

# The elastic and ductile behaviour of CLT wall-floor connections and the influence of fastener length

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## ARTICLE INFO

### Keywords:

Capacity based design  
Cross Laminated Timber  
Cyclic testing  
Ductility  
Engineered wood products  
Sitka spruce  
Strength degradation

## ABSTRACT

An investigation was carried out to examine the influence of the fastener length on the elastic and ductile failure behaviour of typical steel-to-timber bracket connections in Cross Laminated Timber (CLT) manufactured from C16 spruce. The behaviour of single screw-type fasteners within CLT of length 25 mm, 50 mm and 75 mm are examined under tension and shear loading. Multiple screw-type fasteners have been utilised to examine monotonic and cyclic behaviour of steel bracket-fastened CLT connections under shear and tension loading. Characteristic values of the experimental results are compared to analytical models from current timber design codes and the literature. The length of fastener has been shown to influence the failure strength and ductility of the connection. The overstrength factors and strength degradation factors have been shown to be similar, regardless of fastener length, and conservative values are recommended.

## 1. Introduction

The use of Cross Laminated Timber (CLT) for large-scale or multi-storey buildings has increased significantly in the last decade for several reasons, including sustainability considerations, aesthetic appearance and high strength-to-weight ratio of the material. CLT buildings are typically erected using balloon or platform construction methods, connecting walls and floors with angular metal brackets in combination with screws or nails. The use of such connection systems has been found to produce favourable behaviour and allow for quick and reliable installation in CLT structures. Ceccotti et al. [1] published a comprehensive report on connections in CLT as part of the SOFIE Project. In the SOFIE project and subsequent projects conducted by CNR Ivalsa, typical bracket and hold-down connection systems in CLT were examined under seismic and accidental loading actions and the significant capacity of these connection systems was demonstrated [1–5]. Many forms of fasteners have been shown to be suitable for timber-timber connections. A significant amount of research has been performed on the use of large, highly engineered self-tapping screws and threaded rods embedded in timber elements [6–14]. The recent availability on the market of self-tapping screws with continuous threads, characterised by high withdrawal capacity, has led to the development of new approaches to design, allowing connections with higher values of stiffness and load carrying capacity to be achieved. Tomasi et al. [6] investigated timber-to-timber connections with inclined screws resulting

in stiffer connections inducing a shear-compression or shear-tension load under different load modes or a combination of both when applied in a crossed configuration. Dietsch & Brandner [9] presented a series of applications for self-tapping screws as reinforcement within structural timber elements. Their applications focus on the reinforcement of timber elements perpendicular to the grain and in shear.

Current standards regulate the withdrawal and lateral load carrying resistance of fasteners in timber as a function of the fastener diameter, embedment length and of the characteristic density of the timber. Furthermore, highly engineered fasteners and brackets may be specified with the aid of European Technical Approvals (ETAs). The individual fastener characteristics specified in the relevant ETA may then be utilised within the European Yield Model (EYM) in Eurocode 5 [15]. Fig. 1 presents the failure modes for steel-to-timber connections in accordance with the EYM. The validity of the design equations, in Eurocode 5 [15] and ETAs, have been examined in relation to solid and glued laminated structures [16–18] but their relevance in CLT structures requires more attention. Blaß and Uibel [19,20] performed a large study and proposed an alternative calculation model for connections with dowel-type and screw-type fasteners in CLT, where the load-carrying capacity and the failure modes are influenced by the thickness and by the embedding strength of each board layer. While this model has not been included in structural design codes to date, a large proportion of the findings have been included in the Austrian National Annex, ÖNorm B [21].

The high in-plane stiffness of CLT panels means that the ductility

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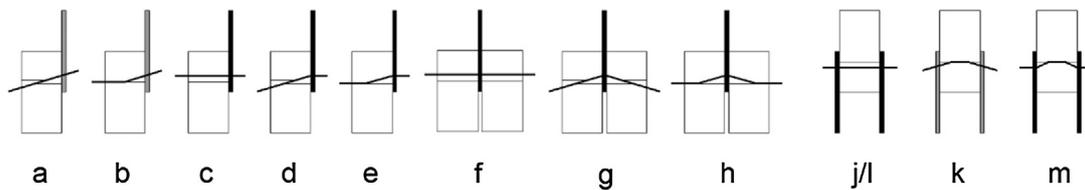


Fig. 1. Failure modes for steel-timber connections in accordance with Eurocode 5 [15].

within timber structures must be obtained through proper design of the connections [3,22]. Appropriate design of the connection system and selection of the fastener type is required to adequately dissipate energy in high load conditions. Under certain loading conditions, CLT elements can be subject to brittle failure when loaded to their maximum capacity. However, brittle failure in a structure can be avoided by utilising overstrength factors in the design of connection systems to ensure ductile failure of the connection occurs. The elastic and ductile behaviour of connections and use of different fastener types in combination with CLT have been the subject of a number of research programmes [2,8,10,13,22–28]. Izzi et al. [22] investigated the mechanical and cyclic behaviour of steel-to-timber connections with annular-ringed shank nails in CLT utilising EN 12512 [29]. Experimental tests compared mean and characteristic strength capacities to analytical predictions using the EYM Eurocode 5 [15] and the model by Blaß and Uibel [19,20]. Additionally, strength degradation factors and overstrength factors for the ductile design of such connections were proposed. A similar study was performed by Gavric et al. [2] where a large number of test configurations, simply connected with highly engineered self-tapping screws, were examined (wall-wall connections, wall-floor connections and floor-floor connections). Overstrength factors were presented for each test configuration but some configurations were still subject to brittle failure. They found that brittle failure occurred when analytical values overestimated the experimental results and they concluded that greater overstrength factors were required. Despite extensive research into the use of CLT technology, limited information is available in structural codes relating to the ductile design, overstrength factors and failure behaviour of typical connection systems.

The studies described above have examined connections in CLT manufactured from European spruce, which is normally graded as strength class C24. The increasing popularity of CLT as a construction material has led to increasing demands on timber resources. In recent studies, the use of lower grade material to manufacture CLT panels has been investigated [30–33]. As material density plays a key role in connection performance, an investigation of the connection behaviour in panels comprising lower grade material is required to determine the applicability of current and proposed design approaches. This paper investigates the behaviour of single screw-type fastener connections and multiple-fastener bracket connections for CLT manufactured using C16 grade material. As fastener length has a significant effect on the failure behaviour of screws under tension and shear loading, the influence of the fastener length is examined in each case. Other important issues related to the cyclic behaviour, ductility, stiffness, overstrength factors and strength degradation factors are also examined. The characteristic values are compared to current analytical models presented in Eurocode 5 [15] and by Blaß and Uibel [19,20].

## 2. Analytical models and design considerations

### 2.1. Intro duction

In the design of CLT structures, it is important to ensure that adequate ductility is provided within the connection. In this section, current analytical models and design considerations to determine the ductile design capacity of metallic fasteners are presented. Eurocode 5 [15] presents design rules for traditional connections for solid timber

and glued laminated timber elements using typical fasteners (nails, staples, screws, dowels and bolts). However, this standard does not include any specific design provisions for connections of CLT elements. A large body of work specific to CLT-fastened connections has been carried out by Blaß and Uibel [19,20] and an analytical model for connections utilising screws, nails and dowel-type fasteners in CLT has been proposed. This analytical model is discussed in the following subsections. Furthermore, there are no codified design rules for metallic connectors such as angle brackets and hold-downs used in CLT construction. Such connections rely on the use of ETAs. As ETAs are generally product specific these require care and attention in their implementation.

Regardless of the analytical model implemented, the CLT connection must be designed to ensure that brittle failure of the connection does not occur, but it is also important not to over-design a connection as this could result in a brittle failure of the connected timber element. Capacity-based design is commonly implemented to ensure ductile failure mechanisms occur prior to the brittle failure of a connecting timber element. This is particularly important in CLT construction given the structural rigidity of a CLT element [28]. In accordance with capacity-based design, the connection is required to provide adequate load transfer, withstand large cyclic deformations and provide stable energy dissipation under high load conditions [22,26,34]. According to Follesa et al. [35,36] and Izzi et al. [22], Eq. (1) must be verified at the ultimate limit state for a structural connection to provide adequate energy dissipation and ductile behaviour.

$$F_d \leq F_{d,ductile} \quad (1)$$

where  $F_d$  is the design load,  $F_{d,ductile}$  is the static design strength of the ductile element. The design strength of a ductile element is defined in Eq. (2) where  $F_{k,ductile}$  is the characteristic design strength,  $k_{mod}$  is the modification factor for duration of load and moisture content,  $\gamma_M$  is the partial factor for the material and  $\beta_{Sd}$  is a reduction factor to account for strength degradation of a connection under cyclic loading.

$$F_{d,ductile} = \beta_{Sd} \frac{k_{mod} F_{k,ductile}}{\gamma_M} \quad (2)$$

Values of  $F_{k,ductile}$  should be determined either by theoretical considerations or from experimental results on monotonic loaded tests. In accordance with Eurocode 8 [37], for dissipative structures the partial factor for material properties  $\gamma_M$  is equal to 1.0 for ductile elements. The design strength of the brittle component or element ( $F_{d,brittle}$ ) is calculated using Eq. (3) while adhering to Eq. (4). The condition outlined in Eq. (4) ensures that the strength of the brittle element is greater than or equal to the design strength of the ductile element ( $F_{d,ductile}$ ) multiplied by an overstrength factor,  $\gamma_{Rd}$  and divided by a reduction factor for strength degradation due to cyclic loading,  $\beta_{Sd}$  [22,35,36].

