and (b)&(d) tensile connection

265 Table 2. Material properties for CLT panels (spruce) [39]

Elastic modulus (MPa)			Poisson's ratio			Shear modulus (MPa)		
$E_{11}$	$E_{22}$	$E_{33}$	$\nu_{12}$	$\nu_{13}$	$\nu_{23}$	$G_{12}$	$G_{13}$	$G_{23}$
11000	370	370	0.48	0.48	0.22	690	690	50
Parallel-to-grain (MPa)		Perpendicular-to-grain (MPa)		Shear strength (MPa)				
$f_{c11}$	$f_{t11}$	$f_{c22}$	$f_{t22}$	$f_v$	$f_{v,roll}$			
36	24	4.3	0.7	6.9	0.5			

266



267

Fig. 9. Dimensions of screws: (a) HBSP12120 (b) LBS7100

268

Table 3. Dimensions of screws used in experiment (in mm)

	L	$d_1$	$d_s$	$d_k$	$d_{uk}$
<b>HBSP12120</b>	120	12	8	20.75	14
<b>LBS7100</b>	100	7	4.4	11	7

269

270 The proposed novel connection specimens were 3D printed. Additive manufacturing was selected for  
 271 its limitless fabrication capabilities, considering the new complex geometry with inclined and curved  
 272 surfaces and the cantilevered steel band (Fig. 10(b)&(d)) of the proposed optimised connection designs.  
 273 For the size of the proposed connections (Fig. 7), a metallic alloy composed of 60% 420 Stainless Steel  
 274 as base material and 40% Bronze for additional strength and resistance was chosen in the printing with  
 275 layer thickness of  $100\mu\text{m}$  using Binder Jetting in Sculpteo [40]. The connection specimens were printed  
 276 along the length of connection. This printing direction was chosen to achieve the most stable printing  
 277 process and minimise the potential deformations in the cantilevering parts. Surface polishing was not  
 278 an option for this size of specimens from the manufacturer, thus the 3D printed connections were all  
 279 finished with granular surface (Fig. 10). To eliminate the risk of unpredictable behaviour due to the  
 280 variation of friction during the movement of male connections within the female connections, the  
 281 contact surfaces were polished to obtain smooth surfaces before testing. Despite the high manufacturing  
 282 accuracy of 3D printing, 1mm tolerance was introduced in the printing of all connection specimens, to  
 283 avoid the potential fitting issues caused by the dimensional distortion, considering the cantilevered  
 284 feature and the scale of printing in this project. In the testing of tensile connections, the remaining gap  
 285 after assembly was eliminated using the built-in function in the loading machine, while it is not  
 286 achievable in the cyclic testing on the shear connections due to the reserve loading. Therefore, small

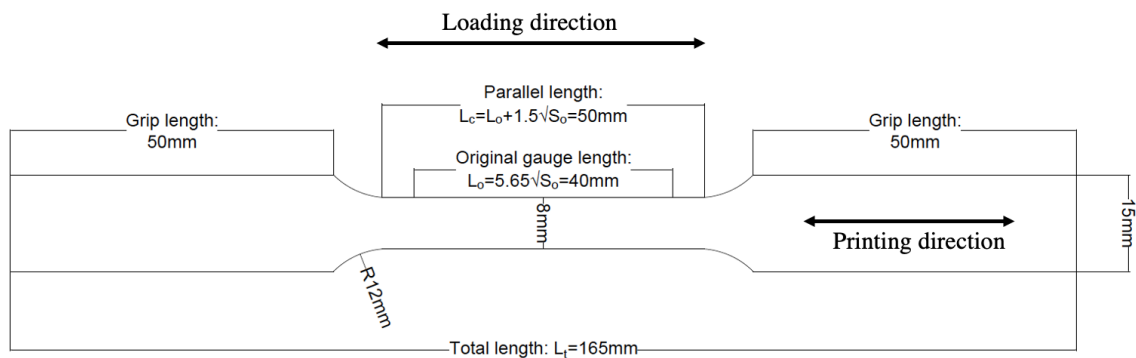
287 amount of sliding with no reaction force can be observed at the beginning of each loading step in the  
 288 cycling testing on the shear connections.



289 Fig. 10. 3D printed connection specimens: (a) shear female connector; (b) shear male connector; (c) tensile female  
 290 connector; (d) tensile male connector

### 291 3.2 Coupon test of 3D printed steel material

292 After the test, tensile coupon tests were undertaken to determine the tensile engineering stress-strain  
 293 properties of the 3D printed SS420/BR in accordance with EN ISO 6892-1 [41] and support future  
 294 numerical validation. Four coupons were machined from the bottom plates of the tested tensile female  
 295 connectors (Fig. 12(a)), which experienced insignificant deformation during the testing. All specimens  
 296 were tested in the direction that parallel to the printing direction under a constant speed of 3 mm/min  
 297 using an Instron testing machine, with strain being measured by an extensometer attached to the  
 298 coupons' surface.



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Fig. 11. Dimensions of the specimens according to EN ISO 6892-1 [41]

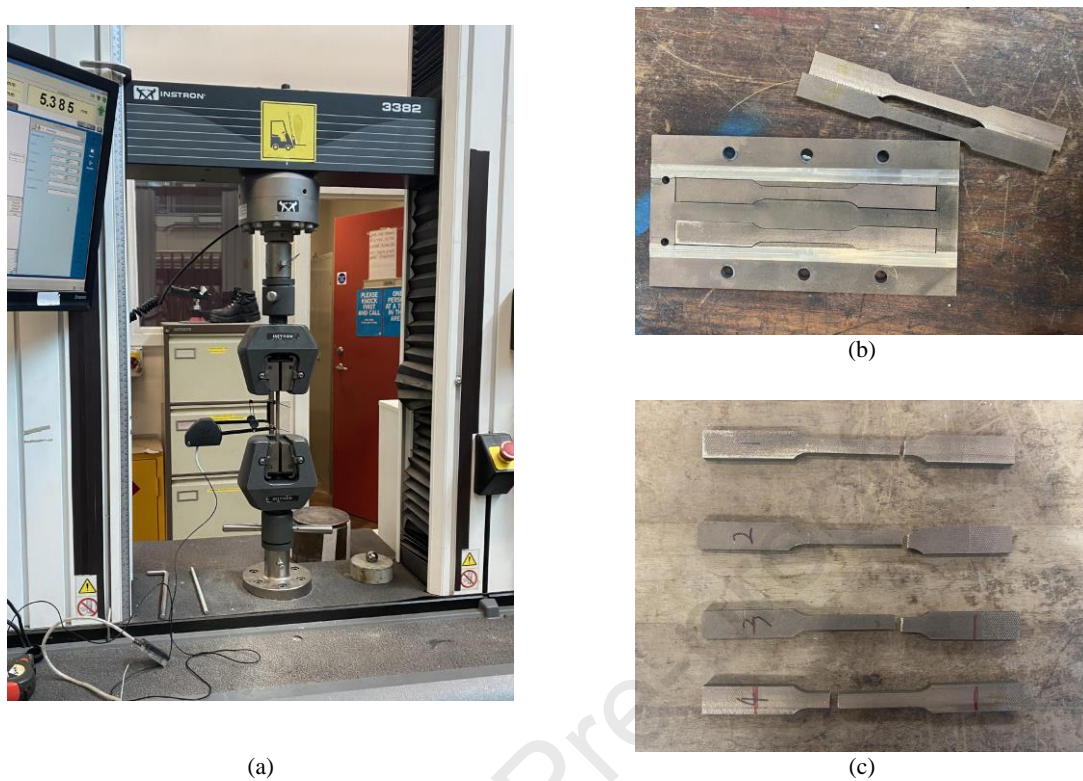
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Fig. 12. Coupon test of SS420 (a) set-up for coupon test (b) machined coupons from the bottom plate of the tested tensile female connectors (c) specimens after testing

