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Experimental investigation on novel shear connections for demountable steel-timber composite (STC) beams and flooring systems

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A R T I C L E I N F O

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A B S T R A C T

To successfully facilitate the construction sector's transition toward sustainability, there is an urgent need for alternatives to the existing practices which are not sustainable. Steel-timber composite (STC) beams, which combine a down-standing steel beam and a timber slab for flooring systems, show large potential as sustainable alternative to conventional steel-concrete composite systems. In STC systems the shear connection is a key element that keeps the elements together and ensures effective composite action. This study presents a comprehensive experimental assessment of novel shear connections for demountable STC beams and flooring systems. The research focused on three newly developed bolted shear connector types: SCT-1, SCT-2, and SCT-3, each incorporating a different shear connection device designed to protect the timber and enable the attainment of the required preload for high-strength bolts. These novel shear connections represent a robust and sustainable alternative to conventional connections, offering superior protection to structural elements. The connections were tested in a double-symmetric push-out test setup, implementing laminated veneer lumber (LVL) plates connected to HEB steel profiles. Their performance was assessed in terms of their load-slip responses, stiffness, and failure mode. The load-slip responses were found to be nonlinear, and the connectors exhibited a significant deformation capacity (greater than 40 mm slip). The results of this study indicate load-bearing capacities per shear connector at a 6 mm slip of 95.7 kN, 104.4 kN, and 120.2 kN for SCT-1, SCT-2, and SCT-3, respectively. Additionally, the maximum loads per shear connector were recorded as 161.4 kN for SCT-1, 173.1 kN for SCT-2, and 163.8 kN for SCT-3. Furthermore, the shear connections introduced in this study offer ease of installation and facilitate the assembly and disassembly of components.

1. Introduction

The construction sector stands as a major contributor to resources depletion and greenhouse gases emissions, exerting a substantial environmental impact. Construction processes involve the extraction and utilization of natural resources, while the generation of construction and demolition waste (CDW) further compounds the environmental burden. This sector alone consumes over half of the world's raw resources, accounts for more than a third of global energy consumption, contributes 11% to global CO_2 emissions, and generates nearly 50% of the global waste stream [1,2]. In the European Union (EU) in 2018, CDW comprised a substantial 36% of the total waste generated, making it the largest waste stream among all sectors [3].

In response to the growing awareness of the detrimental environmental effects of current practices across various sectors, numerous programs and initiatives have emerged to steer activities toward sustainability. Notably, the European Green Deal [4], has set the ambitious goal of making the EU's economy climate-neutral by 2050, while the United Nations' Sustainable Development Goals [5] encompass targets related to sustainable consumption and production.

In this context, the construction sector is in critical need of sustainable alternatives to minimize its environmental footprint. Solutions range from the use of more sustainable materials to recycling practices and the implementation of circular economy principles. Among structural solutions, steel-timber composite (STC) beams, combining a down-standing steel beam and a timber slab for flooring systems, emerge as a sustainable alternative to conventional steel-concrete composite (SCC) systems. Engineered timber products utilized in STC beams have the potential to sequester carbon, reducing the overall carbon footprint of buildings. Furthermore, the suitability of timber for prefabrication and serial production offers the prospect of shorter construction times and decreased greenhouse gas emissions.

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Shear connections constitute a critical component of composite structures, ensuring the integrity and effectiveness of composite action. Due to the growing interest in STC beams and flooring systems, research examining the behaviour of shear connections for steel-timber flooring systems has expanded in recent years.

Asiz and Smith [6] performed push-out tests of CLT panels connected to steel beams by means of screws to determine their load-slip response. The results showed that the load-slip response was influenced by the orientation of the panel and the length of the connector. Larger ultimate loads and stiffness were observed for longer connectors and when the grain of the thicker layers of the CLT plates were aligned with the direction of the load.

Loss et al. [7,8] investigated 20 steel-to-CLT connections with the aim to use them for modular flooring systems fabricated with CLT panels and cold-formed steel beams. The connections investigated included connections with mechanical devices only (e.g. screws, threaded rods, geka connectors) and connections with both mechanical devices and epoxy-based resin, in which the epoxy-resin was used as a filler (e.g. rods or plates embedded in timber and filled with epoxy-resin). Most connections exhibited large deformations and none of them had a brittle failure before reaching a deformation of at least 10 mm. Concerning stiffness, peak load, and deformation capacity the values varied within a broad range for all the connections, however, the connections implementing steel plates embedded in the timber and filled with epoxy resin showed the largest peak forces and stiffnesses, followed by threaded rods reinforced either with geka connectors or with screws, and the least performing connections were those using screws only.

Hassanieh et al. [9–13] characterised the load-slip behaviour and failure modes of steel-CLT and steel-LVL connections with high-strength bolts, coach screws, a combination of glue and coach screws, screws with punched metal plates, and high strength bolts embedded in pockets of grout. The parameters analysed in the experiments, which have influence on the stiffness and the load-bearing response of the connections were mainly: (i) the size and strength of the screws and bolts, (ii) the orientation of the grain with respect to the load direction, (iii) the use of adhesives at the steel-timber interface, (iv) the reinforcing nail plate and, (v) size of the grout pocket. In general, the connections tested showed large deformation capacities. However, when adhesives were used at the steel-timber interface although the peak load increased when compared to non-glued connections, the deformation capacity was significantly reduced, hence, the failure turned out to be brittle. In addition, among all the connectors tested, the bolted connections in grout pockets exhibited the largest peak loads.

Cyclic loading tests on steel-to-CLT connections for floors have been performed by Ataei et al. [14,15] to investigate the failure mode, ductility, energy dissipation capacity, equivalent viscous damping and strength impairment. The connections with screws and bolts previously assessed by Hassanieh et al. through monotonic push-out tests were tested in this study under a low-cycle high-amplitude loading regime. The parameters analysed include the size and type of the connectors (i.e. coach screws, dog screws and bolts) as well as the orientation of the grain of the CLT panels with respect to the load direction. According to the results, these connections have high ductility and energy dissipating capacity, most connections were able to sustain slip values larger than those required by EN-1998–1. Furthermore, a hysteretic analytical model was proposed for these connections and calibrated against test results.

The long-term behaviour of STC shear connections under sustained load was studied experimentally and numerically by Chiniforush et al. [16]. Connections with coach screws, dog screws, bolts and bolts in grout pockets were investigated by means of push-out tests in which a sustained load was applied over a period of 16 months in conditions classified as Service Class 2 according to EN-1995–1. The parameters monitored over the testing period were the slip, the relative humidity, and the load, which was re-adjusted to keep the load within the range of $\pm$ 4% of the target load. The results were used to calibrate a long-term rheological model to predict the slip and the creep coefficient over a service life of 50 years. It was determined that the bolts in grout pocket have the lowest creep coefficient of 0.6, whereas the bolted connection showed the highest creep coefficient of 3.9.

Yang et al. [17] did several static push-out tests of glulam timber connected to steel beams using screws and bolts. The bearing capacity, yield characteristics, and failure modes were investigated together with the effect of the type, size and spacing of the connectors as well as the thickness of the glulam plates. The bearing capacity results obtained in the tests were compared to values from codes (i.e. GB/T 50005–2017, NDS-2018 and Eurocode 5). The results show that the models are conservative and lead to overestimation of the connection response. However, Eurocode models produced reasonable predictions of the bearing capacity when compared with results obtained by GB/T 50005–2017 and NDS-2018.

Similarly, Wang et al. [18] conducted a push-out tests campaign of steel-to-CLT connections with inclined screws. The load-slip behaviour and failure modes were analysed for screws with different sizes and inclinations (i.e. 0°, 30°, 45° with respect to the vertical), moreover, the influence of the use of custom taper washers for inclined screws was assessed. The results show that longer screws increase the embedded depth in the timber, leading to greater peak loads. In addition, inclined screws performed better in terms of peak load but when there were no taper washers there was significant loss in stiffness compared to vertical screws. However, this drop in shear stiffness was mitigated by using taper washers. Hence, the inclined screws with taper washers had better load-bearing performance and stiffness.

More recently, tests have been carried out by Chybinski and Polus [19,20] to study aluminium-to-LVL mechanical connections with bolts and screws. The load-bearing behaviour of connections with and without reinforcing toothed plate connectors was compared. Connectors type C2 (i.e. bulldog) were used to reinforce the connections with bolts and screws, additionally, connectors type C11 (i.e. geka) were implemented in connections with screws. In all the configurations, the toothed connectors were placed at the aluminium-timber interface. In most cases, the toothed-plate connectors improved the load-bearing capacity of the connections, however, the stiffness slightly decreased in some cases and in other cases the stiffness increase was negligible.

Zhao et al. [21] did push-out tests on steel-timber connections with screws and a mortar pocket at the tip of the screw. The aim of the grout pocket at the tip was to actively control the screw's failure mode, inducing Mode III failure characterised by two plastic hinges. These connections showed higher strength and stiffness when compared to connections without the mortar pocket. Then, the response of this connectors was investigated through 4-point bending tests and finite element models [22], in this experimental campaign they tested pockets with mortar and two-part epoxy resin. The results showed that the stiffness and the bending capacity of the beams was improved with the mortar pockets.

Vella et al. [23] developed bespoke shear connectors to connect timber boards to cold-formed steel (CFS) beams and ensure composite action. The connections consist of outer fittings made of plastic or steel which were installed in the particle boards in predrilled holes. Screws or bolts were placed inside the fittings to connect the board and the CFS beam. The aim of the outer fittings was to enhance the timber embedment performance by increasing the area directly in contact with the timber material and therefore reduce the embedment stresses. Six types of connectors were developed and tested, the results showed that the best performing shear connector achieved two times the shear resistance, four times the initial slip modulus and seven times the mid-range slip modulus of ordinary self-drilling screws. Furthermore, an analytical model was used to describe the response of the shear connectors, the model was able to predict the ultimate load, slip at ultimate load and the two slip moduli of the connectors.