$$F_{d,brittle} = \frac{k_{mod} F_{k,brittle}}{\gamma_M} \quad (3)$$

$$\frac{\gamma_{Rd}}{\beta_{Sd}} F_{d,ductile} \leq F_{d,brittle} \quad (4)$$

The overstrength factor ( $\gamma_{Rd}$ ) is an essential component of the design of ductile elements. It accounts for an increased strength of a ductile element and reduces the risk of brittle failure occurring [25,35]. The overstrength factor ( $\gamma_{Rd}$ ) is defined as the ratio of the 95th percentile of

the experimental strength capacity,  $F_{max,95}$ , to the characteristic strength of the same element,  $F_{k,ductile}$  [5,22,25,28]. Izzi et al. [22] present the equation for  $\gamma_{Rd}$  as a function of two factors, as seen in Eq. (5).  $\gamma_{sc}$  is defined as the ratio of  $F_{max,95}$  to  $F_{max,05}$  which describes the range of strength properties determined from experimental tests on a ductile element and  $\gamma_{an}$  is a measure of agreement between the characteristic experimental strength value,  $F_{max,05}$ , and the predicted strength value of an analytical model,  $F_{k,ductile}$ .

$$\gamma_{Rd} = \frac{F_{max,95}}{F_{k,ductile}} = \frac{F_{max,95}}{F_{max,05}} \cdot \frac{F_{max,05}}{F_{k,ductile}} = \gamma_{sc} \cdot \gamma_{an} \quad (5)$$

Finally, the strength degradation factor  $\beta_{Sd}$  must be considered. This factor accounts for the impairment of strength of the dissipative element observed when subjected to cyclic loading conditions. These values are determined based on experimental tests. The strength degradation factor may be determined using Eq. (6) as reported by Izzi et al. [22].

$$\beta_{Sd} = \frac{F_{max,3rd}}{F_{max,1st}} \quad (6)$$

where  $F_{max,1st}$  and  $F_{max,3rd}$  are the maximum strength associated with the first and third envelope of the load-displacement response from cyclic tests, respectively.

## 2.2. Analytical models for axially loaded screw fasteners

In the design of timber structures, analytical models are used to calculate the capacity of fasteners ( $F_{k,ductile}$ ) in connections prior to determining the ductile design capacity. According to Eurocode 5 [15], the load carrying capacity of an axially loaded connection is usually determined as the product of a single screw resistance  $F_{ax}$  and the effective number of fasteners  $n_{ef}$ . The effective number depends on the assembly and orientation of the screws. The resistances of the screws are greatly influenced by the fastener diameter,  $d$ , embedment length,  $l_{ef}$ , and the properties of the surrounding timber,  $\rho_k$ . For axially loaded connections, withdrawal and head-pull through parameters,  $f_{ax}$  and  $f_{head}$ , respectively, together with the steel tensile capacity  $f_{ts}$ , must be considered. The characteristic tensile capacity in accordance with Eurocode 5 [15] is presented in Eq. (7).

$$F_{ax,EC5} = \frac{n_{ef} \int_{ax,k} dl_{ef}}{1.2 \cos^2 \alpha + \sin^2 \alpha} \left( \frac{\rho_k}{350} \right)^{0.8} \quad (7)$$

where  $\alpha$  is the angle between the screw axis and the grain direction with  $\alpha \geq 30^\circ$  and the other parameters are as mentioned previously. According to Eurocode 5 [15], the characteristic tensile strength of a single screw is given by Eq. (8).

$$f_{ax,k} = 0.52 \cdot d^{-0.5} \cdot l_{ef}^{-0.1} \cdot \rho_k^{0.8} \quad (8)$$

where  $d$  (mm) is the thread diameter,  $l_{ef}$  (mm) is the effective penetration length of the threaded part and  $\rho_k$  ( $\text{kg}/\text{m}^3$ ) is the characteristic density of the timber. Similar relationships have been developed in ETAs from screw and nail manufacturers [16–18]. The tension capacity for screws proposed by Blaß & Uibel [19] for fasteners in CLT is given in Eq. (9).

$$F_{ax,B\&U} = \frac{0.35d^{0.8} \rho_k^{0.75} l_{ef}^{0.9}}{1.5 \cos^2 \beta + \sin^2 \beta} \quad (9)$$

where  $d$ ,  $l_{ef}$  and  $\rho_k$  are as described above, and  $\beta$  is a parameter indicating the fasteners positioning relative to the CLT element. For fasteners positioned in the side face,  $\beta$  is  $90^\circ$  (perpendicular), and for those positioned in the narrow face,  $\beta$  is  $0^\circ$  (parallel). In addition to the model proposed by Blaß & Uibel [19], Ringhofer et al. [10,13,14] have derived a generic formula for the withdrawal capacity of axially loaded self-tapping screws. This model considers the tread-to-grain angle, the degree of homogenisation due to the number of penetrated layers and

the negative impact of gaps within a CLT lay-up.

## 2.3. Analytical models for laterally loaded screw fasteners

According to Eurocode 5 [15], the load carrying capacity of a laterally loaded connection is usually determined as the product of screw resistance  $F_v$  of a single fastener and the effective number  $n_{ef}$  of screws. In the case of shear connections, the main parameters defining the connection behaviour are the fastener diameter,  $d$ , effective penetration length,  $l_{ef}$ , the timber embedment strength,  $f_{h,k}$ , and the fastener yield moment,  $M_y$ . The characteristic lateral load carrying capacity is presented in Eq. (10).

$$F_{v,EC5} = F_{lat,k} + 0.25 \cdot F_{ax,k} \quad (10)$$

where  $F_{lat,k}$  is the lateral load capacity according to the EYM proposed by Johansen [38] and the second term is the contribution of the rope effect which is 25% of the characteristic tension capacity of the fastener. The relationship presented in Eq. (10) is also proposed by Blaß & Uibel [19]. The contribution of the rope effect varies with fastener type within Eurocode 5 [15] and can also vary within specific ETAs, for example, ETA-04/0013 [18] and ETA-11/0030 [17] increase the rope effect to 60% of the tension capacity.

The embedment strength of timber is an important factor when defining the shear behaviour of a fastener within a timber connection. According to Eurocode 5 [15], the embedment strength,  $f_{h,k}$  ( $\text{N}/\text{mm}^2$ ), for solid and glued laminated timber is dependent on the fastener diameter,  $d$  (mm), and the characteristic density,  $\rho_k$  ( $\text{kg}/\text{m}^3$ ). The embedment strength for screws with and without predrilled holes is presented in Eq. (11).

$$f_{h,k,EC5} = \begin{cases} 0.082d^{-0.3} \rho_k & \text{Without predrilled holes} \\ 0.082(1 - 0.01d) \rho_k & \text{With predrilled holes} \end{cases} \quad (11)$$

Blaß & Uibel [19,20] experimentally determined these parameters for self-tapping screws, dowels and nails for different CLT layups. Based on experimental tests [19,20], the characteristic embedment strength for screws perpendicular and parallel to the grain is defined in Eq. (12).

$$f_{h,k,B\&U} = \begin{cases} 0.112d^{-0.5} \rho_k^{1.05} & \text{Perpendicular} \\ 0.862d^{-0.5} \rho_{layer,k}^{0.56} & \text{Parallel} \end{cases} \quad (12)$$

where  $d$  (mm) is the fastener diameter,  $\rho_k$  ( $\text{kg}/\text{m}^3$ ) is the characteristic density of the panel perpendicular to the grain in the side face, and  $\rho_{layer}$  ( $\text{kg}/\text{m}^3$ ) is the characteristic density of the layer parallel to grain direction in the narrow face.

## 2.4. Connection stiffness ( $K_{ser}$ )

In this study, the experimental data from monotonic and cyclic tests on screwed CLT connections are compared with values obtained from the analytical formula for the slip modulus ( $K_{ser}$ ) of a connection between timber members given in Eurocode 5 [15]. Eurocode 5 presents an analytical model for screws, seen in Eq. (13), for the prediction of the instantaneous slip modulus for a timber-timber connection per shear plane per fastener.

$$K_{ser} = \rho_m^{1.5} d / 23 \quad (13)$$

where  $K_{ser}$  ( $\text{N}/\text{mm}^2$ ) signifies the slip modulus at the serviceability limit state,  $\rho_m$  ( $\text{kg}/\text{m}^3$ ) is the mean density of timber and  $d$  (mm) is the fastener diameter. In the case of connected timber members with different densities, the mean density is calculated as shown in Eq. (14) where  $\rho_{m,1}$  and  $\rho_{m,2}$  are the mean densities of the respective members.

$$\rho_m = \sqrt{\rho_{m,1} \rho_{m,2}} \quad (14)$$

This value of  $K_{ser}$  may be multiplied by a factor of 2.0 in the case of steel-to-timber connections in accordance with Eurocode 5 [15]. The

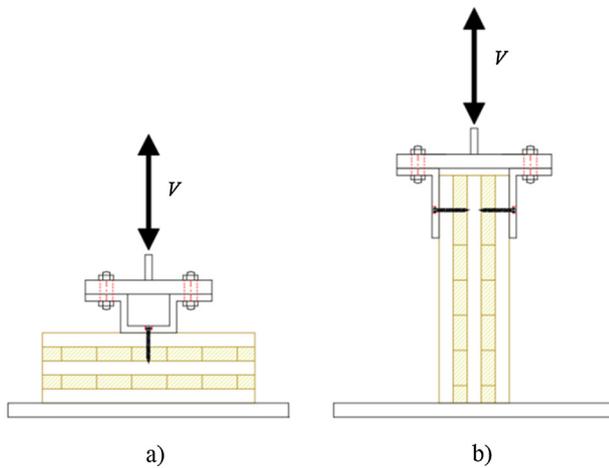


Fig. 2. Single fastener connection test in CLT panel, (a) withdrawal tension test, (b) shear test.

application of this factor to Eq. (13) for connections utilising brackets in combination with CLT will be investigated.