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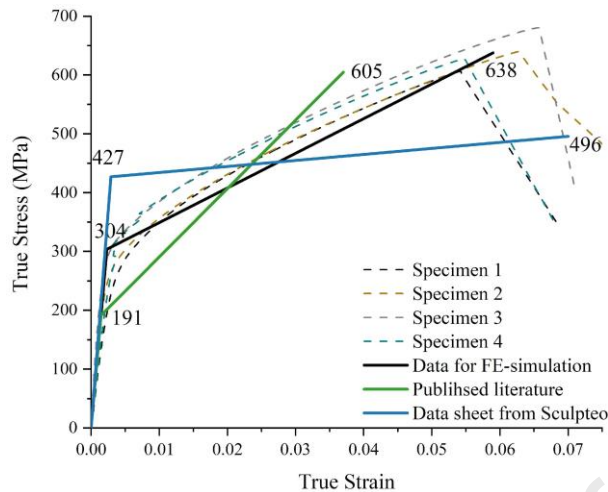
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The original strain and stress data of the coupon tests were converted into true stress and strain, then the yield strength was calculated accordingly using 0.2% offset method. The generated data were relatively consistent with acceptable variation (Fig. 13), so the averaged properties from 4 coupon tests were adopted in the subsequent numerical study. In comparison with the material properties provided by the manufacturer [40] and the published coupon test data of SS420 [42] with the same printing layer thickness and printing direction, the adopted mean values for simulation have similar young's modulus and ultimate strain, while there are variations between the yield strength and the ultimate strength. These can attribute to the sensitivity of 3D printing material to the varied printing parameters such as travel speed and sintering temperature from different manufacturers. It should be noted that, 3D printed material has different behaviours in the directions perpendicular to and parallel to the printing direction. But the variations are only around 2-4% in two printing directions. According to the published experimental data [42], for the same 3D printed material in this research with the same printing layer thickness, the biggest property difference between directions parallel to and perpendicular to the printing direction is 2.1% in the ultimate strength. It can therefore conclude that, this difference would not affect the results significantly, and the material properties obtained in the printing direction of coupon specimen can well represent the material performance on different directions.



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Fig. 13. Experimental data of coupon test and the mean material properties used as input data in simulation

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## 4. Numerical Validation Model

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To validate the numerical models using the experiment output, 3D fully-discretised continuum models of the tensile and shear connections with full details in the unit steel connectors, fasteners and timber were constructed in finite element analysis (FEA) software ABAQUS. Validated modelling methods in publications [43, 44] and nominal or tested material properties was employed in the validation models, which were analysed with ABAQUS/Standard. The dimensions of both connection specimens and the testing set-ups were set to be consistent with the test, as illustrated in Fig. 7, Fig. 8. and Fig. 9.

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### 4.1 Modelling methods

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#### 4.1.1 Materials

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In the models of both tensile and shear connections, the 5-ply CLT panels were modelled using an orthotropic elastoplastic material model with the properties as listed in Table 2. Each layer of panel was considered separately with the first (20mm), third (40mm), and fifth (20mm) layers being loaded parallel to grain and the second (20mm) and fourth (20mm) layers being loaded perpendicular to grain. For all steel fixtures and screws, a modulus of elasticity 200GPa and a Poisson's ratio of 0.3 were assumed. Grade-300PLUS steel with a yield strength of 320MPa and an ultimate strength of 440MPa was used in all steel fixture devices. The Grade 10.9 carbon steel with a yield strength of 940.3MPa and an ultimate strength of 940.3MPa at 0.5% ultimate strain, was adopted for HBSP12120 and LBS7100, which were validated by Tomasi et al. [45]. For the modelling of the 3D printed connectors, a modulus of elasticity of 130GPa, a yield strength of 304MPa, an ultimate strength of 638MPa and an ultimate strain of 5.9 % that extracted from the coupon test were taken. 8-node linear brick with reduced integration (C3D8R) elements were adopted in the mesh throughout the model.

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#### 4.1.2 Modelling of shear connection

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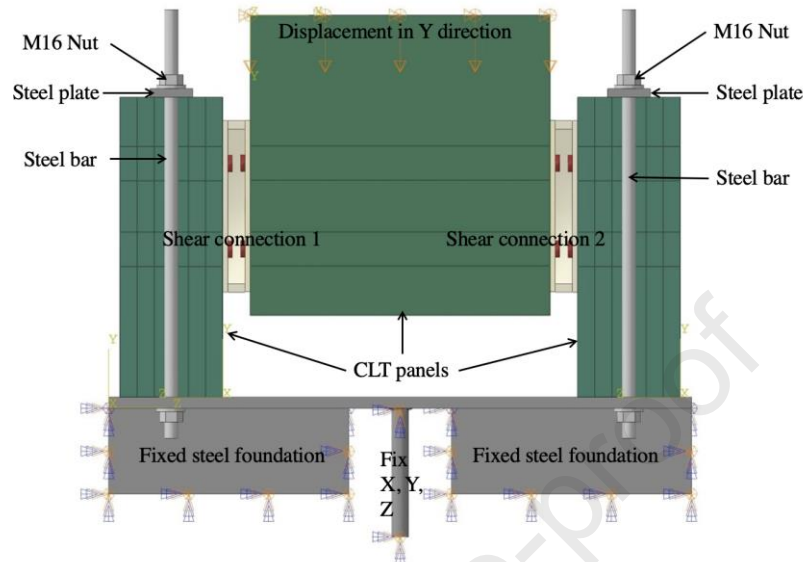
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To better simulate the experiment with sufficient accuracy, the FE model of the shear connection was built according to the experiment set-up (Fig. 14(a)) with full dimensional details of steel fixtures as shown in Fig.8(a) and connections as shown in Fig.7(a) on both sides. The movement restrictions in three directions were applied on the steel foundations representing the support from the loading

347 machine, and on the grip bar connecting the entire test set-up to the loading machine. The middle panel  
 348 with two shear male connectors attached was loaded in a displacement-controlled manner on top in Y-  
 349 direction, simulating the load applied in the experiment.