Concerning demountability and reuse potential, the standard mechanical connectors (e.g. bolts, screws) allow for disassembly of the

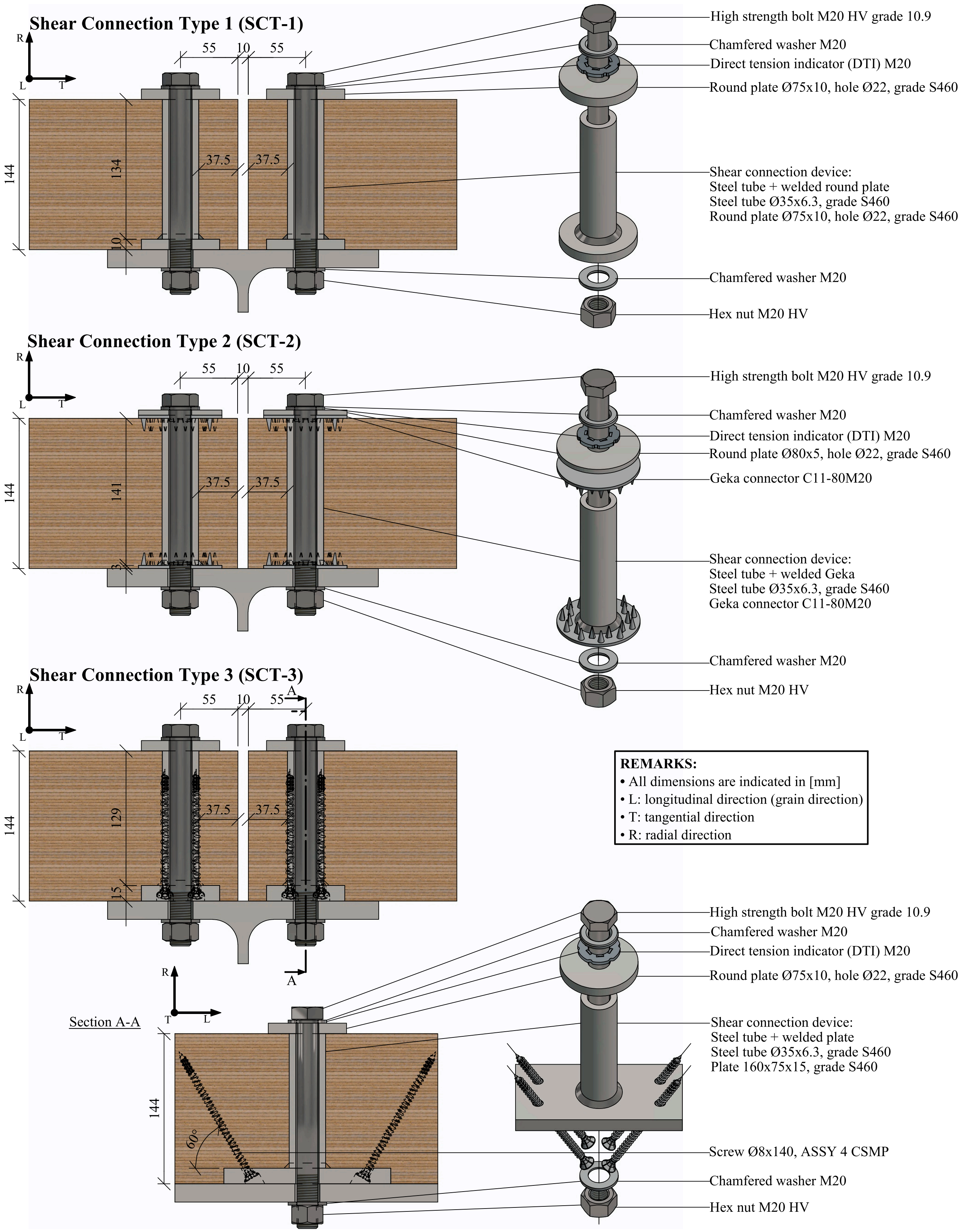

Fig. 1. Details of the shear connections for demountable steel-timber composite beams.

Table 1
Overview of the push-out test series.

Test series	Connection	Test IDs	Specimen IDs
Push-out test series 1	Type 1 (SCT-1)	POT1	POT1-1 POT1-2 POT1-3
Push-out test series 2	Type 2 (SCT-2)	POT2	POT2-1 POT2-2 POT2-3
Push-out test series 3	Type 3 (SCT-3)	POT3	POT3-1 POT3-2 POT3-3

structural elements but the use of adhesives to bond the timber slabs and the steel beams might be avoided. Notwithstanding the advantages of these connectors, there are some aspects at connection level that are of concern when the target is to reuse the timber slabs and the steel beams: (i) the drilling effect of screws and their removal for disassembly could cause permanent damage at the vicinity of the drill; (ii) due to the very low stiffness and strength of timber in the radial direction, when bolts are used as connectors, the timber might be crushed in the radial direction when preloading the bolt, hence, achieving an optimal preload of the bolts is difficult. To overcome these issues, three novel connections have been developed within the framework of the research project “Prefa-SeTi: Steel-Timber Composite Beams” (FNR Luxembourg’s Grant: 15695062). These robust connections allow for disassembly and prevent crushing of the timber in the radial direction while optimal preloads of high-strength bolts can be attained. The investigations done in Prefa-SeTi aim to extend the findings of the RFCS research project REDUCE (Grant No. 710040) [24], in which demountable and reusable steel-concrete composite flooring solutions were investigated.

The load-bearing behaviour of the newly developed connections was investigated experimentally through push-out tests with a double-symmetric test layout according to Eurocode 4 (EN-1994–1) [25] provisions for push-out tests of steel-concrete composites. Three identical specimens were tested to assess each connection type, hence, a total of nine push-out tests were carried out. This contribution presents the details of the connections along with the description of the experimental campaign and the results of the tests.

2. The novel shear connections for demountable steel-timber composite beams

2.1. General

The shear connections introduced in this study represent a robust and sustainable alternative to conventional connections, offering superior protection to structural elements. These connections adhere to the principles of the circular economy and are designed for easy disassembly, enabling the reuse of structural components.

Table 2
Mean strength and elasticity modulus of LVL-C (Kerto-Q) obtained experimentally [36].

Strength		Modulus of elasticity	
Notation	Value [MPa]	Notation	Value [MPa]
$f_{c,1}$	40.4	$E_{c,1}$	7 917.1
$f_{c,2}$	11.1	$E_{c,2}$	1 764.5
$f_{c,3}$	4.0	$E_{c,3}$	95.5
$f_{t,1}$	37.8	$E_{t,1}$	10 680.0
$f_{t,2}$	8.3	$E_{t,2}$	2 199.9
$f_{t,3}$	0.6	$E_{t,3}$	92.1

Subindexes definition: c, compression; t, tension; 1, longitudinal direction; 2, tangential direction; 3, radial direction

The novel shear connections, denoted as Shear Connection -Type 1 (SCT-1), -Type 2 (SCT-2), and -Type 3 (SCT-3), are depicted in Fig. 1. The drawings illustrate the sizes and dimensions of the components employed in this testing campaign, where crossbanded LVL plates (Kerto-Q [26]) with a thickness of 144 mm were connected to I-shaped steel profiles (HEB 260). Each of these connections comprises a "shear connection device" embedded in the timber slab (see sub Section 2.2) and removable components (see sub Section 2.3). The shear connection devices consist of steel tubes and steel elements welded to their base, similar connections implementing bolts or screws inside steel tubes have been used in timber-concrete composite (TCC) beams to connect concrete slabs to timber beams [27,28], and in demountable SCC beams to connect the concrete slabs to steel beams [29].

2.2. Shear connection devices

The shear connection devices, as shown in Fig. 1, consist of a steel tube made of grade S460 in compliance with EN 10025 [30]. The tube has an outer diameter of 35 mm and a wall thickness of 6.3 mm, with its base welded to a reinforcing steel element, which varies depending on the connection type:

i. for SCT-1, it is a round steel plate (S460 according to EN 10025 [30]) with an outer diameter of 75 mm, a thickness of 10 mm, and an inner hole with a diameter of 22 mm;
ii. for SCT-2, it incorporates a Geka connector for bolts M20 (C11-80M20) complying with EN 912 [31];
iii. for SCT-3, it consists of a rectangular steel plate (S460 according to EN 10025 [30]) 160×75×15 mm with custom drills to allow the installation of self-tapping timber screws ⌀8 × 140 mm (Würth ASSY 4 CSMP, ETA-11/0190 [32]) with an inclination of 60º with respect to the horizontal.

The shear connection devices aim to enhance the connections’ performance by:

i. Maximizing embedment in timber due to a substantial contact surface area, especially at the steel-timber interface where significant forces are transferred from the connection device to the timber.
ii. Ensuring slip resistance through the preload of the bolts, allowing for the attainment of adequate preload levels.

2.3. Removable components

In addition to the shear connection device embedded in the timber slab, each connection includes removable components such as bolts, nuts, and washers. The removable components of SCT-1 and SCT-3 are common for both connection types, these components are as follows:

i. A partially threaded high-strength HV bolt M20×210 mm, grade 10.9
ii. A direct tension indicator (DTI) situated beneath the washer at the bolt head’s face
iii. Two washers for HV bolts—one at the bolt head side and another at the nut side
iv. A nut for HV assemblies
v. A round steel plate with the same dimensions (⌀75 ×10 mm, hole ⌀22 mm) as the one welded to the tube’s bottom side of SCT-1

The removable components of SCT-2 include the components i to iv of SCT-1 and SCT-3 as well as two additional components as follows:

v. A round steel plate (⌀80 ×5 mm, hole ⌀22 mm)
vi. A Geka connector with the same specifications (C11-80M20) as the one welded to the tube’s bottom side

Fig. 2. Pictures of the components of the shear connections.