### 3. Experimental programme

#### 3.1. Introduction

This study examines the elastic and ductile behaviour of typical single-fastener and multiple-fastener bracket connections on CLT manufactured from C16 grade material. The influence of fastener length is examined under tension and shear loading conditions. Monotonic tests, in accordance with EN 26891 [39], were carried out on single self-tapping screws of diameter 5 mm and of varying length (25 mm, 50 mm and 75 mm) to establish if current analytical models can safely predict failure in these connections. In the next phase, multiple screw-type fasteners in combination with steel brackets were then used to connect CLT elements. These connections were subjected to cyclic tension and shear tests in accordance with EN 12512 [29] to evaluate the failure load and the strength degradation, overstrength factors associated with such connection. These connections, which utilised multiple screws of varying lengths, were then also compared to current analytical model predictions.

#### 3.2. Test materials

The CLT material utilised in this study was manufactured from C16 grade Sitka spruce. Sourced in Ireland, this timber has an average rotation length of 30–40 years [40] and is characterised as a fast-growing, low-density timber. This C16 grade timber has been shown to be suitable for the production of CLT panels [30,41]. In this study, 3-layer and 5-layer CLT panels were manufactured using C16 grade material. CLT panels were manufactured from three layers of 20 mm and 40 mm boards to give total panel thicknesses of 60 mm and 120 mm, respectively, and from five layers of 20 mm boards to give panels of 100 mm thickness. The boards were initially conditioned at a relative humidity of  $65 \pm 5\%$  and at a temperature of  $20 \pm 2^\circ\text{C}$ . Each board was planed prior to manufacturing. A one-component PUR adhesive was applied (face bonding, no edge bonding) and clamped to a pressure of  $0.6 \text{ N/mm}^2$ . The CLT panels were carefully manufactured to minimise the occurrence of gaps. The self-tapping screws and brackets, although not included in Eurocode 5 [15], may be used with the aid of ETAs. The screws and brackets used in the experimental test programme of this study are produced by Rothoblaas. The capacity of the 5 mm diameter screws (LBS) are defined in ETA-11/0030 - *Screws for use in timber construction* [17], and the angle brackets (WBR-70) for timber-to-timber

connections are in accordance with ETA-09/0323 - *Three-dimensional nailing plate (Angle brackets for timber-to-timber or timber-to-concrete of steel connections)* [42]. Screw lengths of 25 mm, 50 mm and 75 mm were chosen to examine the influence of screw length on the failure behaviour in C16 grade CLT. While ETA-09/0323 provides design values for 40 mm and 60 mm fasteners, the screw lengths of 25 mm, 50 mm and 75 mm were chosen to extend the range of screw lengths. The screw length of 25 mm was chosen as a lower limit for this test programme and while 25 mm screws may not be commonly used for bracket fastened connections, they are commercially available on the market and their suitability for use in C16 CLT required attention. The screws were positioned and fastened perpendicular to the plane in all cases. The panel configurations and screw length were varied to ensure two layers of the CLT panel were penetrated in each case. As a result, the panels thicknesses used in the tension and shear tests for the 25 mm, 50 mm and 75 mm screw lengths were 60 mm, 100 mm and 120 mm, respectively. The angle brackets used in the connection tests were fastened using four screws on the horizontal face and four screws on the vertical face. The horizontal and vertical faces of the brackets were 2 mm in thickness.

#### 3.3. Monotonic tension and shear tests on single fasteners

Monotonic tests loaded to failure were performed on 5 mm diameter screws of varying length (25 mm, 50 mm and 75 mm). These fasteners were screwed into CLT panels perpendicular to the plane to examine the behaviour of a single fastener under different loading conditions and the influence of the length of the fastener. No predrilling was carried out on any of the test specimens. Each test fastener was subjected to monotonic loading to failure in accordance with EN 26891 [39] and each specimen was prepared in accordance with EN 1382 [43]. Thirty tests specimens (10 of each fastener length) were subjected to tension tests under tension loading (Fig. 2a), thirty tests specimens (10 of each fastener length) were subjected to shear loading perpendicular to the grain (outermost layer) and thirty tests specimens (10 of each fastener length) were subjected to shear loading parallel to the grain (Fig. 2b). CLT specimens were stored and tested under controlled environmental conditions at a relative humidity of  $65 \pm 5\%$  and a temperature of  $20 \pm 2^\circ\text{C}$ . Test specimens were anchored to the test bed using steel plates and threaded bar. The steel plates were over-designed to ensure minimal deformation of the CLT occurred during each test. This was verified with additional dial gauge measurements (omitted from Fig. 2 for clarity) during the test. Monotonic tests were carried out by displacement controlled loading at a rate varying from 0.05 to 0.2 mm/s in accordance with EN 26891 [39]. Displacement was measure through cross-head movement of the test frame.

The typical load-deformation response to failure can be seen in

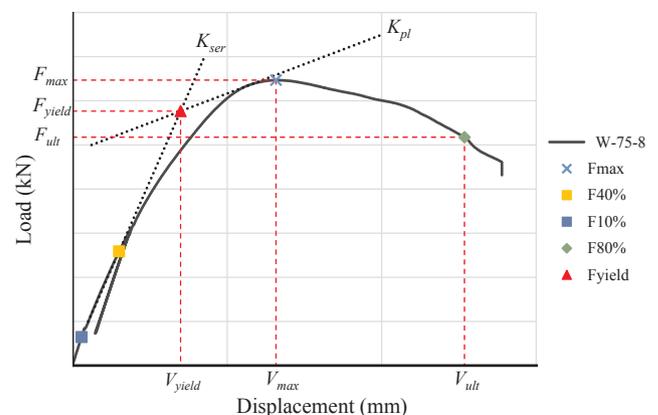


Fig. 3. Analysis of experimental data (W-75-8) to determine  $F_{max}$ ,  $F_{yield}$  and  $F_{ult}$  in accordance with EN 12512 [29].

Fig. 3, which indicates the maximum load ( $F_{max}$ ) and maximum displacement ( $V_{max}$ ), the yield load ( $F_{yield}$ ) and yield displacement ( $V_{yield}$ ), and the ultimate load ( $F_{ult}$ ) and ultimate displacement ( $V_{ult}$ ) [29]. The value of the ultimate displacement and corresponding value of ultimate load is determined as either the displacement at failure, a displacement corresponding to 80% of the maximum load or a limiting ultimate failure displacement of 30 mm, whichever occurs first. The instantaneous elastic behaviour of the connection also known as  $K_{ser}$  is the slope of the line between 10% and 40% of  $F_{max}$ . The slope of the plastic behaviour is indicated by  $K_{pl}$ . In accordance with EN 12512 [29],  $K_{pl}$  is defined as the slope of the plastic displacement of a load-displacement curve comprising two well-defined linear parts or alternatively,  $K_{pl}$  may be defined as a tangent line to the curve with a slope equal to  $K_{ser}/6$ . In this study, the load-displacement curves did not comprise two well-defined linear parts and  $K_{pl}$  is determined from a tangent line to the curve with a slope equal to  $K_{ser}/6$ . The intersection between  $K_{ser}$  and  $K_{pl}$  represents the yield load and yield displacement. The ductility (*Duc.*) of the connection is defined as shown in Eq. (15). After each test, the moisture content and density of the CLT specimens were measured in accordance with EN 13183-1 [44].

$$Duc. = \frac{V_{ult}}{V_{yield}} \tag{15}$$

### 3.4. Cyclic wall-floor connection tests

In both tension and shear loading, the connection was formed using four 5 mm diameter LBS screws [17] in the vertical and in the horizontal face of WBR-70 brackets [42] with only the screw length varying. The tension connection test set-up and the shear connection test set-up can be seen in Fig. 4a and Fig. 4b, respectively. In the tension tests (Fig. 4a), screws on the horizontal face are subjected to a tension load and screws on the vertical face to a shear load. The shear test set-up (Fig. 4b) subjects each screw to a shear load on the horizontal and vertical faces of the bracket. Threaded bar and steel plates were used to anchor the CLT elements to the test frame and load head. The threaded bar and steel plates were sized to minimise localised deformations in the CLT. The standard procedure for cyclic testing of connections made with mechanical fasteners, prescribed by EN 12512 [29], was followed, with an input displacement rate varying from 0.05 to 0.2 mm/s. Firstly, monotonic tests on CLT connections were carried out by displacement

controlled loading at a rate varying from 0.05 to 0.2 mm/s in accordance with EN 26891 [39] to establish the predefined yield displacement ( $V_{yield}$ ) of each connection configuration. The hysteretic loop or cyclic displacement schedule in accordance with EN 12512 [29] is applied as a function of the mean value of yield displacement or slip obtained in the monotonic tests. All cyclic shear tests were then conducted using a reversed cyclic loading procedure with the predefined yield values. All cyclic tension tests were subjected to a non-reversed modification of the procedure outlined in EN 12512 [29] as outlined by Ceccotti et al. [3]. The displacement was cycled from zero to a positive (tensile) value without the need to induced negative (compression) values.