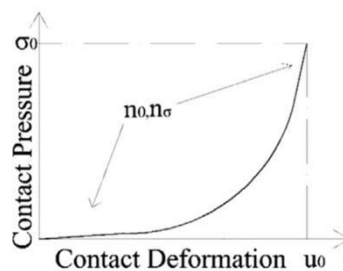


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Fig. 14. The outline and boundary conditions of FE model of the shear connection

352 In the models of timber connections, the interfaces between timber and steel fasteners are crucial to the  
 353 overall accuracy of the simulation. When loaded in shear, the screws in timber connections are mainly  
 354 subjected to compression from the surrounding timber and the steel plate, which is mainly bored by the  
 355 shank part of screws. Consequently, the screws in interlocking shear connection were modelled as  
 356 cylindrical elements with a diameter equals to  $d_s$  in Table 3. The surface-to-surface discretisation  
 357 method was adopted for the contact of all interfaces in the model with the “Hard Contact” option in the  
 358 normal direction, and the “penalty friction formulation” option in the tangential direction, while using  
 359 a coefficient of friction of 0.4 for all steel-steel contacts [43] and 0.25 for all steel-CLT contacts [46].  
 360 Tie constraints were adopted in all interfaces between nuts and threaded bars to simulate the fastening  
 361 effect, as no relative movements was observed during the test. In the tangential direction of the screws-  
 362 timber interfaces, a non-linear relation of contact pressure and displacement (Eq.2) was used in the  
 363 normal direction along with the “Tabular” option in ABAQUS to simulate the weakening effect of pre-  
 364 drilling on the timber around screws, considering the chunky screws used in the shear connections.



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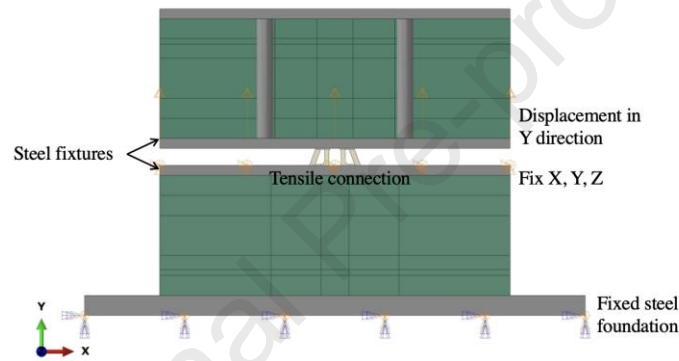
Fig. 15. Function represents the weakened timber around fasteners [43]

$$367 \quad \left(\frac{u_{inter}}{u_0}\right)n_u + \left(\frac{\sigma_0 - \sigma_L}{\sigma_0}\right)n_\sigma = 1 \quad (2)$$

368 Where  $n_u$  and  $n_\sigma$  are the control factors of the function curvature and were taken as 3.9 and 1.1, the  
 369 contact deformation  $u_0$  was taken as 0.35mm, all of which were extracted from the test on  $\emptyset 12$  dowels  
 370 conducted by Dorn [47].  $\sigma_0$  is the maximum compressive strength of timber that was assumed to be  
 371 30MPa [43]. This formula represents the low timber strength around the surface of screws. Once the  
 372 deformed screws reach the unaffected area, the strength of timber will be recovered.

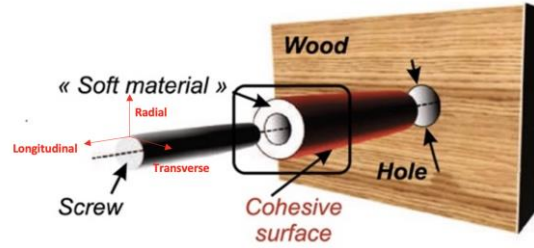
#### 373 4.1.3 Modelling of tensile connection

374 Similar to the shear connection model, the boundary conditions in the tensile connection model were  
 375 set according to the experiment set-up. A simplified steel fixture was modelled for applying the  
 376 movement restrictions as shown in Fig. 16. Displacement-controlled loading in the Y-direction was  
 377 applied on the steel fixture on the panel with the male connector.



378 Fig. 16. The outline and boundary conditions of FE model of the tensile connection

379 When simulating timber screwed connections loaded in tension, two modelling methods are commonly  
 380 adopted. The first one is to build the complete model of the screws including full details of the threaded  
 381 part [48], thus the performance of screws is fully dependent on the geometric characteristics of screws  
 382 and no assumption of screw properties is needed. The second one is a simplified method proposed and  
 383 validated by Avez et al. [44] and Bedon et al. [49], which introduces a fictitious ‘soft material’ (Fig.  
 384 17) to represent the threaded part with the diameter equals to  $d_1$  in Table 3. The ‘soft material’ is  
 385 assumed to be perfectly elastic and has the same capacity as the wood, except for the radial modulus  
 386 that is reduced to 50MPa to eliminate its contribution in the compressive direction, representing the  
 387 weakening effect of threads on the timber. Wrapped by ‘soft material’ is the ‘core’ that simulates the  
 388 shank of screws, the dimensional value of which is taken as  $d_s$  of LBS7100 (Table 3). Compared to the  
 389 first method, the second method is more computationally efficient due to the elimination of the small  
 390 details of the threads and the large contacts area between the threads and timber, thus it is adopted in  
 391 this study.



392

393

Fig. 17. Simplified FE model for screws in tension [38]

394 The withdrawal capacity of screws was simulated by a cohesive surface between the internal surface of  
 395 drilled holes in the timber panel and the external surface of the ‘soft material’ with an assumed rigidity  
 396 of  $40\text{N/mm}^3$  in the longitudinal direction and  $0\text{N/mm}^3$  in the tangential and radial directions (Fig. 17)  
 397 [38]. The maximum nominal stress (MAXS) approach was applied at the cohesive surface in the  
 398 ‘damage initiation criterion’ option to simulate the initiation of interaction failure:

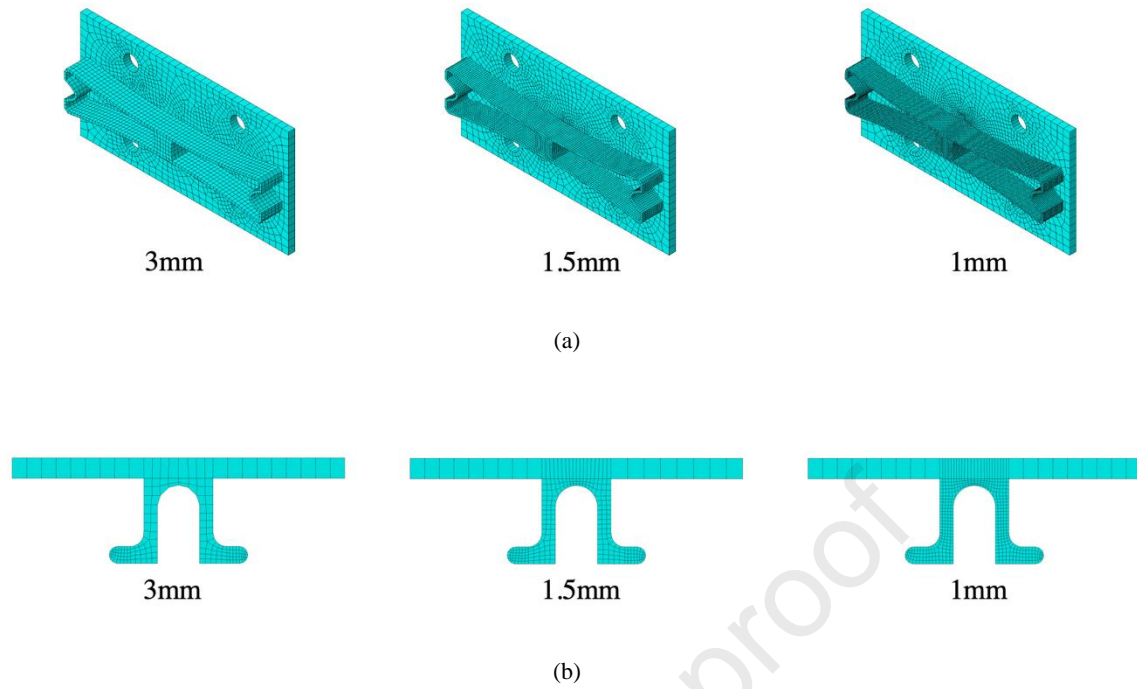
$$399 \quad \max\left\{\frac{t_n}{t_n^0}, \frac{t_s}{t_s^0}, \frac{t_t}{t_t^0}\right\} = 1 \quad (3)$$

400 where the  $t_n^0$ ,  $t_s^0$  and  $t_t^0$  are the peak allowable stresses in the normal (n), first (s) and second (t)  
 401 direction of the bonding interface, the values of which were taken as 36MPa, 6.9MPa and 6.9MPa,  
 402 equalling to the compressive and shear strength of timber as listed in Table 2.

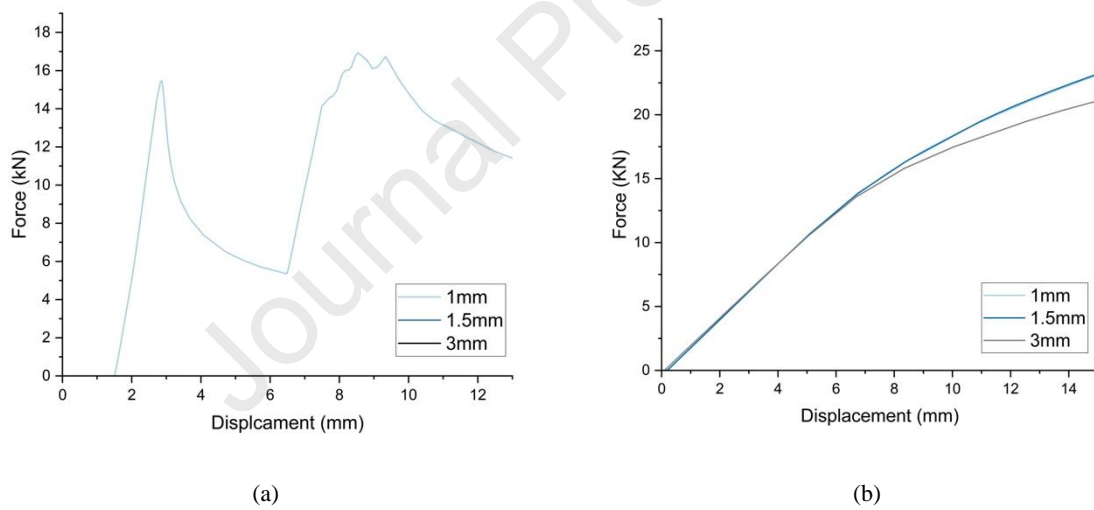
403 After the cohesive resistance is reached, the strength degradation of bonding begins and the brittle post-  
 404 damage behaviour of the connection under tension interaction was simulated by a ‘linear damage  
 405 evolution’ law, assuming that the full residual stiffness of the cohesive surface is achieved when 4mm  
 406 deformation first attained, so the contact behaviour became elastic-brittle, simulating the brittle failure  
 407 of dowel-type timber connection under tension. The cohesive interaction was combined with “Hard  
 408 Contact” in the normal direction and the “Penalty Friction Formulation” with a coefficient of friction  
 409 of 0.4 in the tangential direction to avoid the penetration of screws into timber at the post-failure stage.

#### 410 4.1.4 Mesh sensitivity

411 To study the impact of mesh size on the FE predictions and ensure the adequate adoption of mesh size,  
 412 the mesh sensitivity analysis was conducted. The timber panels, fasteners, metal fixtures and the female  
 413 connectors, which experienced insignificantly deformation in the test, were proved to be independent  
 414 to the mesh size. Therefore, increasingly smaller mesh size was only applied in the interested areas of  
 415 both connection models (Fig.18), while the mesh size in other parts of the testing set-up remained  
 416 consistent. The male connectors of both connections, which are designed to govern connection  
 417 properties, were first studied with 3mm mesh size. Then the mesh size was refined to 1.5mm and 1mm.  
 418 It can be observed in Fig.19(a) that, the changes in mesh size have little impacts on the numerical  
 419 predictions of shear connector model. In the model of tensile male connector (Fig.19(b)), the initial  
 420 stiffness is independent to the mesh size, while yielding strength is slightly sensitive. The 1.5 mm mesh  
 421 size showed a good compromise between results accuracy and the calculation efficiency in both models.  
 422 It was therefore chosen in the subsequent study.



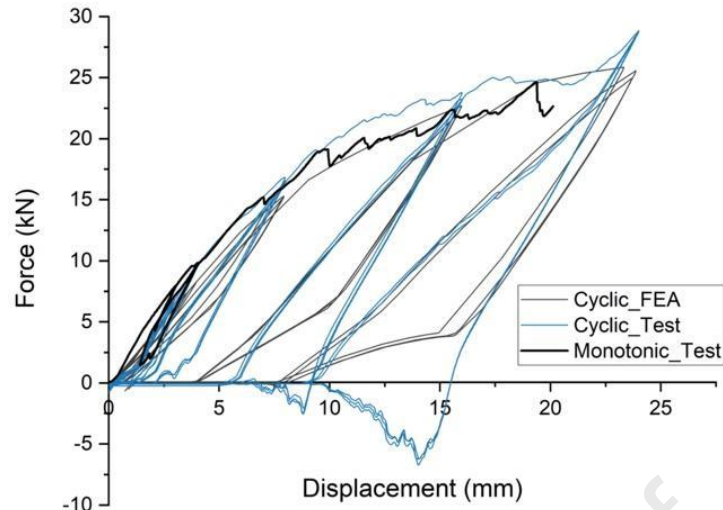
423 Fig. 18. Mesh size considered in the mesh sensitivity study of (a) shear male connector (b) tensile male connector