Components (i) to (iv) (i.e., bolt, DTI, washers, and nut) are common to all three connection types and adhere to EN 14399 [33] standards for high-strength HV assemblies. The selection of HV bolts grade 10.9 is deliberate to prevent premature bolt failure due to bearing and shear. This bolt was preloaded to 70% of its ultimate load in accordance with EN 1993–1-8 [34] and EN 14399 [33].

3. Experimental testing campaign

3.1. General

The evaluation of the connections' performance involved a series of push-out tests aimed at assessing the load-slip characteristics of the three types of demountable shear connectors and determining their respective

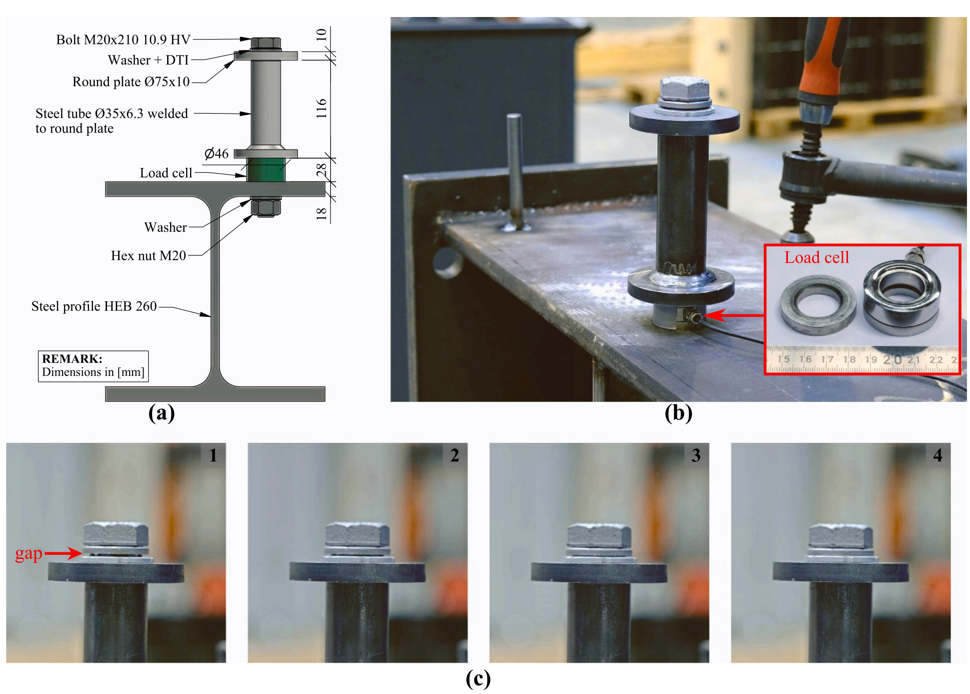

Fig. 3. Preload calibration tests: (a) test setup components; (b) test setup picture and washer load cell; (c) set of pictures showing the DTI's gap closing as the nut is tightened.

failure modes. The test setup was designed following the guidelines outlined in Eurocode 4 (EN 1994–1-1) [25] for push-out tests of steel-concrete connections, leading to the execution of double-symmetric push-out tests.

The experimental testing campaign presented in this contribution consists of three series of push-out tests (POT), with each series corresponding to one shear connection type. Each series comprises three identical specimens, an overview of the tests is shown in Table 1. Specifically, POT1 covers connection type 1, POT2 tests connection type 2, and POT3 examines connection type 3.

3.2. Materials

3.2.1. Laminated veneer lumber (LVL)

Laminated veneer lumber is an engineered wood product formed by bonding and layering 3 mm thick wood veneers, creating wood panels with various dimensions and layups. Two standard layups are distinguished [35]: LVL-P, where veneer layers run parallel to each other, and LVL-C, with approximately 20% of veneer layers oriented perpendicular to the grain.

In this experimental study, crossbanded LVL-C from MetsäWood (i.e. Kerto-Q) [26] with a thickness of 144 mm were used to produce the timber plates (650×300×144 mm) of the push-out tests specimens. The layup of the panels consists of 48 veneers with a thickness of 3 mm as follows: II-IIIII-III-III-IIIII-II-IIIII-III-III-IIIII-II, where "I" represents the veneers with the grain aligned in the longitudinal direction and "-" represents the veneers with the grain placed perpendicular to the grain of the "I" veneers. This LVL product is crafted with Scandinavian Spruce (i.e. Picea abies) wood. The crossbanded veneers in the LVL matrix, enhance the strength and stiffness of the timber elements in the direction perpendicular to the grain.

LVL and cross laminated timber (CLT) are among the most suitable engineered wood products for steel-timber composite flooring applications. However, LVL was chosen for this study due to its low variability in properties, enhanced strength and stiffness, and efficient utilization of smaller diameter logs and low-grade timber. Additionally, since the shear connections were positioned near the timber's edge, which is susceptible to splitting failure, the crossbanded veneers reinforce this region, preventing such failure.

The strength and stiffness properties of the LVL were obtained experimentally [36] and are summarized in the Table 2, the three orthogonal directions considered for the LVL were 1 - longitudinal, 2 – tangential, and 3 - radial. The mean density of these LVL panels is 510 kg/m^3. Before the preparation of the plates for the assembly of the specimens the LVL panels were stored indoors at room temperature. The average moisture content of the LVL plates was 12%, it was measured using a capacitive moisture sensor from Ahlborn (sensor ID: FHA 696 MF) and datalogger ALMEMO 2590.

3.2.2. Steel profiles

The steel profiles used for the push-out test specimens were crafted from an I-shaped european hot-rolled section HEB 260 with steel grade S355 in accordance with EN 10025 [30], without coating or painting. The length of the steel profile was 700 mm in all the specimens. A capping steel plate S355 was welded to one end of the steel profile to transfer the load from the jack to the specimen. Eight coupon shape specimens were tested in tension, and were conducted according to EN ISO 6892–1 [37] in order to determine the mechanical properties of the structural steel S355 of the steel profiles. A mean yield strength of 399 MPa with a coefficient of variation (CV) of 1.7%, a mean ultimate strength of 512 MPa with a CV of 1.7%, and a mean modulus of elasticity of 207 GPa with a CV of 1.9%, were obtained.

3.2.3. Shear connection devices and round steel plates

The shear connection devices described in section 2 are depicted in Fig. 2, they consist of a steel tube grade S460 according to EN 10025 [30] with an external diameter of 35 mm and a wall thickness of 6.3 mm. These tubes were welded to steel elements that differed based on the shear connection type. For SCT-1, a round steel plate was employed, while SCT-3 featured a rectangular plate, both made from steel grade S460. SCT-2 incorporated a Geka connector welded to the tube. None of these steel tubes or plates had coatings or paint.

Round steel plates, made of S460 steel, were placed beneath the bolt head and below standard washers to prevent uplift of the timber panels. SCT-1 and SCT-3 featured round plates with an outer diameter of 75 mm and a thickness of 10 mm, while SCT-2 utilized round plates with an outer diameter of 80 mm and a thickness of 5 mm. Four coupon shape specimens were tested in tension, and were conducted according to EN ISO 6892–1 [37] in order to determine the mechanical properties of the steel S460 of the shear connection devices. A mean yield strength of 526 MPa with a CV of 0.2%, a mean ultimate strength of 581 MPa with a CV of 0.3%, and a mean modulus of elasticity of 203 GPa with a CV of 1.6%, were obtained.

3.2.4. Geka connectors

Connectors type C11 (i.e. Geka connectors) for bolts M20 (C11–80M20) complying with EN 912 [31] were used in shear connection type 2 (SCT-2), these connectors are shown in Fig. 2. One Geka connector was welded to the steel tubes described in Section 3.2.3 and another connector was placed underneath the bolt head and the round steel plate. The Geka connectors were made of galvanized cast iron, they had an outer diameter of 80 mm, a hole with a diameter of 21 mm, and a total height (teeth and plate) of 15 mm.

3.2.5. Screws

Universal partial-thread screws (see Fig. 2) were installed in shear connection type 3 (SCT-3). The screws fix the shear connection device to the timber and make the connection more robust, they help to reduce slip and prevent crushing of wood at early loading stages. These

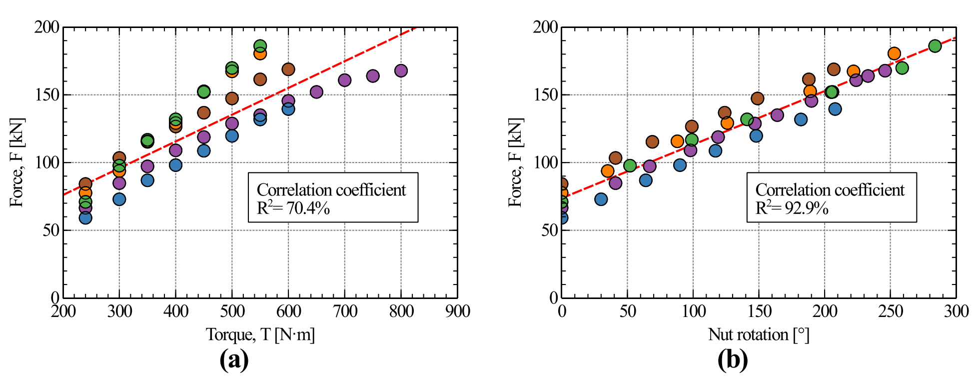

Fig. 4. Results of preload calibration tests: (a) force vs. torque, and (b) force vs. nut rotation.