The equivalent viscous damping ratio,  $v_{eq,i}$ , expresses the hysteresis damping properties of the connection and is defined as the ratio of the dissipated energy in the cycle  $i$  ( $E_{d,i}$ ) to the available potential energy in that cycle ( $E_{p,i}$ ) multiplied by  $4\pi$  as seen in Eq. (16). The dissipated energy  $E_{d,i}$  is the area of the hysteretic loop per half cycle as presented in EN 12512 [29]. The available potential energy  $E_{p,i}$  can be determined as  $E_{p,i} = \frac{1}{2}F_i V_i$ , where  $F_i$  and  $V_i$  are the maximum load and maximum displacement attained in cycle  $i$ , respectively [5].

$$v_{eq,i} = \frac{E_{d,i}}{4\pi E_{p,i}} \tag{16}$$

### 3.5. Statistical analysis

Statistical methods have been implemented to examine the distribution of the experimental strength results. Characteristic values (5th percentile) have been calculated from experimental test results in accordance with EN 14358 [45]. The characteristic value ( $x_{05}$ ) is calculated using Eq. (17) where  $\bar{\mu}$  is the mean value and  $\bar{\sigma}$  is the standard deviation corrected by the factor,  $k_s$ , which is dependent upon the sample size and is given in EN 14358 [45].

$$x_{05} = \exp(\bar{\mu} - k_s \cdot \bar{\sigma}) \tag{17}$$

The 95th percentile is similarly calculated by inverting the sign of the  $k_s$  factor.

Statistical methods were also utilised to ensure the material properties of the CLT, namely density and moisture content were similar for all groups tested. Student’s t-tests were carried out to compare the means of each sample to one another. All statistical tests were carried

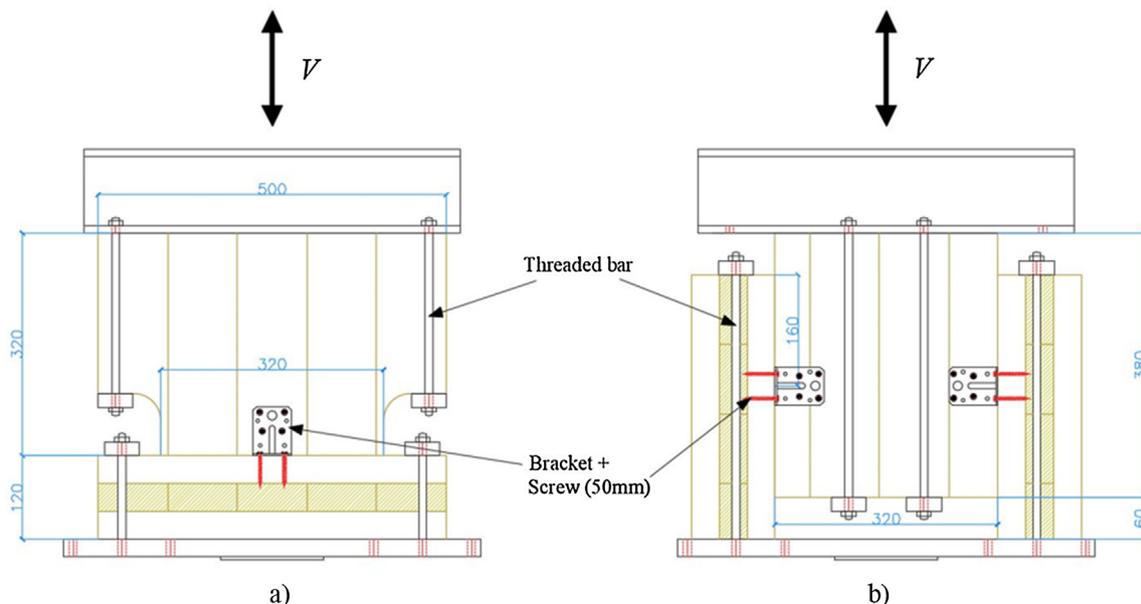


Fig. 4. Cyclic connection test set-up, (a) tension connection test set-up, (b) shear connection test set-up.

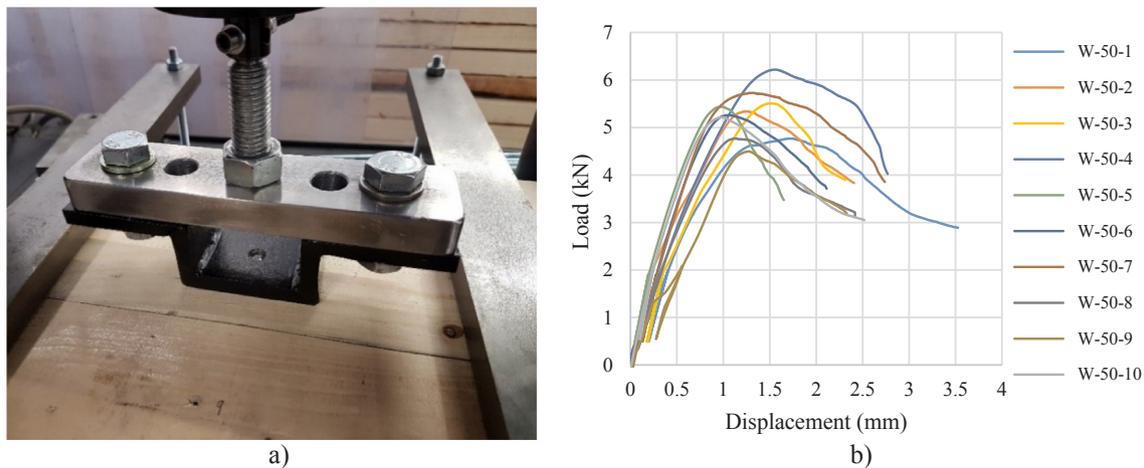


Fig. 5. Tension test: (a) tension test set-up (specimen W-50-10) prior to testing, (b) load-displacement behaviour of 50 mm screw tension tests.

out to a significance level of 0.95 or  $\alpha = 0.05$  and it was shown that there was no statistically significant difference observed between the means of each group when examining density and moisture content.

#### 4. Experimental test results

##### 4.1. Fastener tension test results

The monotonic tension test set-up, in accordance with EN 26891 [39], can be seen in Fig. 5a. All test specimens failed through screw withdrawal regardless of screw length. The experimental load to failure test results for 50 mm long screws subjected to tension load can be seen in Fig. 5b and this type of response is seen for all lengths tested. As seen in Fig. 5b, the load-displacement behaviour was linear elastic until the yield load was reached. Thereafter, the load continues to increase until the maximum load is achieved followed by a decrease in load until ultimate failure.

The maximum load ( $F_{max}$ ), yield load ( $F_{yield}$ ), ultimate load ( $F_{ult}$ ), max displacement ( $V_{max}$ ), yield displacement ( $V_{yield}$ ), ultimate displacement ( $V_{ult}$ ), elastic stiffness ( $K_{ser}$ ), plastic stiffness ( $K_{pl}$ ) and monotonic connection ductility ( $Duc. m.$ ) are presented in Table 1, in addition to the density ( $\rho$ ) and moisture content ( $u$ ) of the CLT timber. The test results show that a mean maximum load of 1452.2 N, 5273.4 N and 7795.4 N was achieved for the 25 mm, 50 mm and 75 mm screws, respectively. This increasing trend with increasing screw length represents an increase of 263% in mean maximum load, when comparing the results of 50 mm long screws to 25 mm long screws and an increase

Table 1  
Experimental tension test results on fasteners of length 25 mm, 50 mm and 75 mm and properties of CLT specimens used in the tests.

Property	Fastener length					
	25 mm (n = 10)		50 mm (n = 10)		75 mm (n = 10)	
	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.
$F_{max}$ (N)	1452.2	296.8	5273.4	506.4	7795.4	925.1
$F_{yield}$ (N)	1319.0	307.9	4743.3	480.6	7292.3	975.8
$F_{ult}$ (N)	1161.8	237.4	4218.7	405.1	6236.3	740.1
$V_{max}$ (mm)	0.61	0.11	1.29	0.25	1.60	0.35
$V_{yield}$ (mm)	0.28	0.07	0.76	0.17	1.16	0.32
$V_{ult}$ (mm)	1.12	0.22	2.04	0.41	2.26	0.31
$K_{ser}$ (kN/mm)	4.88	1.51	6.58	1.43	6.43	1.07
$K_{pl}$ (kN/mm)	0.42	0.17	1.03	0.22	1.15	0.28
$Duc. m.$ (-)	4.38	1.82	2.73	0.46	2.04	0.58
$\rho$ (kg/m <sup>3</sup> )	405	19	431	25	420	20
$u$ (%)	12.4%	1.1%	13.4%	0.4%	13.3%	1.0%

of 48% in mean maximum load when comparing the results of 75 mm long screws to 50 mm long screws. This trend of increasing load capacity with increased fastener length was also observed when examining the yield load, ultimate load, elastic stiffness and plastic stiffness. However, increasing fastener length resulted in a decrease in ductility. A mean ductility of 4.38, 2.73 and 2.04 was observed for fasteners of 25 mm, 50 mm and 75 mm length, respectively. This corresponds to a 60% reduction when the results of 50 mm long screws are compared to 25 mm long screws. A further reduction of 34% was found when comparing the 75 mm screws to 50 mm long screws.