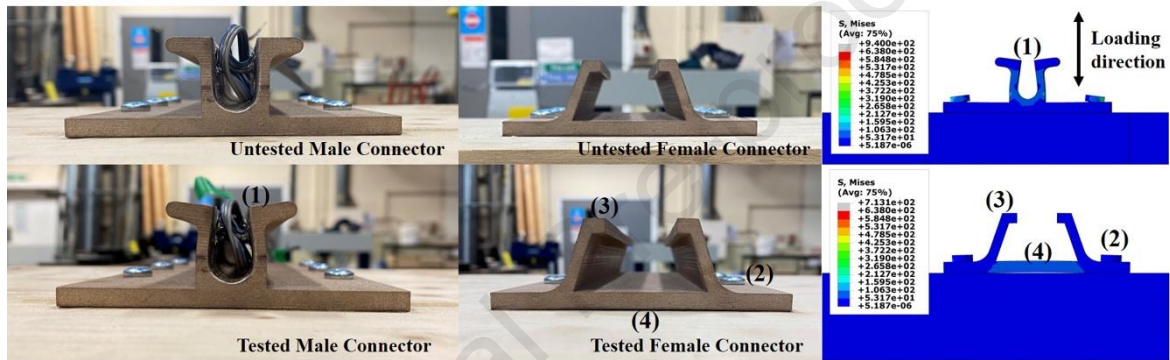
424 Fig. 19. Force-displacement response obtained from different mesh size in (a) shear male connector model and (b) tensile male  
425 connector model

## 426 4.2 Numerical Validation

### 427 4.2.1 Tensile connection



(a)



(b)

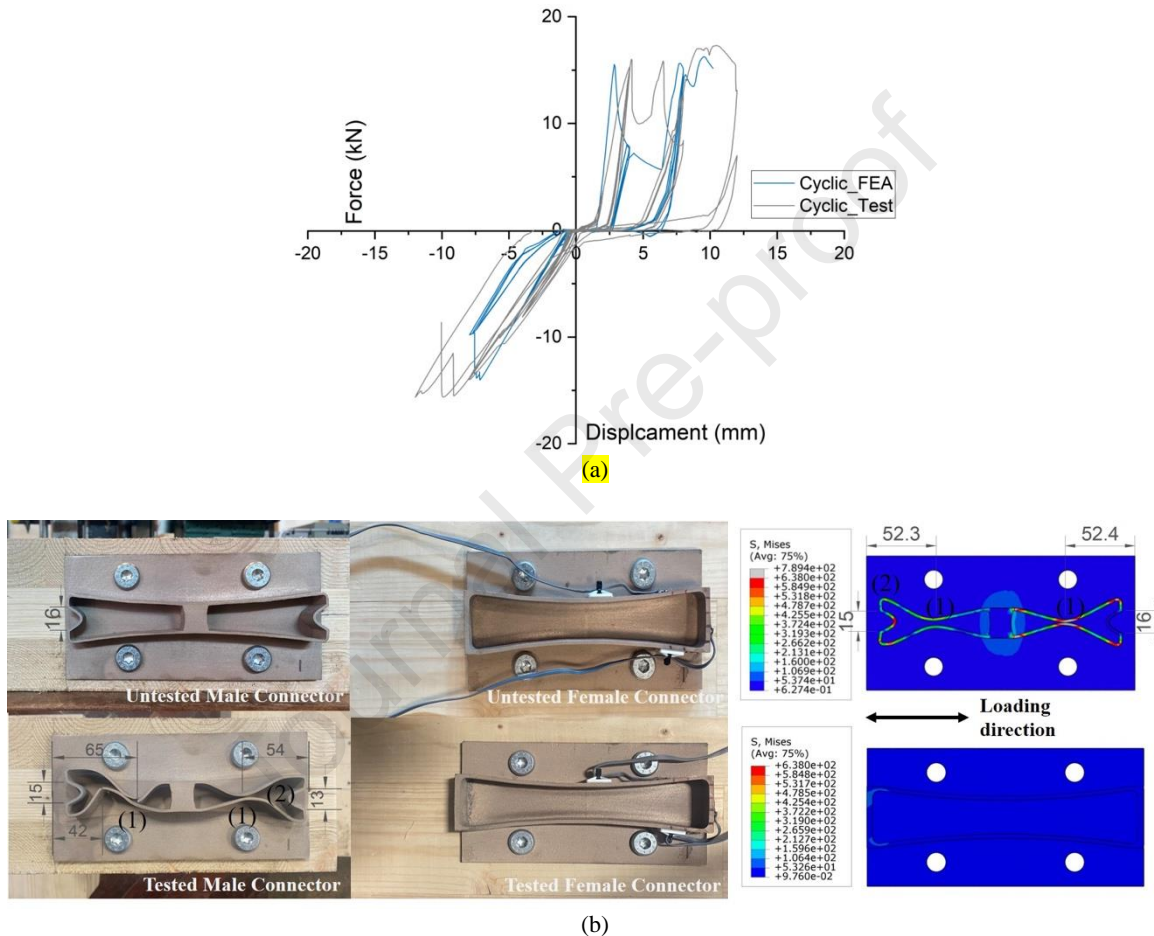
428 Fig. 20. Comparisons between experimental result and FEA in (a) hysteresis loops and (b) the deformation modes in tensile  
 429 connection specimens (in mm)

430 Fig. 20(a) shows the hysteresis loops for the proposed tensile connections. Different from conventional  
 431 timber connections, the connection performance in the first cycle of each loading step in cyclic test  
 432 showed good consistency with the monotonic test, as stiffness and maximum strength in monotonic  
 433 loading at each step can be reached in cyclic loading with no strength degradation, indicating the plastic  
 434 deformation in the steel male connector does not affect the maximum capacity in the subsequent  
 435 loading. However, the plastic deformation caused gaps between two connectors, leading to the sliding  
 436 of the male connector within the female connector with no reaction force. With the increase in applied  
 437 displacement, the bearing walls of the female connector started to open-up and close elastically, along  
 438 with the upward and downward movement of the male connector. This indicated that the bearing walls  
 439 also contributed to the connection stiffness under large displacement, which led to the degraded stiffness  
 440 degradation of each repeated circle, as similar to the pinching effect in the conventional timber  
 441 connections.

442 The primary deformation mode in the tensile connection was the bending of the L-shaped elements in  
 443 the male connector as designed (point 1 in Fig. 20(b)). The female connector and screws on the other  
 444 hand, remained mostly undeformed (point 2 in Fig. 20(b)), and only very slight open-up in the bearing  
 445 walls (point 3 in Fig. 20(b)) and slight bending in the bottom plate (point 4 in Fig. 20(b)) were visible  
 446 at the end of testing, so the deformed male connector can be easily slide out.