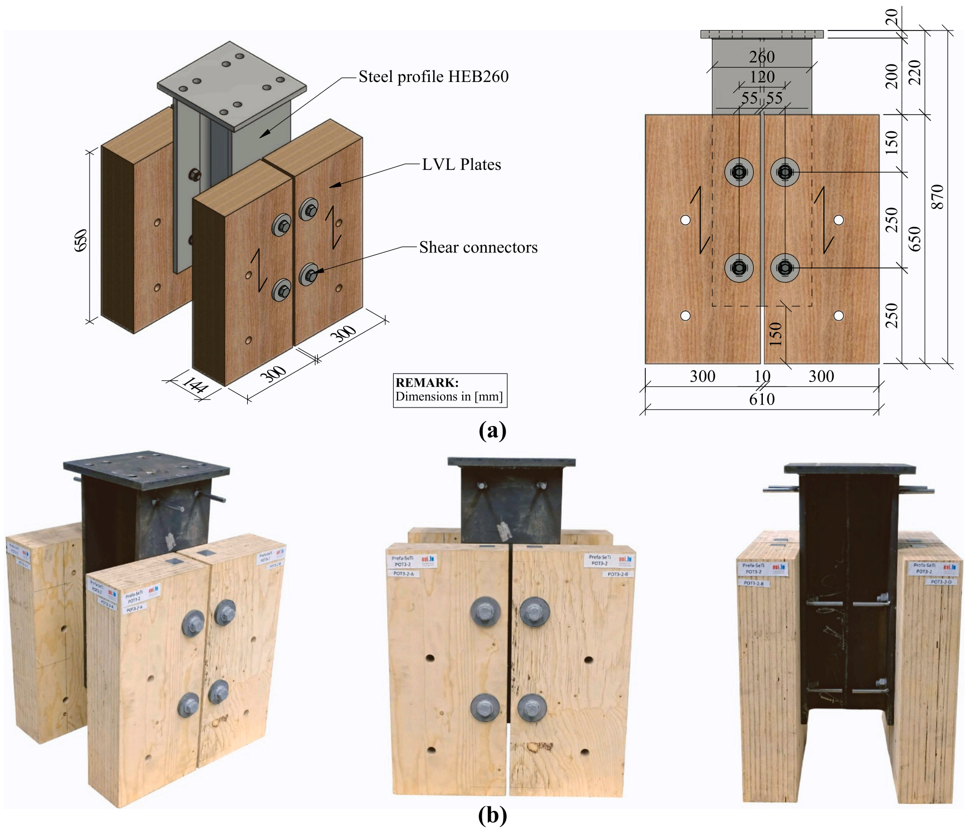

Fig. 5. Push-out test specimen: (a) drawings showing components and dimensions; and (b) pictures.

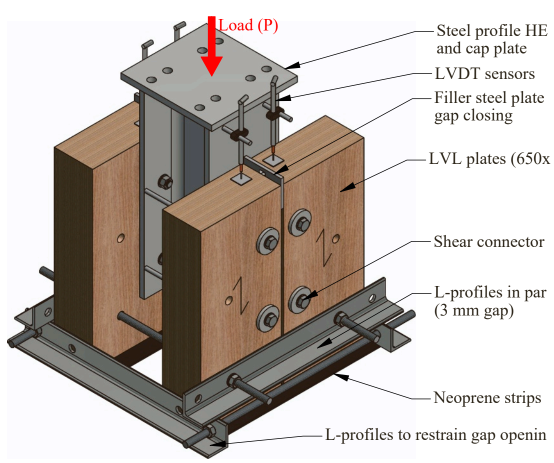

Fig. 6. Push-out test setup components.

universal partial-thread screws (ASSY 4 CSMP) made of stainless steel from Würth (ETA 11/0190) [32] with a diameter of 8 mm and a length of 140 mm were installed with an inclination of 60º with respect to the horizontal. According to the European Technical Approval of the product (ETA 11/0190) [32], the characteristic tensile strength ($f_{t,k}$) is 21.5 kN and the characteristic yield moment ($M_{y,k}$) is 23 Nm.

3.2.6. Bolts, washers and direct tension indicators (DTI)

High-strength partially threaded bolts M20 10.9 HV according to EN 14399 [33] (see Fig. 2), with a diameter of 20 mm and a length of 210 mm, were used in the three shear connections. A flat washer with chamfer for high strength fittings and hot dip galvanisation in accordance with EN 14399 [33] was positioned under the head of the bolt and another washer was placed under the nut, the washers are shown in Fig. 2. The washer has an outer diameter of 37 mm, an inner diameter of 21 mm and a thickness of 4 mm. The hexagon nuts (see Fig. 2) used in the assemblies comply with EN 14399 [33], and were made with hot-dip galvanised steel class 10Z, the nuts had a height of 16 mm.

The direct tension indicators (DTI) (Fig. 2) manufactured by TurnaSure, made of galvanized steel, were placed under the washer at the bolt head side to ensure the minimum preload requirement was met. Before preloading the bolt, the protrusions of the DTI leave a gap between the DTI and the element placed next to it (e.g. washer, nut, bolt head, etc.), during the preloading the gap reduces, this gap is checked with a feeler gauge which has a calibrated thickness. The minimum preload has been reached when the minimum number of feeler gauge refusals as defined in EN 14399–9 [33] is observed.

3.3. Bolt preload calibration

In accordance to Eurocode 3 [34] and EN 1090–2 [38] the bolts were preloaded at 70% of its ultimate strength. The required preload for the M20 10.9 bolts used in this study's connection assemblies was 172 kN. There are different methods to ensure the minimum required preload for slip resistant connections is attained: (i) torque method, (ii) combined method, (iii) HRC method, and (iv) direct tension indicator method (DTI method). In this study, the DTI method and the combined method (i.e. torque+nut rotation) were used to ensure that the minimum preload was reached.

To identify a suitable torquing method, tests were conducted on a connection assembly of SCT-1. The test setup shown in Fig. 3, consists of all elements of SCT-1, however, the shear connection device (i.e. the tube welded to a round plate) was shortened in order to accommodate a load-cell washer, which was placed between the shear connection device and the flange of the beam to measure the applied force while tightening the nut. The load cell used to measure the preload of the bolts in the tests

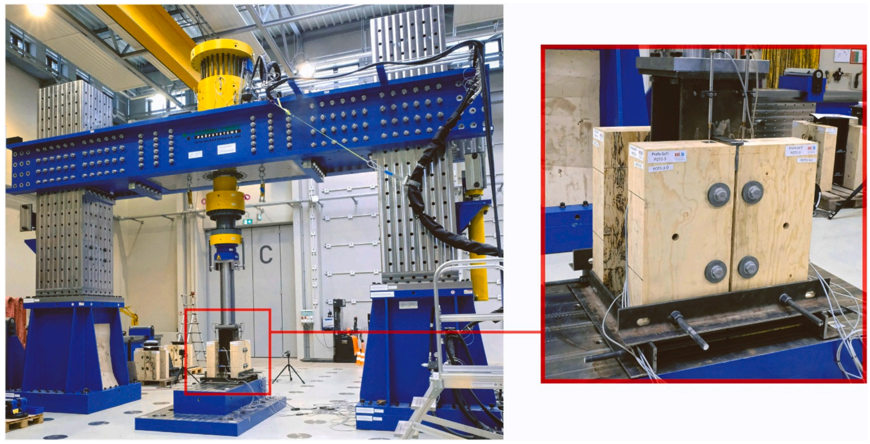

Fig. 7. Push-out test setup pictures.

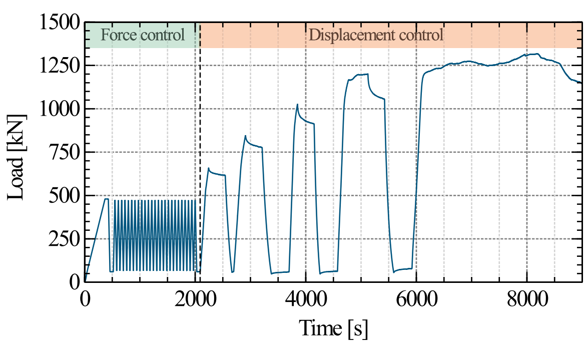

Fig. 8. Time history of the applied load.

was an HBM washer load cell 1-KMR/300KN (see Fig. 3b). The bolts, nuts and washers were used as delivered and no lubricants were applied to any part of the assembly.

The procedure involved the following steps:

1. Initially, the nut was tightened with a torque wrench with the torque cut-out set to 240 Nm, during this step and subsequent steps, the bolt head was secured with a wrench to prevent its rotation;
2. After the initial torquing step, the preload was applied gradually by increasing the torque cut-out of the torque wrench;
3. The torque cut-out value, the nut rotation, and the force readings from the washer load cell were recorded for each torque cut-out increment;
4. The test was finished when a load between 170 kN and 190 kN was reached.

This procedure was repeated for five new bolts. Additionally, the gap between the DTI and the washer was measured with a feeler gauge to determine the number of refusals, as defined in EN 14399–9 [33].

The data collected during the preload tests was used to construct the plots displayed in Fig. 4. The two plots show the values from an initial point at which a torque of 240 Nm was applied, meaning that, for a nut rotation of 0° as indicated in the plot of Fig. 4b, a torque of 240 Nm had already been applied. The correlation between force and torque was 70.4%, and the correlation between force and nut rotation was 92.9%. Since the correlation between force and nut rotation was significantly stronger, the full torque-based method was discarded, and a combined torque+nut rotation method for tightening the bolts of this testing campaign was defined as follows: (i) an initial torque of 240 Nm was applied, then (ii) the nut was turned 260° to ensure the required preload.