##### 4.2. Fastener shear test results

The monotonic shear test set-up, in accordance with EN 26891 [39], can be seen in Fig. 6a. Failure modes in the shear tests on single fasteners were found to resemble failure mode “e” (Fig. 1) through combined embedment and yielding of the fastener in all cases for 50 mm and 75 mm long screws. The 25 mm screws resembled failure mode “a” with no yielding of the screw. In some cases, screw fracture occurred, regardless of fastener length, but no brittle failure modes were observed.

The load-displacement behaviour of the 75 mm fasteners shown in Fig. 6b is typical of the response for all fastener lengths. The results of the shear tests loaded parallel and perpendicular to the grain, are tabulated in Table 2 and Table 3, respectively. In experimental shear tests two screws were simultaneously loaded, and the tabulated results are based on single screw values. Testing of two screws in a symmetric manner is essential to avoid twisting of the test specimen or apparatus and avoid eccentric loading of the screw. As such, the values presented may be subject to a homogenising or group effect as the two screws engage, deform and embed within the layers of the CLT. The experimental results show increasing mean maximum load with increasing fastener length regardless of the orientation to the grain. Similar trends were observed for the mean yield load and ultimate failure load.

Similar to the tension tests on single screws, the ductility measurement has been shown to decrease with increasing fastener length. The mean values for ductility are generally lower for fasteners loaded perpendicular to the grain. The mean ductility measurement ranged from a mean maximum of 12.13 for 25 mm screws to a mean minimum of 3.62 for 75 mm screws parallel to the grain. Perpendicular to the grain, the mean ductility measurement ranged from a mean maximum 8.48 for 25 mm screws to a mean minimum of 2.26 for 75 mm screws. The ductility was found to be larger for screws loaded in shear when compared to screws loaded in tension. For screws loaded in shear, significant deformation occurred after the yield point prior to failure. Screws loaded in tension were significantly less ductile than those

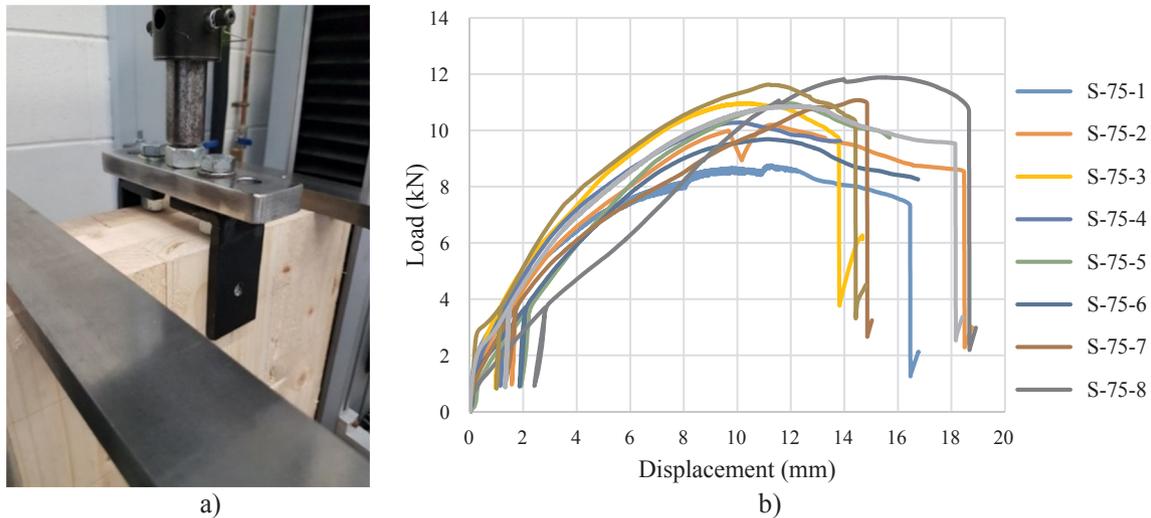


Fig. 6. Shear test: (a) shear test set-up loaded parallel to the grain, (b) load-displacement behaviour of 75 mm screw shear tests parallel to the grain.

Table 2

Experimental shear test results on single fasteners parallel to the grain direction, and properties of CLT specimens used in the tests.

Property	Fastener length					
	25 mm (n = 10)		50 mm (n = 10)		75 mm (n = 10)	
	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.
$F_{max}$ (N)	1768.1	100.1	4034.5	574.3	5314.7	464.0
$F_{yield}$ (N)	1464.8	129.7	3446.1	617.9	4518.4	506.0
$F_{ult}$ (N)	1495.4	166.1	3227.6	459.4	4698.9	618.0
$V_{max}$ (mm)	5.14	0.90	4.09	1.01	5.97	0.86
$V_{yield}$ (mm)	0.84	0.76	1.23	0.44	2.50	1.10
$V_{ult}$ (mm)	6.27	0.67	5.04	0.81	8.05	0.94
$K_{ser}$ (kN/mm)	1.40	0.87	1.39	0.39	0.92	0.24
$K_{pl}$ (kN/mm)	0.04	0.01	0.11	0.03	0.11	0.03
$Duc. m.$ (-)	12.13	7.08	4.52	1.63	3.62	1.23
$\rho$ (kg/m <sup>3</sup> )	420	23	425	20	383	26
$u$ (%)	12.3%	0.7%	13.0%	0.5%	13.1%	0.6%

Table 3

Experimental shear test results on single fasteners perpendicular to the grain direction, and properties of CLT specimens used in the tests.

Property	Fastener length					
	25 mm (n = 10)		50 mm (n = 10)		75 mm (n = 10)	
	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.
$F_{max}$ (N)	2102.5	400.0	3876.6	666.3	5352.4	676.6
$F_{yield}$ (N)	1605.4	279.8	3169.6	546.3	4631.0	702.2
$F_{ult}$ (N)	1766.5	415.0	3594.0	859.1	5095.5	802.0
$V_{max}$ (mm)	4.59	0.97	3.83	1.30	5.92	1.61
$V_{yield}$ (mm)	0.95	0.68	1.26	0.59	3.30	1.44
$V_{ult}$ (mm)	5.86	1.02	5.03	1.08	6.91	2.27
$K_{ser}$ (kN/mm)	0.92	0.34	1.39	0.57	0.77	0.41
$K_{pl}$ (kN/mm)	0.08	0.07	0.15	0.09	0.16	0.12
$Duc. m.$ (-)	8.48	4.55	4.78	2.19	2.26	0.60
$\rho$ (kg/m <sup>3</sup> )	427	22	420	35	415	50
$u$ (%)	12.2%	0.6%	12.1%	0.4%	13.1%	0.7%

loaded in shear with significant drop off in load after reaching  $F_{max}$ .

4.3. Cyclic test on wall-floor connections

The experimental cyclic test results of typical bracket connections in accordance with EN 12512 [29] are presented. Fasteners of length

25 mm, 50 mm and 75 mm were utilised in CLT connections to examine their influence on the strength, stiffness, ductility and energy dissipation behaviour of such connections. The values presented for the tension connection tests were analysed taking into account the positive side of the hysteretic loop only as negative displacement did not occur during the test. In contrast, the values presented for the shear connection tests considered both the positive and negative side of the hysteretic loop.

4.3.1. Tension connection tests results

All cyclic tests on tension specimens demonstrated failure of screws on the floor side of the connection due to the vertical displacement of the panel. This behaviour was observed for all specimens regardless of screw length. Significant bending deformation could be observed in the bracket, but generally, failure was dictated by either withdrawal of the screw, tearing of the screw head (Fig. 7b) or by a combination of screw withdrawal and screw bending. This behaviour is commonly seen in typical bracket connections when subjected to tension loading situations [23].

The hysteretic loop or cyclic behaviour of a tension specimen (T-75-3) can be seen in Fig. 7c. The first, second and third cycle backbone curves represent the peak loads associated with the first, second and third repetition for each cycle subjected to a prescribed connection slip in accordance with EN 12512 [29]. The load-displacement behaviour of the monotonic test specimen is also shown for comparison. The analysis of the first cycle backbone curve, as seen in Fig. 7c, allows the maximum load ( $F_{max}$ ), maximum displacement ( $V_{max}$ ), the yield load ( $F_{yield}$ ), yield displacement ( $V_{yield}$ ), the ultimate load ( $F_{ult}$ ), ultimate displacement ( $V_{ult}$ ), the elastic stiffness ( $K_{ser}$ ), plastic stiffness ( $K_{pl}$ ) and connection ductility ( $Duc.$ ) to be determined for each specimen. Further analysis of the backbone curves allows the impairment in strength ( $\Delta F_{1-3}$ ), the equivalent viscous damping ratio for the first ( $\nu_{eq(1st)}$ ) and third ( $\nu_{eq(3rd)}$ ) cycle groups and the strength degradation factor ( $\beta_{sd}$ ) of each specimen to be calculated.