447 The FE model of tensile connection has good agreement with the experimental output, as the maximum  
 448 strength of each loading cycle, the initial and reduced stiffness can all be well captured. Four  
 449 deformation modes in the tensile connection as discussed above were also well reproduced in terms of  
 450 shape and dimension, proving the feasibility of the proposed modelling method for interlocking tensile  
 451 connection. Though slight pulling out can be observed from the screws in male connector in the model,  
 452 the continuous force-displacement curve indicated that the screws-timber interfaces still remained  
 453 elastic, as the ‘linear damage evolution’ in FE model was not activated.

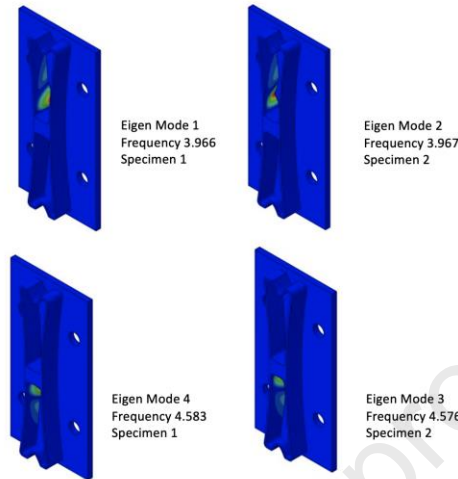
#### 454 4.2.2 Shear connection



455 Fig. 21. Comparisons between experimental result and FEA in (a) hysteresis loops and (b) the deformation modes in shear  
 456 connection specimens at -8mm displacement (in:mm)

457 In the cyclic testing of shear connections (Fig. 21(a)), a sudden drop in force can be observed when first  
 458 reaching the displacement of 4mm, which was followed by another one at the displacement of 6.5mm  
 459 under the similar forces (around 16kN). These indicated the buckling happened in the shear connections  
 460 on both sides (point 1 in Fig. 21(b)), which caused instability in the experimental set-up. After the  
 461 buckling, the connection strength continued to develop in the deformed steel band and the end sunken  
 462 (point 2 in Fig. 21(b)), until the breakage happened at the buckling point of one shear connection (Fig.  
 463 21(b)), leading to the plumper hysteresis loop at large displacement. Therefore, two major deformation  
 464 modes can be identified in the tested shear connections after the testing: buckling at the middle of the  
 465 deformable band and bending at the sunken design at the end of the steel band. As shown in Fig. 21(a),  
 466 the buckling and plastic deformations in the shear connection specimens caused unsymmetric

467 behaviours in two loading directions, which are unfavourable considering the stability of structures and  
 468 limit the connection's capacity in dissipating energy. The buckling locations were relatively consistent  
 469 among all the specimens, while unsymmetric buckling shapes can be observed in some specimens due  
 470 to manufacturing imperfections and the post-buckling loading on the buckled steel elements.



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Fig. 22. Four Buckling Modes Extracted by 'Block Lanczos' Method

473 As buckling was observed in the shear connection, geometrically and materially nonlinear analysis with  
 474 imperfections included (GMNIA) was conducted in the shear connection validation model. The linear  
 475 Eigen-buckling analysis, which is a common method for predicting the buckling strength and generating  
 476 'imperfection' on the model to trigger nonlinear buckling analysis, was first conducted. In the Eigen  
 477 buckling analysis, a unit pressure was applied at the mid-span to predict the critical load of buckling  
 478 and the buckling mode shapes of the ideal structure based on the 'Block Lanczos' method. As shown  
 479 in Fig. 22, the generated four buckling modes in the male connectors on both sides agreed qualitatively  
 480 with the test results, so they were extracted and imported into the nonlinear analysis as 'imperfection'  
 481 with a factor of 0.2. As the primary deformation form in the tensile male connector was bending instead  
 482 of buckling, GMNIA was not considered in the tensile connector model.

483 With the inclusion of 1mm gap in the shear connection model, sliding in the initial stage of each loading  
 484 step that corresponding to the experiment can be observed in the numerical output. The model with the  
 485 initial imperfections was able to simulate the negative stiffness of buckling. The buckling shapes on  
 486 both sides as well as the bending at the end sunken were also well reproduced in terms of locations and  
 487 dimensions, which proves the adequate accuracy of the validation model.

### 488 4.3 Conclusions

489 The results of the monotonic and cyclic tests on the shear and tensile connections demonstrated that,  
 490 two connections behaved very differently in terms of mechanical properties and deformation forms.  
 491 The tensile connection had lower stiffness but higher maximum strength and better ductility with a  
 492 ductile failure mode (due to bending), while the shear connection showed higher stiffness but lower  
 493 maximum strength and ductility with a brittle failure mode (due to buckling). In both connections, there  
 494 was no sign of pulling out in screws and no visible relative movement between the connectors and  
 495 timber (Fig. 23) after the attainment of the maximum strength. These indicated that the deformation  
 496 was successfully managed within the male connectors in both shear and tensile connections, with very  
 497 small deformation developed in the female connector but without any brittle breakage in timber. This  
 498 is a clear demonstration of the ability of connections to localise plastic deformation within the male  
 499 connectors under the control damage approach.



Fig. 23. Removal of screws from (a) shear connections and (b) tensile connection after test

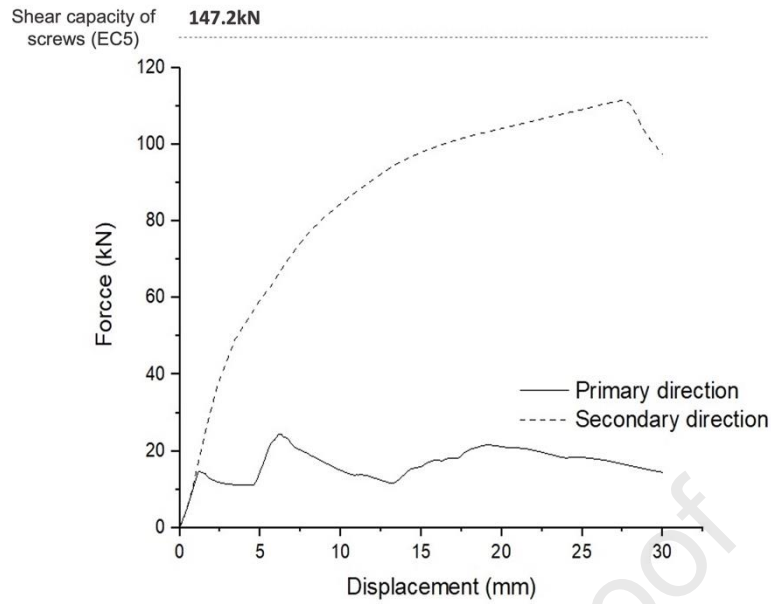
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## 501 5. Numerical Simulations

502 To further characterise the translational behaviours of the proposed shear and tensile connections and  
 503 explore their potential with more common and ductile steel material (S235), a monotonic analysis was  
 504 performed in the validated models at the primary and the secondary working directions (Fig. 7) of both  
 505 connections, with material properties of S235 (modulus of elasticity of 210GPa, yield strength of  
 506 235MPa and ultimate strength of 360MPa).