3.4. Specimen details

According to the literature presented in the introductory section of this contribution, double symmetric push-out test setup is the preferred testing setup to determine the mechanical properties of steel-to-timber connections. In this investigation, the specimen's arrangement and its components sizes were defined following the recommendations for push-out tests established in Eurocode 4 [25] for steel-concrete composites. All the POT specimens of this testing campaign shared basic components, materials, dimensions and connections arrangement, the difference between them lies in the shear connection, which is different for each test series. Hence, the POT specimens have the following main components (see Fig. 5):

i. a hot rolled steel section HEB 260 with steel grade S355 as described in Section 3.2.2, the holes drilled in the flanges for the bolts had a diameter of 24 mm, oversized holes with a diameter of 24 mm instead of 22 mm have been chosen to give tolerances during the assembly of the STC slabs and because the target of the

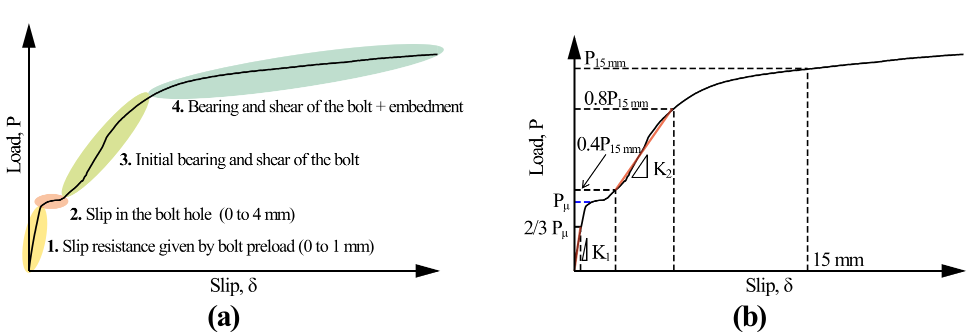

Fig. 9. Idealized load-slip curves: (a) branches of the idealized load-slip curve; (b) parameters of the load-slip curve.

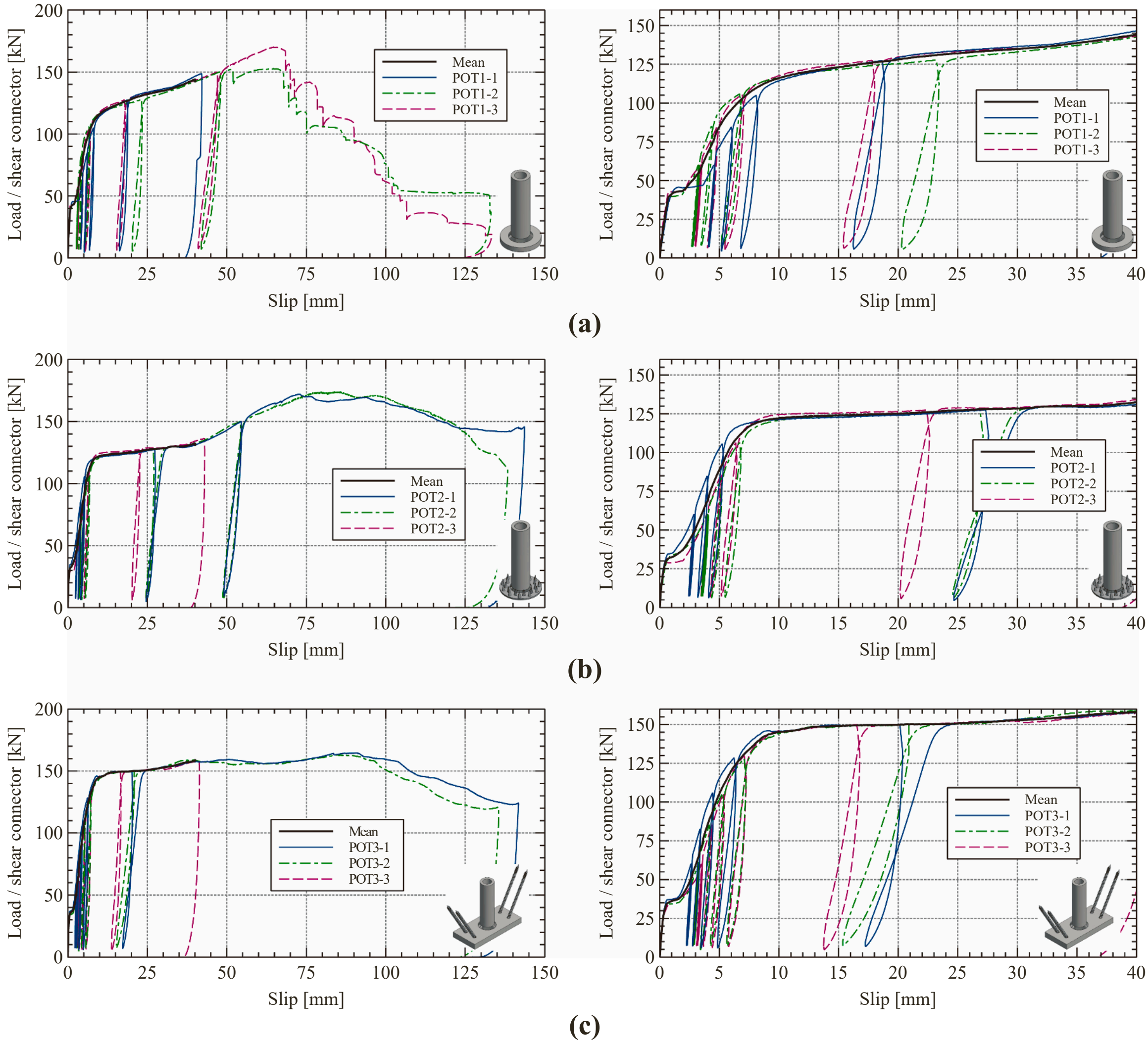

Fig. 10. Load-slip curves of the test series: (a) POT-1 for SCT-1, (b) POT-2 for SCT-2, and (c) POT-3 for SCT-3.

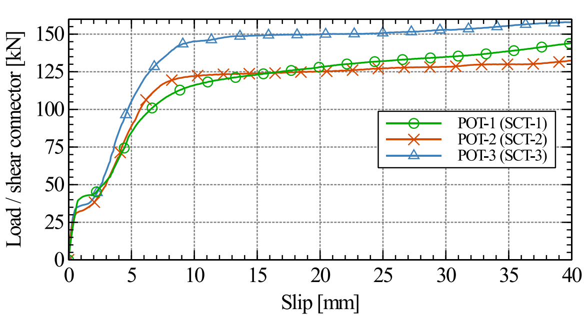

Fig. 11. Mean load-slip curves of the three push-out test series.

connections is to be used in demountable and reusable flooring systems where tolerances are important;

ii. four LVL plates with dimensions 650×300×144 mm, two plates were connected to each flange of the steel profile leaving a gap of 10 mm between adjacent timber plates, the orientation of the grain was parallel to the application of the load, the holes to accommodate the shear connection devices had exactly the diameter of the steel tubes (⌀35 mm) so that there was no gap;

iii. a total of eight shear connectors to attach the timber plates to the steel profile, two rows of connectors were placed on each flange of the beam with a longitudinal spacing of 250 mm and transversal spacing of 120 mm.

During specimen assembly, the shear connections introduced in this study demonstrated ease of installation and facilitated the overall component assembly process.

3.5. Test setup and instrumentation

The testing setup is depicted in Fig. 6 and Fig. 7, it consists of the specimen with the components described in Section 3.4, sensors to measure displacements, and additional elements to secure the specimen and restrain the displacement of the LVL plates. Linear variable differential transformer (LVDT) sensors were installed in the specimen to measure the relative displacement (i.e. slip) between the steel profile and each one of the four LVL plates. To prevent the gap between adjacent plates from closing, steel plates were placed in the gap. Additionally, steel L-profiles fixed with threaded rods were positioned at the lower part of the specimen on two sides of the specimen to prevent the gap from opening. Similar steel L-profiles, also fixed with threaded rods, were used to prevent components from separating once the connection was lost, in this case, a gap of 3 mm was left between the L-profiles and the timber plates. The specimen was placed on a steel block covered by neoprene strips. The load was applied vertically using the test portal PP 4000 HK-2DH from Form+Test, with a capacity of 4000 kN.

Table 3
Summary of parameters obtained from the load-slip curves according to Fig. 9b.

Series	Specimen	P_μ [kN]		$P_{6\ mm}$ [kN]		$P_{15\ mm}$ [kN]		K_1 [kN/mm]		K_2 [kN/mm]		P_{max} [kN]	
		Test	Mean	Test	Mean	Test	Mean	Test	Mean	Test	Mean	Test	Mean
POT-1	POT-1-1	46.2	43.3 (6.9)	84.6	95.7 (10.1)	122.9	124.9 (1.5)	49.7	61.9 (17.5)	15.1	16.4 (11.9)	-	161.4 (7.6)
	POT-1-2	40.3		102.4		125.3		65.5		18.7		152.7	
	POT-1-3	43.3		100.0		126.6		70.4		15.5		170.1	
POT-2	POT-2-1	34.9	32.9 (8.7)	113.8	104.4 (7.9)	122.8	124.0 (1.1)	86.2	82.8 (21.5)	21.5	20.4 (6.4)	172.0	173.1 (0.9)
	POT-2-2	34.3		98.3		123.8		63.5		18.9		174.2	
	POT-2-3	29.6		101.3		125.5		98.7		20.7		-	
POT-3	POT-3-1	37.5	36.8 (5.6)	126.5	120.2 (4.6)	149.6	149.1 (0.4)	92.6	87.9 (5.7)	24.5	23.5 (6.6)	164.7	163.8 (0.7)
	POT-3-2	34.5		116.6		148.4		88.5		21.7		163.0	
	POT-3-3	38.4		117.5		149.2		82.6		24.3		-	

Remark: The values in parentheses indicate the coefficient of variation in %.