In total, twenty tension connection specimens were subjected to cyclic loading in accordance with EN 12512 [29]. This comprised five specimens utilising 25 mm screws, seven specimens utilising 50 mm screws and eight specimens utilising 75 mm screws. The experimental test results for the tension connections are tabulated in Table 4. The experimental results have shown the positive influence of increased fastener length with mean maximum loads of 6913.7 N, 17451.4 N and 21978.6 N achieved for the 25 mm, 50 mm and 75 mm screws, respectively. Although the increased fastener length has achieved greater mean load carrying capacities under tension, the mean elastic

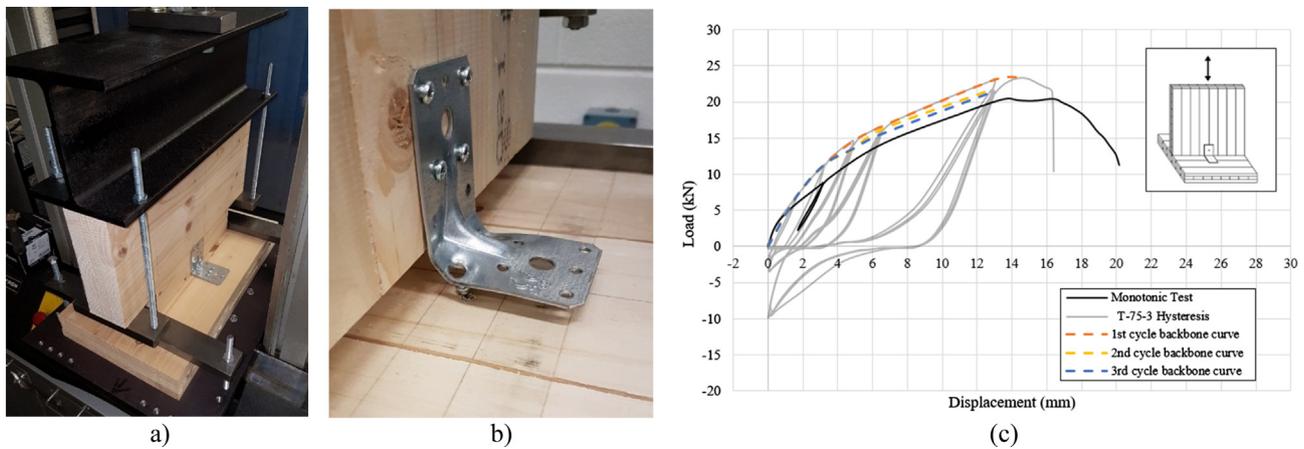


Fig. 7. Cyclic CLT tension connection test, (a) CLT connection test set-up, (b) typical withdrawal behaviour, (c) Hysteresis loop of tension test specimen, T-75-3, with associated backbone curves and monotonic test curve.

connection stiffness ( $K_{ser}$ ) appears to decrease with values of 3.44 kN/mm, 3.20 kN/mm and 2.82 kN/mm being achieved for 25 mm, 50 mm and 75 mm fasteners, respectively.

The mean connection ductility values for all fastener lengths was found to be quite similar and there does not appear to be any influence due to the fastener length. The mean strength degradation factor also appears to remain relatively constant regardless of fastener length with values of 0.91, 0.89 and 0.91 for 25 mm, 50 mm and 75 mm long fasteners, respectively.

4.3.2. Shear connection tests results

In the shear connection specimens, deformation of the steel bracket was observed on the floor and wall side of each connection, leading to failure due to local buckling of the bracket and screw failure. The combined bending of the bracket and embedment and withdrawal of screws can be seen in Fig. 8b. Bending of the screws was also observed in both the floor and wall side of each connection. For the 25 mm long screws, failure was due to embedment and withdrawal. For the 50 mm and 75 mm long screws, bending failure was observed in addition to embedment and withdrawal failure. The typical cyclic behaviour of a shear connection (S-25-4) can be seen in Fig. 8c. The first, second and third cycle backbone curves in tension and compression are plotted in addition to the load-displacement behaviour of the monotonic test specimen for comparison. In total, twenty shear connection test were

subjected to cyclic shear loading in accordance with EN 12512 [29].

The experimental test results for the shear connections are summarised in Table 5. Similar to the tension connection results, the mean maximum failure load of the shear connection increases with increased fastener length. The elastic stiffness was found to be similar for all fastener lengths tested. The mean ductility values were found to decrease with increased fastener length with a mean maximum ductility of 5.35 for 25 mm screws to a mean minimum of 4.20 for 75 mm screws.

The mean strength degradation factor remains relatively constant regardless of fastener length with values of 0.81, 0.84 and 0.82 for the 25 mm, 50 mm and 75 mm long fasteners, respectively. The tension connections also experienced relatively constant albeit higher strength degradation factors for different fastener lengths. The strength degradation factor appears to be dependent on the loading condition rather than the screw length. Experimental characteristic values of 0.84 and 0.75 have been calculated for the tension and shear loading condition in accordance with EN 14358 [45] by combining all strength degradation results regardless of screw length. Based on this finding, conservative characteristic strength degradation factors of 0.80 and 0.70 are recommended for steel-to-timber connections utilising fasteners under tension and shear loading, respectively.

Table 4

Experimental cyclic tension test results of bracket fastener CLT connections utilising fasteners of length 25 mm, 50 mm and 75 mm, and properties of CLT specimens used in the tests.

Property	Fastener length					
	25 mm (n = 5)		50 mm (n = 7)		75 mm (n = 8)	
	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.
$F_{max}$ (N)	6913.7	1149.7	17451.4	2121.0	21978.6	2084.1
$F_{yield}$ (N)	5427.2	1171.5	12530.9	1272.7	13723.6	2864.6
$F_{ult}$ (N)	5503.6	927.3	13961.1	1696.8	17582.9	1667.3
$V_{max}$ (mm)	3.77	1.04	10.08	2.23	14.76	2.23
$V_{yield}$ (mm)	1.60	0.64	3.78	0.52	5.11	1.27
$V_{ult}$ (mm)	5.16	1.36	13.20	2.21	17.06	2.66
$K_{ser}$ (kN/mm)	3.44	0.48	3.20	0.53	2.82	0.55
$K_{pt}$ (kN/mm)	0.70	0.12	0.79	0.12	0.71	0.11
Duc. (-)	3.38	0.81	3.52	0.51	3.43	0.54
Duc. m. (-)	4.12	-	3.17	-	2.87	-
$\rho_m$ (kg/m <sup>3</sup> )	405	18	411	25	410	16
$\Delta F_{1-3}$ (%)	9.10	3.47	13.71	9.01	10.04	4.64
$\nu_{eq(1st)}$ (%)	14.44	1.14	12.69	1.04	12.21	0.48
$\nu_{eq(3rd)}$ (%)	11.49	1.45	7.95	0.68	7.24	0.40
$\beta_{Sd} = F_{max,3rd}/F_{max,1st}$	0.91	0.03	0.89	0.05	0.91	0.04

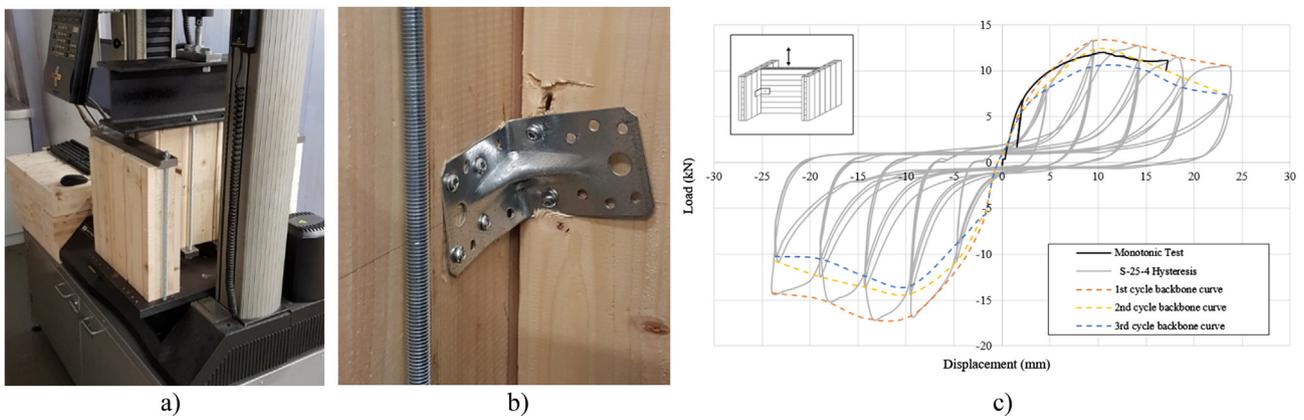


Fig. 8. Cyclic CLT shear connection test, (a) CLT connection test set-up, (b) typical embedment and withdrawal behaviour, (c) Hysteresis loops of the shear test specimen, S-25-4, with associated backbone curves and monotonic test curve.

Table 5

Experimental cyclic shear test results of bracket fastener CLT connections utilising fasteners of length 25 mm, 50 mm and 75 mm, and properties of CLT specimens used in the tests.