### 507 5.1 Translational behaviours of the shear connection

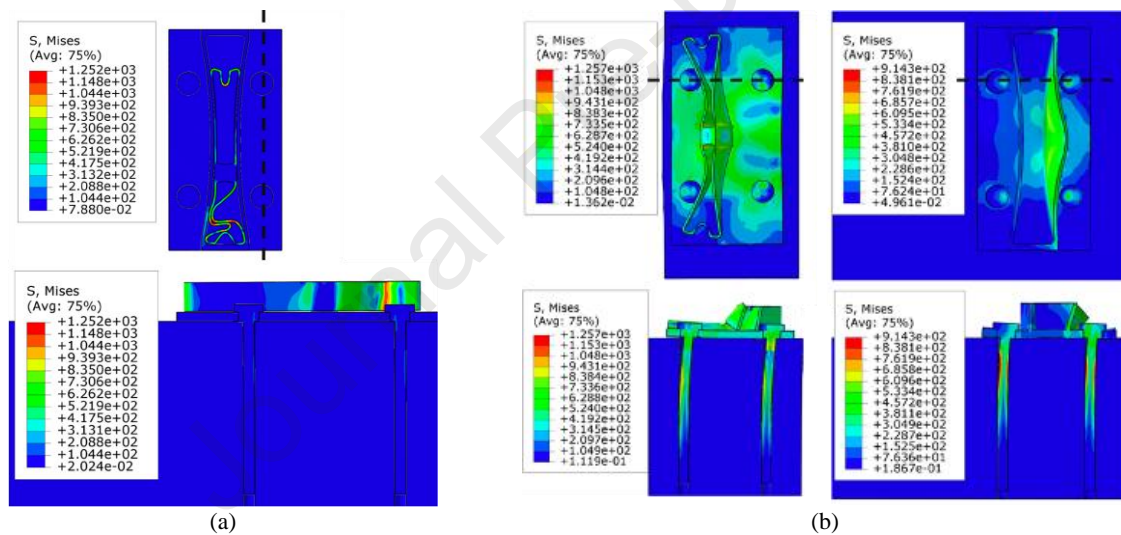
508 The force-displacement responses of the interlocking shear connection FE model in the primary and  
 509 secondary directions are reported in Fig. 22. The numerical model can also monitor the deformation  
 510 developing trends in the shear connections to help better understand the connection behaviours, which  
 511 was not available in the test due to the concealed design nature of the shear connection. Similar to the  
 512 tested specimens, the shear connection with S235 first buckled at the cantilevered steel band, resulting  
 513 in negative stiffness in the FE model. With the continuous loading, the buckled steel band continued to  
 514 bend and the deformation started to develop in the end sucken on the other side. Corresponding to the  
 515 experiments, most of the plasticity was observed in the designated deformable band (Fig. 25(a)) in the  
 516 male connector after a 30mm displacement in the primary direction, while litter plastic strain was  
 517 developed in the female connector and fasteners, which also resulted in very small damage on timber  
 518 panels.



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Fig. 24. Force and displacement curve of interlocking shear connection



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Fig. 25. Stress distribution on shear connections and screws under the displacement of 30mm in (a) the primary direction and (b) the secondary direction

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When working in the secondary direction, the interlocking shear connection showed much higher strength and ductility than that in the primary direction, but the process of plastic deformation was mainly managed by the bending of the middle cubic support in the male connector and the bending of the wall in the female connector at the loaded side. The increased plastic strain in the screws proved that the fasteners also contributed to the connection strength when working in the secondary direction.

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## 5.2 Translational behaviours of the tensile connection

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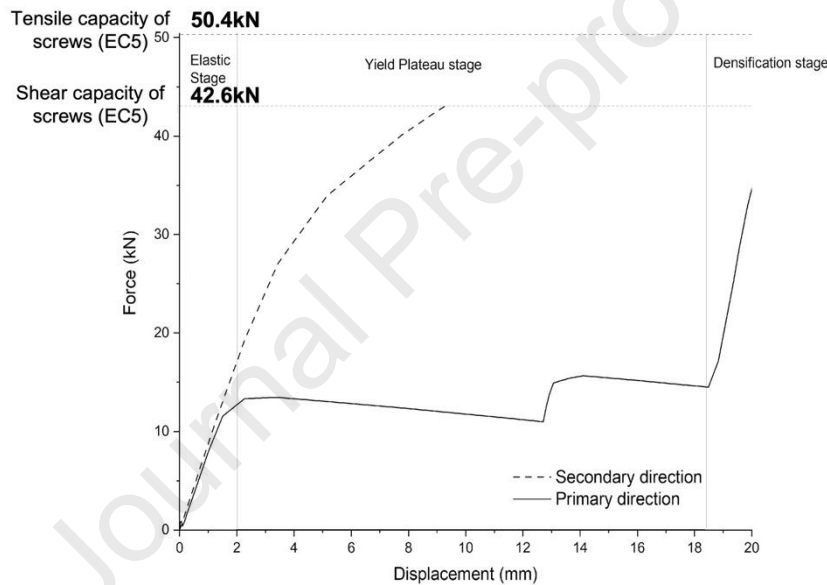
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As demonstrated in Fig. 24, the behaviour of the interlocking tensile connection with S235 in the primary working direction can be classified into three stages: the elastic stage, the yield plateau stage, and the densification stage. When the loading started, the male connector started to move upwards within the female connector, and the sloping walls of which compressed the L-shaped elements to bend