3.6. Loading procedure

The time history of the load applied in the push-out tests is illustrated in Fig. 8. Due to the lack of testing standards targeting steel-timber composite structures, the loading protocol implemented in this study considers the loading procedure for push-out tests of Eurocode 4 (EN 1994–1-1) [25] presented in Annex B and the loading protocol of EN 26891 [39]. The loading procedure had two main phases, in the first phase the loading was carried out in force control mode, and in the second phase the loading was in displacement control mode. In the first phase the specimen was loaded up to 480 kN which is about 40% of the estimated maximum load (P_{est}), then unloaded to 60 kN ($\sim 0.05P_{est}$), similar to what is established in the initial loading-unloading phase of EN 26891. After this initial step, 25 loading-unloading cycles within 60 kN and 480 kN were applied at a frequency of 1 cycle per minute, this was done as it is recommended in Eurocode 4 for push-out tests of SCC connections. After the 25th cycle, the system was switched to displacement control. Then, subsequent loading-unloading steps were carried out at 660 kN, 840 kN, 1020 kN and 1200 kN. After loading the specimen up to each one of these load levels there was a waiting period of five minutes, followed by unloading to 60 kN, when this load was reached there was a waiting period, which was of 30 seconds after the loading step at 660 kN and of five minutes in all subsequent steps. The last loading step, after loading at 1200 kN and unloading at 60 kN, the specimen was either loaded up to failure of the connections or stopped when the maximum available deformation of the testing setup (~140 mm) was reached.

Only one push-out test specimen of each type of shear connection was tested up to a displacement of about 40–45 mm, when this slip was reached the test was halted to later do cuttings of these specimens and observe the state of the connections at this level of deformation. This approach was applied to specimens POT-1–1, POT-2–3 and POT-3–3. The other two specimens of each series were tested until the shear connectors fractured or stopped when the maximum available deformation of the testing setup was reached.

4. Results and discussion

4.1. Load-slip response

4.1.1. Load bearing mechanism and sequence

The load-slip ($P - \delta$) response of the three types of shear connectors exhibited nonlinearity with significant deformation capacity. Despite differences in the magnitude of loads and bearing stiffness, the load-slip patterns were similar among the three types of shear connections and can be described as follows (see Fig. 9a):

1. An initial stiff response due to the friction provided by the preload of the bolt (i.e. slip resistance), which ends when the slip resistance is overcome. The preload increases the friction between the surface of the steel beam flange and the shear connection device, nevertheless, there is a small displacement which can be attributed to a combination of both, a relative displacement between the flange and the shear connection device, and initial embedment of the shear connection device in the timber.
2. When the slip resistance was overcome, and due to the clearance between the bolt and the hole in the flange of the beam, the sliding friction mechanism came into effect until the flange and the bolt were in full contact.
3. When contact between the bolt and top flange of the beam occurred, the load was transferred via shear in the bolt and respective bearing pressure at the contact point in the top flange. Due to the pressure of the bolt on the shear connection device there was embedment of the shear connection device in the wood, resulting in a nonlinear, monotonically increasing response.
4. In the next branch the stiffness of the system reduced considerably due to yielding of the connection, there was bearing and shear of the bolt as well as embedment of the shear connection in the timber, this branch also exhibited a monotonically increasing response.

4.1.2. Characteristics of the load-slip curves

The detailed load-slip curves of the three test series are shown in Fig. 10, and a comparison of the mean curves is depicted in Fig. 11. Table 3 gives a summary of different parameters calculated for the curves, these parameters are illustrated in Fig. 9b. The load-slip curves of each specimen were obtained by averaging the slip recorded by the four LVDTs on the top side of the specimen, then the load applied to the specimen was divided by eight, which is the number of connectors in each specimen. In this way the load-slip curves presented in Fig. 10 display the average load taken by each connector and the respective average slip. The mean curves of each test series (Fig. 11) were obtained as the average of the force values for constant slip values. The results of the tests show that the connection type, the position of the bolt inside the hole, and the bolt preload have relevant influence on the load-slip response.

4.1.3. Slip resistance and initial stiffness

The bolts preloading procedure explained in Section *3.3 Bolt preload calibration* was developed to ensure consistent preload and slip resistance in the test specimens with the same type of shear connection. Hence, the first branch of the load-slip curve exhibits a high stiffness due to the friction between the shear connection device and the flange of the steel beam, which is enhanced by the preload of the bolts. The average slip resistances (P_μ) obtained in each test series were 43.3 kN, 32.9 kN, and 36.8 kN for SCT-1, SCT-2, and SCT-3 respectively, with coefficients of variation of 6.9%, 8.7% and 5.6%, respectively for each shear connection type. The stiffness (K_1) of the initial branch of the load-slip curve was estimated as the slope of a secant line intersecting the curve

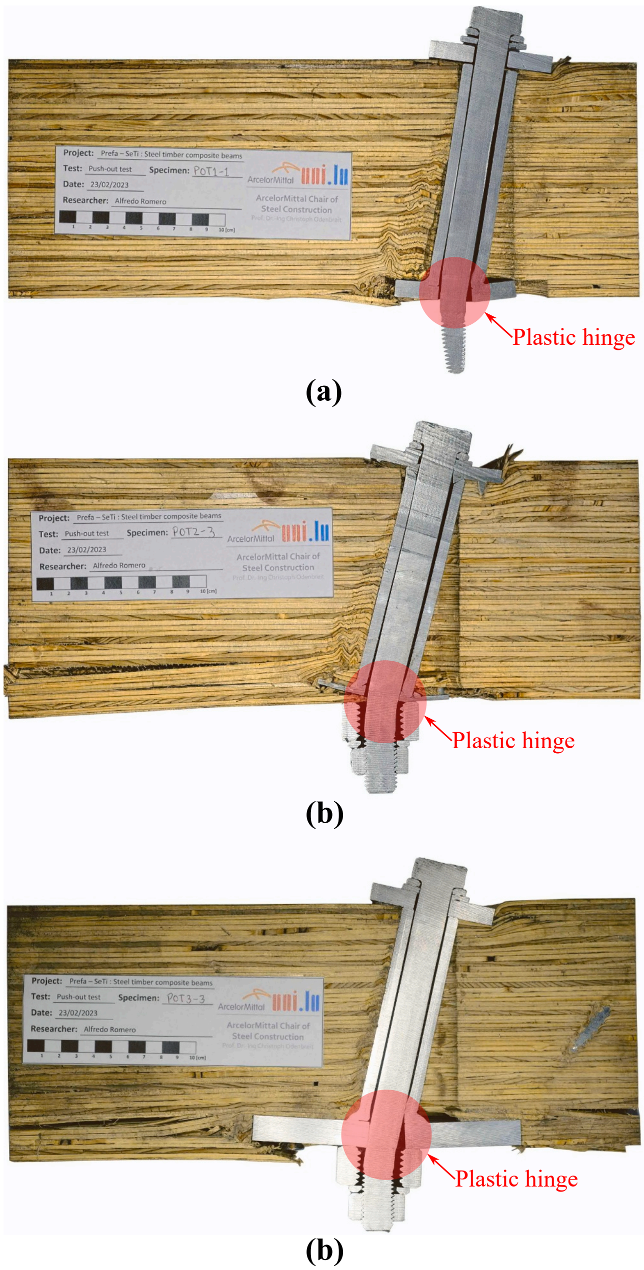

Fig. 12. Cuttings of shear connections from tests that were stopped at 40–45 mm slip: (a) SCT-1, (b) SCT-2, and (c) SCT-3.

at $2/3P_{\mu}$, the average values obtained respectively for each shear connection type were 61.9 kN/mm, 82.8 kN/mm and 87.9 kN/mm with coefficients of variation of 17.5%, 21.5% and 5.7%.

4.1.4. *Influence of the position of the bolt in the hole*

The diameter of the bolt in the three types of shear connection was 20 mm and the diameter of the holes in the flanges was 24 mm in all the specimens, this means that the bolt could have a clearance within the hole of up to 4 mm, hence, the initial slip before the bolt starts to work in bearing and shear could be up to 4 mm. During the assembly of the specimens the intention was to place the bolt at the center of the hole, this was achieved in most of the specimens, which in general showed a slip of about 2 mm at the end of the second branch of the load-slip curves (branch 2 from Fig. 9a), however, the specimen POT-2–1 and POT-2–2 had a very small initial slip (~1 mm) in the bolt hole before starting the bearing branch, and the specimen POT-1–1 showed almost 4 mm of initial slip in the bolt hole. This demonstrates the influence of the position of the bolt inside the hole of the flange on the load-slip response, which causes a horizontal shift of the bearing branch (branch 3 from Fig. 9a) in the load-slip curves, hence, depending on the position of the bolt, this bearing branch starts to develop within the range of 0 to 4 mm of slip.