Property	Fastener length					
	25 mm (n = 5)		50 mm (n = 7)		75 mm (n = 8)	
	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.
$F_{max}$ (N)	12432.4	530.8	26579.2	1990.4	27383.1	1878.5
$F_{yield}$ (N)	8866.1	593.1	17811.1	677.5	17821.3	2919.1
$F_{ult}$ (N)	10246.8	606.4	22600.9	3183.8	22189.9	3008.5
$V_{max}$ (mm)	11.60	2.42	20.11	2.80	21.81	4.96
$V_{yield}$ (mm)	3.30	0.98	4.55	0.85	7.18	4.48
$V_{ult}$ (mm)	16.77	3.61	25.54	5.18	27.81	4.57
$K_{ser}$ (kN/mm)	3.18	0.81	3.33	0.74	3.23	1.14
$K_{pt}$ (kN/mm)	0.46	0.21	0.60	0.09	0.66	0.24
Duc. (-)	5.35	1.60	4.99	1.01	4.20	1.62
Duc. m. (-)	7.16	-	5.03	-	3.56	2.32
$\rho_m$ (kg/m <sup>3</sup> )	412	27	419	21	421	26
$\Delta F_{1-3}$ (%)	21.16	5.72	17.38	6.19	19.51	4.93
$\nu_{eq(1st)}$ (%)	10.21	1.17	10.76	2.27	10.22	0.89
$\nu_{eq(3rd)}$ (%)	6.60	0.87	7.99	2.01	7.10	0.99
$\beta_{Sd} = F_{max,3rd}/F_{max,1st}$	0.81	0.05	0.84	0.05	0.82	0.04

#### 4.4. Experimental and analytical model comparison

##### 4.4.1. Single fastener tests

The experimental results from the monotonic tension and shear tests are compared to analytical models presented in Eurocode 5 [15] and Blaß & Uibel [19]. The characteristic 5th percentile strength values have been calculated from experimental test results in accordance with EN 14358 [45]. In Fig. 9, the experimental results, characteristic 5th percentile strength and analytically determined strength values for each fastener length are presented. The experimentally determined increase in capacity with increasing screw length is predicted by both analytical models. The experimental characteristic strength values are higher than those provided by the analytical models for the 50 mm and 75 mm screws. However, the analytical models over predict the tension capacity of the 25 mm screws. This was found to be the case when examining the results and comparing the model developed by Ringhofer et al. [10,13,14] when considering CLT of this density. The model calculated values approximately equal to those provided by the Blaß & Uibel [19].

In Fig. 10a and b, the experimental results, characteristic 5th percentile strength and analytical design shear strength values for 25 mm, 50 mm and 75 mm long screws loaded parallel and perpendicular to the grain, respectively, are presented. The general increase in strength with increasing screw length, which was observed experimentally, is predicted by both analytical models. In each case, the characteristic

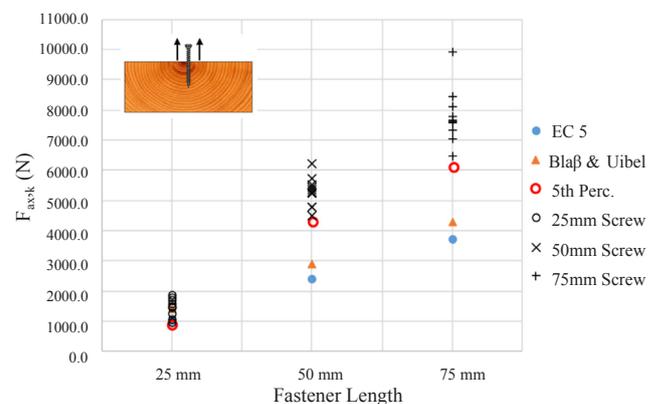


Fig. 9. Comparison between analytical models and the experimentally determined maximum failure load for different fastener lengths -tension tests.

strength values are in excess of that predicted from the analytical model prescribed in Eurocode 5 [15]. The analytical model prescribed by Blaß & Uibel [19] over-predicts the shear strength of the 25 mm long screws when tested parallel and perpendicular to the grain. Overall, results from the Blaß & Uibel model [19] are closer to the experimental results than the Eurocode 5 model [15], which presents more conservative capacity values. This finding is not surprising as the model by Blaß &

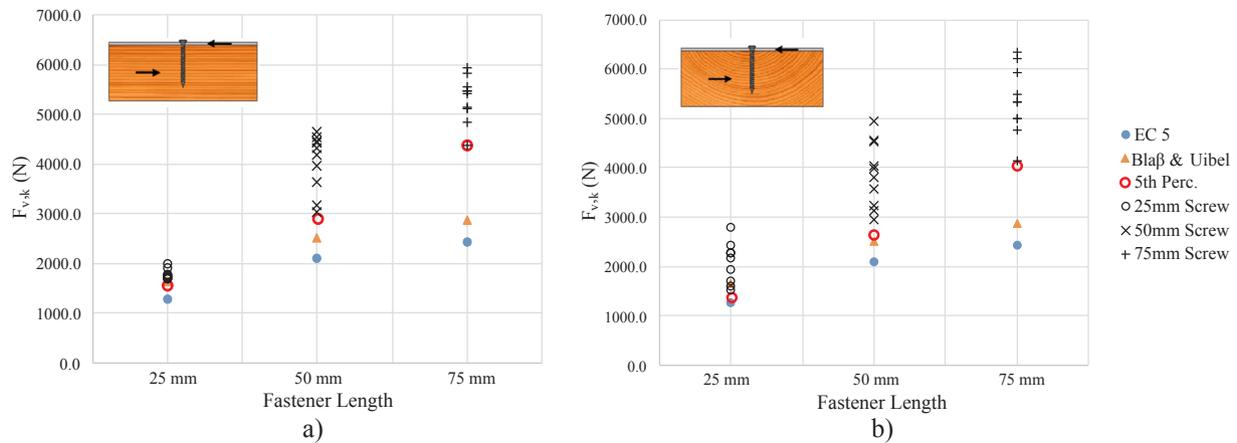


Fig. 10. Comparison between analytical models and the experimentally determined maximum failure load for different fastener lengths, (a) shear test parallel to the grain, (b) shear test perpendicular to the grain.

Uibel [19] was specifically developed for CLT. These results validate their suitability when examining connections utilising lower density CLT, with the exception of 25 mm long screws.

In Table 6, the analytical models are compared to the characteristic 5th percentile values. The ratio  $F_{ax,k,EC5}$  to  $F_{max,05}$  is used to determine the accuracy of the Eurocode 5 [15] model compared to the experimental test results. A value less than 1.0 indicates a conservative design value and a value greater than 1.0 indicates an over-prediction of the design load. All values calculated with the Eurocode 5 [15] model result in values less than 1.0 (0.55–0.93) with the exception of the 25 mm screws under tension loading (1.18). The model provided by Blaß & Uibel [19] produced similar results. The ratio  $F_{ax,k,B\&U}$  to  $F_{max,05}$  is used to determine the accuracy of their model compared to the experimental test results and it can be seen that values in excess of 1.0 (1.55, 1.04 and 1.20) were achieved for the 25 mm screws under tension and shear loading, both parallel and perpendicular to the grain. The 50 mm and 75 mm resulted in values less than 1.0, ranging from 0.65 to 0.95, demonstrating the suitability of the model when examining screws of these lengths. Until further test results are available, it is not recommended to use the Blaß & Uibel [19] model for screw lengths less than 50 mm for C16 CLT panels.

4.4.2. Cyclic connection tests

The experimental results of the cyclic tests on tension and shear bracket connections are compared with analytical model predictions presented in Eurocode 5 [15] and by Blaß & Uibel [19] in Fig. 11a and Fig. 11b, respectively. It can be seen that the experimental characteristic strength values are higher than those provided by the analytical models for the 50 mm and 75 mm screws. However, the analytical model presented by Blaß & Uibel [19] and the model developed by Ringhofer et al. [10,13,14] over-predict the tension capacity of the

connection utilising 25 mm screws. This is also the case when examining the experimental, characteristic and analytical design shear strength capacity of all screw lengths subjected to shear loading as seen in Fig. 11b, however, it must be noted that screws of this length were not experimentally examined by Blaß & Uibel [19] or Ringhofer et al. [10,13,14].

The models are compared to the characteristic strength capacities in Table 7. The experimental results have demonstrated characteristic strengths in excess of those calculated using the analytical model presented in Eurocode 5 [15] under tension and shear loading. The ratio  $F_{ax,k,EC5}$  to  $F_{max,05}$  is used to determine the accuracy of the model compared to the experimental test results. All values calculated with the Eurocode 5 [15] model result in values less than 1.0 (0.74–0.91). The Blaß & Uibel model gives non-conservative values for the 25 mm screws under both shear and tension loading. The 50 mm and 75 mm resulted in values very close to 1.0, ranging from 0.89 to 0.98, demonstrating more conservative strength predictions when examining screws of this length.

The experimental elastic stiffness of the tension and shear connections are compared to the analytical model ( $K_{el,EC5}$ ) in Eurocode 5 [15] in Table 8. The elastic stiffness, as defined in Eurocode 5 [15], signifies the slip modulus at the serviceability limit state and is a function of the mean density of the timber element and the fastener diameter. See Eqs. (13) and (14).

The experimentally determined stiffness was lower in all cases when compared to the analytical model. Eurocode 5 [15] states that the elastic stiffness may be increased by a factor of 2.0 for steel-to-timber connections. This 2.0 factor is applied to the theoretical stiffness calculated in this study. This theoretical analytical stiffness is not attained by the experimental results presented.

Table 6 Comparison between analytical models and the experimentally determined characteristic failure load.