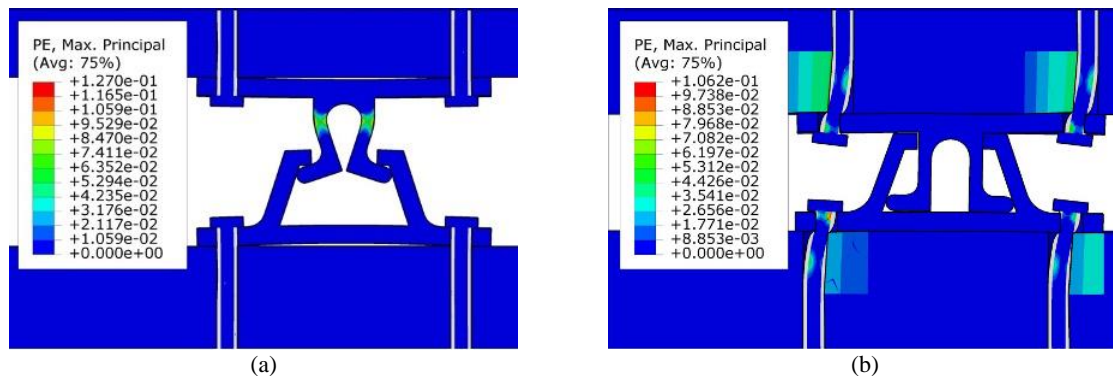
533 inwards. With the continuous vertical movement of the male connector, the L-shaped elements yielded  
 534 and the walls of the female connector started to open up slowly due to the increased moment at the wall  
 535 base, resulting in slightly decreased reaction force of the connection at the yield plateau stage. When  
 536 the male connector came into contact with the top of the female connector, it stopped moving and the  
 537 stiffness of the connection increased significantly, indicating that the behaviour of connection enters  
 538 the densification stage, in which the connection becomes rigid to prevent further displacement  
 539 (opening). Thus, the  $F-\delta$  curve of tensile connection in the primary working direction can be idealised  
 540 using three parameters: the initial stiffness, the yielding strength and the displacement at the onset of  
 541 densification. Fig. 27 shows that, at the displacement of 15mm in the primary working direction, the L-  
 542 shaped elements in the male connection processed most of the plastic deformation, while little can be  
 543 observed in other connection components. Though the stress of the screws under tension cannot be  
 544 reflected directly from the stress contour due to the employment of fictitious elastic 'soft material', no  
 545 strength reduction can be observed in the force-displacement curve, indicating that brittle failure did  
 546 not appear in screws at 20mm slip and the screws were still in the elastic stage.



547

548

Fig. 26. Force and displacement curve of interlocking tensile connection



549

550

Fig. 27. Von Mises Stress contour in the interlocking tensile connections under the displacement of 15mm in (a) the primary direction and (b) the secondary direction

551 When loaded in the secondary working direction, the plastic strain contour plot depicts that the  
552 interlocking tensile connection developed a similar working mechanism to the conventional steel plate  
553 connections, in which the bending of screws and the crushing of timber are the primary deformation  
554 modes. The embedment strength of timber and the bending strength of screws therefore became the  
555 main contributors of the connection strength, while the interlocking steel connectors were relatively  
556 rigid.

## 557 **6. Discussions and Limitations**

558 The FE models of both interlocking shear and tensile connections with S235 demonstrated that the  
559 proposed connection designs can provide adequate strength and ductility under translational forces.  
560 When working in the primary direction, the male connectors in both connections yielded before  
561 reaching the full capacity of the surrounding fasteners, so most of the plastic deformation is managed  
562 within the designated areas (male connectors), while the female connectors, fasteners and timber remain  
563 mostly intact. This indicates that the dissipating element in the novel interlocking connection shifts  
564 from the fasteners like in conventional connections to the steel elements in the new connection. With  
565 the specially designed deformable steel elements, this connection system can absorb energy in  
566 predictable manners, preventing plastic deformation from developing in fasteners and timber to avoid  
567 brittle failure and reducing the risk of significant failure of structures under extraordinary loads such as  
568 seismic load. In addition, the initial deformation of the steel male connectors can be monitored and act  
569 as an early warning system since they can be relatively easy to access. This also means that only the  
570 male connector would fail after a severe event and needs to be repaired or replaced, while the screws  
571 connecting the unit and the panel will remain intact and thus the timber fibres will not be affected by  
572 the bending of the screws as it is normally the case. It therefore helps to improve the structural integrity  
573 and to reduce the time needed for maintenance. When damage happens in the connection units, the  
574 damaged module can be removed following the installation sequence for the repair or replacement. Due  
575 to the short unit length of connection (same to the steel specimen dimensions), rapid and cost-effective  
576 supply can be achieved with 3D printing of the male connector. Also, the timber panels can be fully  
577 reused since the proposed novel connection system promotes no fastener failure.

578 In the secondary working direction, the deformation in the connection is less ductile and managed by  
579 the composite effect between the fasteners and the steel connector, which is similar to the working  
580 mechanism of the conventional steel plate connections and is characterised by its high stiffness but  
581 relative lower ductility than in the primary working direction. When comparing the deformation forms  
582 in two working directions, the significantly reduced deformation in screws in the primary direction  
583 indicates the successful damage control in connections through the introduction of the proposed  
584 connector design.

585 Different from other novel connections for CLT volumetric structures, the one proposed in this study  
586 requires no modifications for fitting, thus is applicable to different flat module specifications. The  
587 equipped damage-limiting capacity also makes it superior to the conventional connection in terms of  
588 life-safety performance and material reuse after the end-of-life of structures. The proposed connection  
589 system holds the potential of improving the dismantlability in CLT volumetric structures, promoting a  
590 more flexible volumetric construction with changeable, demountable and fully reusable structural  
591 elements.

592 Despite the superiorities, the immediate practical application of the proposed connection system may  
 593 be limited by the existing construction tools. The sliding and stacking of modules require specially-  
 594 designed lifting and moving machines and scaffolds frames. Also, due to the geometric complexity, the  
 595 mass production of the proposed connections is difficult to be achieved with the conventional  
 596 manufacturing methods. However, with the gradually increasing manufacturing scale of 3D printing  
 597 and the reducing printing cost, the mass production is expected to be achievable with 3D printing in the  
 598 future.

## 599 **7. Conclusions**

600 In this paper, a comprehensive review of the recent advancements in the study of connection systems  
 601 for CLT volumetric structures is first provided. The factors that relate to connection systems and  
 602 constrain the development of multi-storey CLT volumetric structures have been analysed and  
 603 summarised. As a result, a novel interlocking connection system is then proposed for addressing the  
 604 low construction efficiency, inaccessibility of inter-module connections and the insufficient ductility of  
 605 typical timber connections in VTC. The designed ability of managing deformations within one part of  
 606 the connection and eliminating damage in timber panels in the primary working direction was proved  
 607 via experiments and numerical analyses on the 3D printed connection specimens. Numerical models  
 608 with good accuracy were proposed for both connections using only nominal material properties, owing  
 609 to the fact that the load-carrying capacity and ductility are correlated to the proposed interlocking metal  
 610 connectors. Ultimately, the adequate translational strength and ductility of the proposed connections  
 611 were proved by the validated numerical models.

612 Future research will include a comprehensive parametric study to identify the critical parameters of the  
 613 proposed connection systems and their detailed mechanical properties, as well as conduct a direct  
 614 comparative study between typical connections and the novel connection system via global models of  
 615 a full-scale CLT volumetric building. The study will be centred to the contribution of the connections  
 616 to the overall lateral resistance of the structure.

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## Highlights

- Detailed review of connection systems for Cross Laminated Timber volumetric structures.
- Development of an interlocking connection system for CLT volumetric structures.
- Finite element analyses on the translational behaviours of the proposed connection system.
- Monotonic and cyclic testing on 3D printed specimens demonstrates damage-control capacity of the proposed design.

Journal Pre-proof

**Declaration of interests**

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

The authors declare the following financial interests/personal relationships which may be considered as potential competing interests:

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