4.1.5. *Loads reached at 6 mm slip*

It is expected that the connections work at slip values smaller than 6 mm, this is to avoid damages of the connections and in the timber. This is in line with the assumptions made in Eurocode 4 for shear connections of steel-concrete composite structures, in which the connectors are required to have a slip capacity of at least 6 mm, so that beams reach their capacity when a slip of about 6 mm occurs. Hence, the loads corresponding to a slip of 6 mm ($P_{6\ mm}$) were also obtained for each test. The average $P_{6\ mm}$ loads for each type of shear connection were 95.7 kN, 104.4 kN and 120.2 kN for SCT-1, SCT-2 and SCT-3 respectively, with their corresponding coefficients of variation of 10.1%, 7.9% and 4.6%.

4.1.6. *Loads reached at 15 mm slip*

For comparison purposes the load at a slip of 15 mm ($P_{15\ mm}$) of each test specimen was extracted from the data. According to EN 26891 [39] the load at 15 mm shall be recorded as the maximum load, however, the connections reached the maximum loads at slip values which exceed considerably the 15 mm limit established in EN 26891 [39], therefore, the parameter has been included in this analysis as a reference value.

Regardless of the initial slip before the bearing branch, the load-slip curves reach values of the same order of magnitude at a slip of 15 mm, this is a point at which they nearly converge. The load corresponding to this slip was taken as a reference for comparison of the different types of connections and for the definition of the lower and upper bounds for the evaluation of the stiffness of the bearing branch. The average $P_{15\ mm}$ loads for each type of shear connection were 124.9 kN, 124.0 kN and 149.1 kN for SCT-1, SCT-2 and SCT-3 respectively, with their corresponding coefficients of variation of 1.5%, 1.1% and 0.4%, which validates the near-convergence of curves at this point. SCT-1 and SCT-2 reached similar load levels, whereas the load reached by SCT-3 were about 20% higher. This confirms that the implementation of screws to reinforce the connections can significantly increase the load-carrying capacity of the connections.

4.1.7. *Stiffness of the connection in the bearing branch*

The stiffness (K_2) of the connection in the bearing branch (branch 3 from Fig. 9b) was obtained through linear regression analyses. These regression analyses were done in a region enclosed by the points corresponding to $0.4P_{15\ mm}$ and $0.8P_{15\ mm}$. The average stiffness values obtained in each test series were 16.4 kN/mm, 20.4 kN/mm and 23.5 kN/mm, with coefficients of variation of 11.9%, 6.4% and 6.6% for SCT-1, SCT-2 and SCT-3 respectively. Moreover, in all the regression analyses, correlation coefficients (R^2) greater than 0.97 were obtained. The SCT-2 and SCT-3 had larger stiffness than SCT-1, this could be due to the use of Geka connectors in SCT-2 and inclined screws in SCT-3 along with a thicker plate. In terms of stiffness, the SCT-3 had the best performance.

4.1.8. *Post-yielding response*

In the bearing branch there was yielding of the connection, at this point the initial bearing stiffness reduced significantly and the branch 4 from Fig. 9a developed. The yielding of the connection is owed to a combination of non-reversible timber crushing / embedment of the shear connection device, and plastic yield deformation of the bolt and the shear connection device. SCT-1 exhibited hardening from the yielding point, whereas in the SCT-2 and SCT-3 this branch was close to a plateau between a slip of 10 mm and 35 mm. From a slip of 35 mm SCT-2 and SCT-3 displayed hardening, in SCT-3 hardening was very

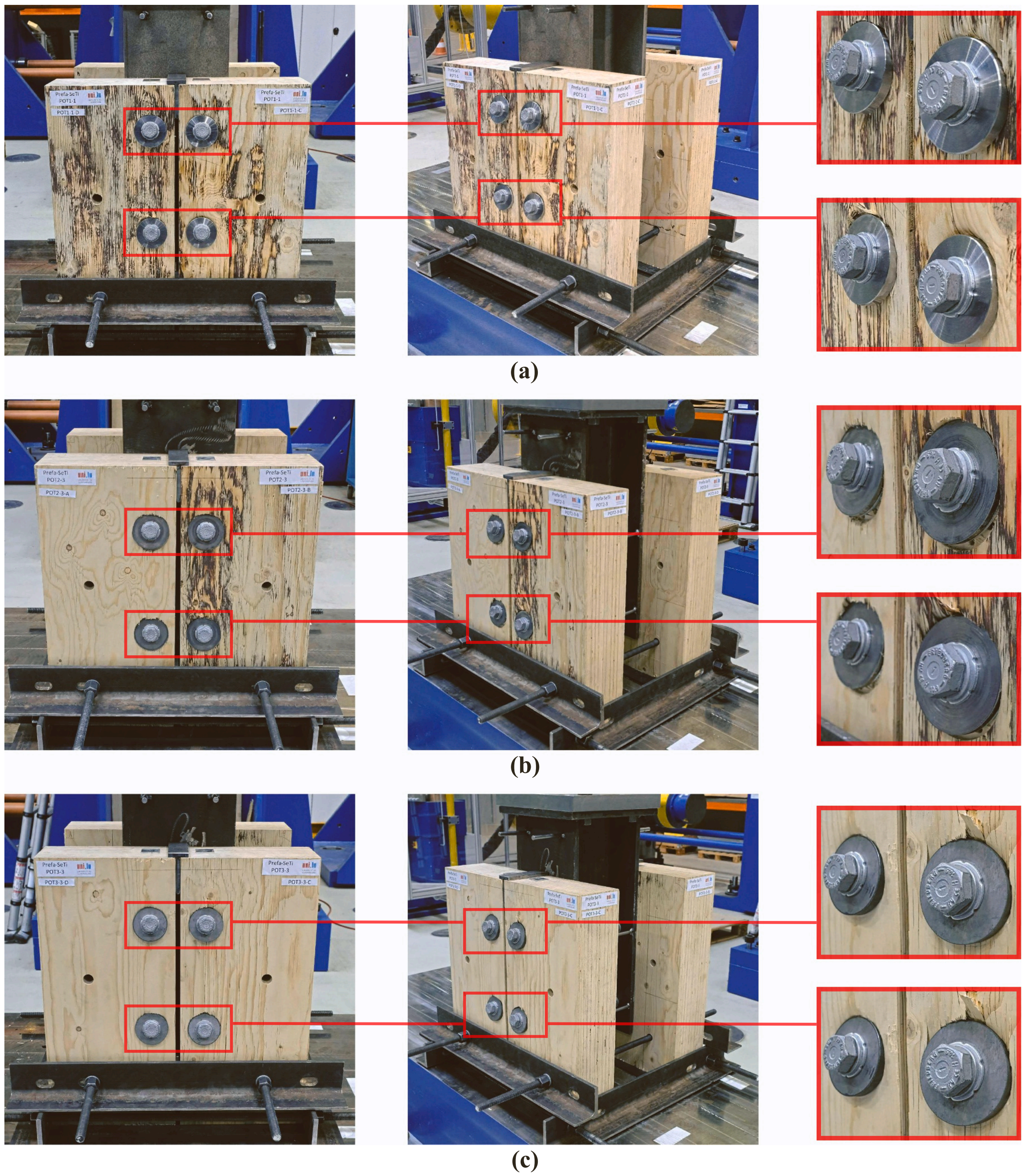

Fig. 13. Pictures of the specimens tested up to a slip of 40–45 mm: (a) SCT-1, (b) SCT-2, and (c) SCT-3.

small, and the post-yielding branch was close to a plateau. This hardening was related to the densification of timber and hardening of the steel components of the connection (i.e. shear connection devices and bolt).

For the specimens that were tested up to rupture of the connectors or up to the maximum available deformation of the testing setup, the maximum loads (P_{max}) were obtained from the load-slip data. The average values for SCT-1, SCT-2 and SCT-3 were 161.4 kN, 173.1 kN, and 163.8 kN respectively. After reaching this peak load, these specimens of SCT-2 and SCT-3 exhibited a post-peak softening branch, while the specimens of SCT-1 experienced rupture of the bolts.

4.2. Failure mode

The three types of shear connection showed a similar failure mode as shown in Fig. 12. In these connections, there was timber crushing due to embedment of the shear connection device and one plastic hinge developed in the steel components at the steel-timber interface.

In the specimens that were not tested up to rupture (i.e. POT-1–1, POT-2–3 and POT-3–3), pictures shown in Fig. 12 (connections' cuttings) and Fig. 13 (specimens), it can be noticed that there was rotation of the connection and embedment of the steel tube in the timber as well as little but noticeable embedment of the round steel plate—placed under the bolt head—in the timber plate surface.

The three connections exhibited a large deformation capacity post-yielding. In the case of SCT-1, the post peak behaviour was characterized by softening followed by rupture of the bolts which did not fail simultaneously, this can be seen in the load-slip plots of SCT-1 (see Fig. 10a), the failure of each bolt caused a sudden drop in the post-peak branch, hence, this branch has several load drops. One of the bolts that

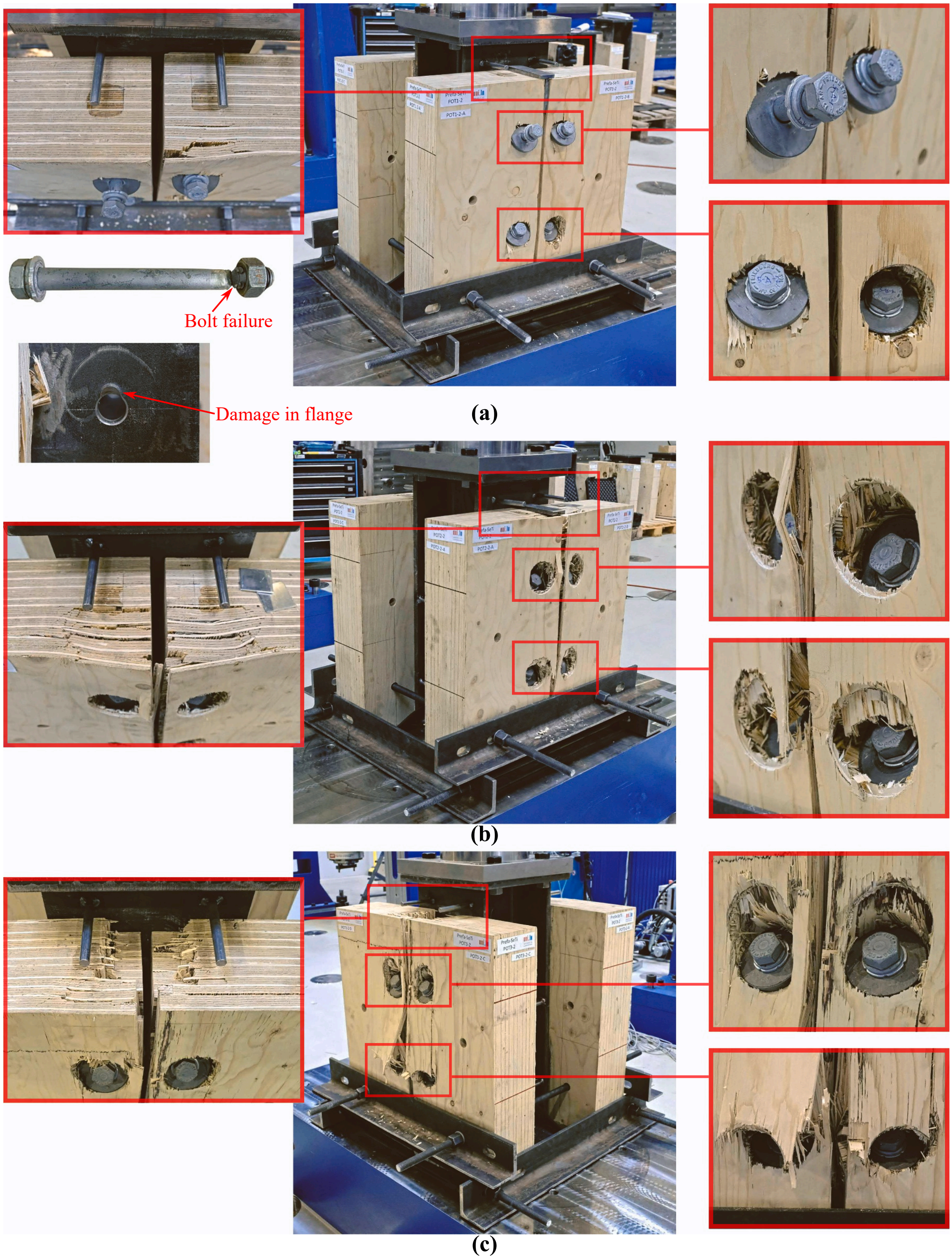

Fig. 14. Pictures and details of the specimens tested up to bolt rupture in the case of (a) SCT-1, and up to the maximum available deformation (~140 mm) of the testing setup in the case of (b) SCT-2 and (c) SCT-3.

experienced rupture is depicted in Fig. 14a. The SCT-2 and SCT-3 were stopped when the maximum available deformation of the testing setup was reached (i.e. ~140 mm), up to this point there was no rupture of the bolts, nevertheless, in both cases a softening branch post-peak was observed.

At large deformations (e.g. ~140 mm), in the specimens tested up to rupture, the head of the connections were fully embedded in the timber as depicted in Fig. 14. In the specimens of SCT-1 (Fig. 14a) the round plate placed under the bolt head was embedded in the surface of the timber plates. Additionally, in some timber plates with SCT-1, downwards pulling of the wood was observed at their top side. Similarly, in specimens of SCT-2 (see Fig. 14b) the Geka and the round plate placed under the head of the bolt was fully embedded but, in this case, the downwards pulling of veneers at the top of the timber plates was more evident, this is related to the anchoring effect of the Geka teeth which pull downwards the veneers near the surface of the timber plates. The SCT-3 (see Fig. 14c), exhibited a similar but more visible downwards pulling effect, which is larger at the veneers located near the face next to the flange of the steel profile, this is due to the pulling effect of the inclined screws placed in that side of the timber plate.

5. Conclusions

To achieve a successful transition towards a sustainable construction sector, based on the principles of the circular economy, the exploration of alternatives to existing non-sustainable practices is imperative. This study addressed this imperative by presenting: (i) the details of three novel connections for demountable and reusable STC beams and flooring systems, and (ii) results of push-out tests conducted to evaluate their performance.

The key findings and conclusions of this investigation can be summarized as follows:

- The shear connections introduced in this study offer ease of installation and facilitate the assembly and disassembly of components. These characteristics align with the principles of the circular economy.
- The shear connection devices protect the timber and facilitate the attainment of the required preload for high-strength bolts, preventing timber crushing in the radial direction.
- Preload calibration tests demonstrated a weak correlation (70.4%) between force and torque, whereas force and nut rotation exhibited a strong correlation (92.9%). Hence, a combined method involving torque and nut rotation is preferred to achieve the minimum preload. It is important to note that the rotation values obtained in this study were significantly influenced by the use of the DTI in the connection assembly, since the protrusions compress significantly, it has an important influence on the rotation of the nut. Therefore, if the DTI is removed from the assembly a new calibration of the required rotation to attain the minimum preload is needed.
- All three connection types exhibited a nonlinear load-slip response. While there were differences in terms of the loads, stiffness, and post-peak behaviour, they shared a similar load-slip pattern.
- The clearance between the bolt and the holes in the flanges and the shear connection devices significantly impacts the initial slip of the connection, determining when the bolt starts to work in bearing and shear.
- In terms of stiffness of the bearing branch (K_2) , SCT-1 exhibited the lowest stiffness (16.4 kN/mm) while SCT-2 and SCT-3 performed better (20.4 kN/mm and 23.5 kN/mm, respectively).
- At a slip of 15 mm, the load-slip curves of each test series reach similar values, this was demonstrated by the low coefficient of variation observed in the three specimens of each test series. At this point, SCT-1 and SCT-2 reached a load P_{15mm} of approximately 124 kN, while SCT-3 reached 149.1 kN.
- The connectors exhibited a load bearing capacity per shear connector at a 6 mm slip of 95.7 kN, 104.4 kN, and 120.2 kN for SCT-1, SCT-2, and SCT-3, respectively.
- The peak loads for SCT-1 and SCT-3 were within the same order of magnitude, at 161.4 kN and 163.8 kN, respectively, whereas SCT-2 outperformed both with a peak load of 173.1 kN.
- The higher stiffness and loads reached by SCT-3 at early loading stages (e.g. $\delta < 40mm$) can be attributed to their reinforcement provided by inclined screws, which contribute with their withdrawal and bearing capacities.
- All three connection types displayed a significant deformation capacity (greater than 40 mm slip), attributed to the high compressibility of timber, and the strength of the bolts and the shear connection devices.
- The three connection types exhibited a similar failure mode. There was timber crushing due to embedment of the shear connection device and one plastic hinge developed in the steel components at the steel-timber interface.
- The three connection types reached a maximum load value, followed by rupture of bolts in the case of SCT-1 and softening for SCT-2 and SCT-3 without rupture of the bolts.
- Only SCT-1 exhibited rupture of the bolts, the failure of the first bolt occurred at a slip of 65-75 mm, SCT-2 and SCT-3 were able to reach the maximum available deformation of the testing setup which was approximately 140 mm, at this deformation the push-out tests of SCT-1 and SCT-3 were stopped and no rupture of the connections was observed.

This study provided valuable insights into the behaviour of novel demountable connections in crossbanded LVL, focusing on connections implementing bolts with a diameter of 20 mm and a steel grade of 10.9, which were installed in LVL panels 144 mm thick. While these findings contribute significantly to our understanding of these novel connections with this particular configuration, it is important to note the limitations: the study was confined to a single bolt size and grade, utilized only crossbanded LVL with the grain aligned parallel to the load, and did not explore varying panel thicknesses or other timber materials. Due to these constraints, further research is necessary to broaden the scope of these results. Investigations into different orientations of LVL, different timber materials such as CLT, variations in bolt sizes and grades, exploration of diverse geometries, and varying parameters would deepen our understanding of the structural behaviour and performance of these connections.

The connections introduced in this study offer a robust and sustainable alternative to conventional mechanical connections. These connections are easy to install and facilitate the assembly and disassembly of components. These characteristics not only provide a practical advantage over traditional connections but also align with the principles of the circular economy. By promoting reusability and reducing waste, these connections embody a sustainable approach to building design, potentially leading to more environmentally conscious construction practices. However, while they present these benefits, certain factors need careful consideration. The preparation and installation of the shear connection devices can be time-consuming, and the complexity of the connections, which consist of multiple elements and require preloading, may pose challenges. Furthermore, ensuring precise alignment for the holes in the steel beams to accurately match those in the LVL slabs, is crucial for the successful implementation of these connections.

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CRediT authorship contribution statement

Romero Alfredo: Writing – review & editing, Writing – original draft, Visualization, Validation, Software, Resources, Project administration, Methodology, Investigation, Funding acquisition, Formal analysis, Data curation, Conceptualization. **Odenbreit Christoph:** Writing – review & editing, Validation, Supervision, Resources, Project administration, Methodology, Funding acquisition, Formal analysis, Conceptualization.

Declaration of Competing Interest

The authors declare the following financial interests/personal relationships which may be considered as potential competing interests: Alfredo Romero reports financial support by Fonds National de la Recherche (FNR) and by PREFALUX. If there are other authors, they declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data availability

Data are contained within the article.

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