Test	Fastener Length (mm)	$F_{max,05}$ (N)	$F_{ax,k,EC5}$ (N)	$\frac{F_{ax,k,EC5}}{F_{max,05}}$	$F_{ax,k,B\&U}$ (N)	$\frac{F_{ax,k,B\&U}}{F_{max,05}}$
Tension (n = 10)	25 mm	896.0	1061.7	1.18	1389.0	1.55
	50 mm	4440.4	2388.9	0.54	2881.8	0.65
	75 mm	6102.0	3716.1	0.61	4289.0	0.70
Shear parallel (n = 10)	25 mm	1574.0	1269.8	0.81	1643.0	1.04
	50 mm	2903.9	2096.4	0.72	2517.3	0.87
	75 mm	4380.6	2428.2	0.55	2869.1	0.65
Shear perpendicular (n = 10)	25 mm	1371.6	1269.8	0.93	1643.0	1.20
	50 mm	2657.8	2096.4	0.79	2517.3	0.95
	75 mm	4052.2	2428.2	0.60	2869.1	0.71

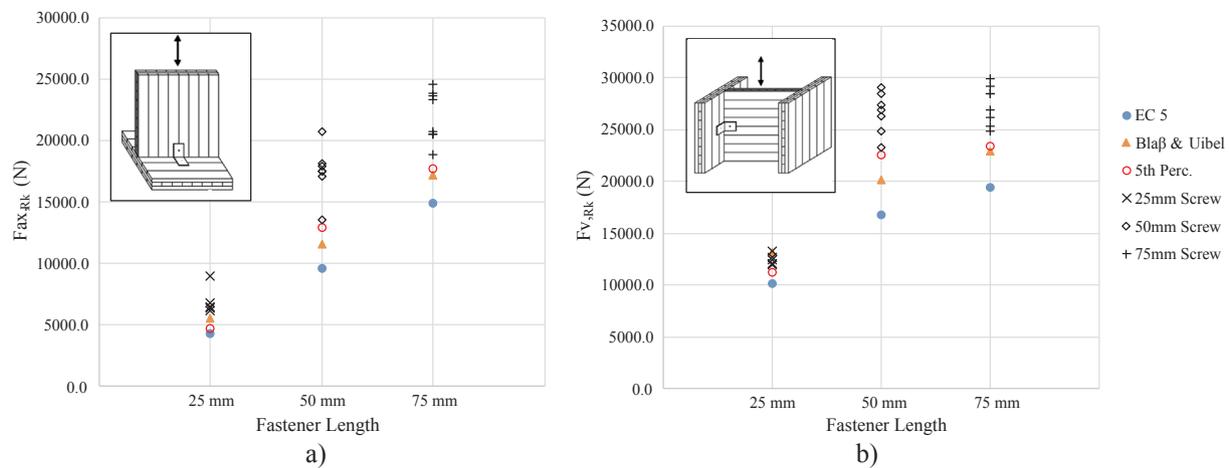


Fig. 11. Comparison between analytical models and the experimentally determined maximum tension bracket connection failure load for different fastener lengths, (a) Tension connection, (b) shear connection.

4.4.3. Overstrength factor

The overstrength factors ( $\gamma_{RD}$ ) for all fastener lengths subjected to tension and shear loading parallel and perpendicular to grain are presented in Table 9. The overstrength factor is considered in the design of ductile connection systems to ensure ductile failure occurs before brittle failure of the adjoining brittle member. The overstrength factors are calculated using Eq. (5) where  $\gamma_{an}$  is determined using the Eurocode 5 model or the Blaß & Uibel model. For tension tests, Table 9 shows an increase in overstrength factor when the screw length increases from 25 mm to 50 mm, but no further increase is found for the 75 mm length. The value of  $\gamma_{an}$ , which is an indicator of the accuracy of the model compared to the experimental result, is particularly high (1.86 for the Eurocode 5 model and 1.54 for the Blaß & Uibel model) for the 50 mm screws. This may be due to the higher density of these CLT specimens compared to those for the other screw lengths (Table 1). The overstrength values for shear tests, parallel and perpendicular to the grain, can be seen in Table 9. The general trend from the experimental results indicates an increasing overstrength factor with increased screw length regardless of orientation to the grain. The overstrength values presented for the Blaß & Uibel model are lower than those provided by the Eurocode 5 model in all cases. This is due to the higher analytical values determined from the Blaß & Uibel model. However, as the Blaß & Uibel model gives non-conservative predictions for the strength of 25 mm screws under tension and shear loading, it is recommended to limit the use of these overstrength values to screw lengths of 50 mm and greater when using C16 grade timber.

The overstrength factors for the bracket connections tests for all fastener lengths subjected to tension and shear loading are presented in Table 10. For bracket connections subjected to tension loading utilising 25 mm, 50 mm and 75 mm screws, the overstrength factors determined from the Eurocode 5 model are 2.11, 2.44 and 1.83, respectively. The corresponding overstrength values determined from the Blaß & Uibel model are 1.61, 1.12 and 1.58, respectively. There does not appear to

Table 7

Comparison between analytical models and the experimentally determined characteristic failure loads.

Test	Fastener length (mm)	n	$F_{max,05}$ (N)	$F_{ax,k,EC5}$ (N)	$\frac{F_{ax,k,EC5}}{F_{max,05}}$	$F_{ax,k,U\&B}$ (N)	$\frac{F_{ax,k,B\&U}}{F_{max,05}}$
Shear	25	5	11195.2	10158.6	0.91	13144.0	1.17
	50	7	22523.2	16771.0	0.74	20138.0	0.89
	75	8	23406.7	19425.4	0.83	22952.5	0.98
Tension	25	5	4704.8	4247.0	0.90	5555.9	1.18
	50	7	12909.6	9555.7	0.74	11527.0	0.89
	75	8	17647.3	14864.4	0.84	17155.9	0.97

Table 8

Comparison between analytical stiffness model (Eurocode 5 [15]) and the mean experimentally determined connection stiffness.

Test	Fastener length (mm)	n	$K_{ser}$ (kN/mm)	$K_{el,EC5}$ (kN/mm)	$\frac{K_{el,EC5}}{K_{ser}}$
Shear	25	5	3.18	3.64	1.14
	50	7	3.33	3.73	1.12
	75	8	3.23	3.75	1.16
Tension	25	5	3.44	3.55	1.03
	50	7	3.20	3.62	1.13
	75	8	2.82	3.61	1.28

be any particular trend with screw length. Similarly, there does not appear to be any influence due to fastener length for the bracket connections subjected to shear loading.

The value of  $\gamma_{an}$ , which is an indicator of the accuracy of the model when compared to the experimental result is particularly high for the 50 mm fastener length (1.35 for tension and 1.34 for shear connections using the Eurocode 5 model and 1.12 for tension and shear connections using the Blaß & Uibel model) indicating that both analytical models may be particularly conservative for fasteners of this length. While the characteristic values used to calculate the overstrength factors were adjusted for sample size in accordance with EN 14358 [45], it is noted that the sample size is small and an increased number of connection tests are required further examine the influence of fastener on the overstrength factors.

5. Summary and conclusions

The influence of fastener length on the elastic and ductile behaviour of connections utilising CLT panels manufactured from C16 grade material has been presented. Tension and shear tests have been performed on single and multiple fasteners in C16 grade CLT. Monotonic tests have been carried out in accordance with EN 26891 [39] and cyclic tests in accordance with EN 12512 [29] have been utilised to examine the

**Table 9**  
Overstrength factors ( $\gamma_{Rd}$ ) for single fasteners subjected to tension loads, and shear loads parallel and perpendicular to the grain direction.

Property	Tension Fastener length			Shear - parallel Fastener length			Shear - perpendicular Fastener length		
	25 mm	50 mm	75 mm	25 mm	50 mm	75 mm	25 mm	50 mm	75 mm
$F_{max,05}$ (N)	896.0	4440.4	6102.0	1574.0	2903.9	4380.6	1371.6	2657.8	4052.2
$F_{max,95}$ (N)	2258.2	6423.7	9840.4	1932.7	4979.2	6077.9	2760.5	4972.7	6465.5
$F_{ax,k,EC5}$ (N)	1061.7	2388.9	3716.1	1269.8	2096.4	2428.2	1269.8	2096.4	2428.2
$\gamma_{sc}$	2.52	1.45	1.61	1.23	1.71	1.39	2.01	1.87	1.60
$\gamma_{an}$	0.84	1.86	1.64	1.24	1.39	1.80	1.08	1.27	1.67
$\gamma_{Rd}$	2.13	2.69	2.65	1.52	2.38	2.50	2.17	2.37	2.66
$F_{ax,k,B\&U}$ (N)	1389.0	2881.8	4289.0	1643.0	2517.3	2869.1	1643.0	2517.3	2869.1
$\gamma_{sc}$	2.52	1.45	1.61	1.23	1.71	1.39	2.01	1.87	1.60
$\gamma_{an}$	0.65	1.54	1.42	0.96	1.15	1.53	0.83	1.06	1.41
$\gamma_{Rd}$	1.63	2.23	2.29	1.18	1.98	2.12	1.68	1.98	2.25

**Table 10**  
Overstrength factors ( $\gamma_{Rd}$ ) of bracket connection tests subjected to tension load.

Property	Tension connection Fastener length			Shear connection Fastener length		
	25 mm	50 mm	75 mm	25 mm	50 mm	75 mm
$F_{max,05}$ (N)	4704.8	12909.6	17647.3	11195.2	22523.2	23406.7
$F_{max,95}$ (N)	8956.5	23276.4	27154.8	13232.2	33756.5	31902.5
$F_{ax,k,EC5}$ (N)	4247.0	9555.7	14864.4	10158.6	16771.0	19425.4
$\gamma_{sc}$	1.90	1.80	1.54	1.18	1.50	1.36
$\gamma_{an}$	1.11	1.35	1.19	1.10	1.34	1.20
$\gamma_{Rd}$	2.11	2.44	1.83	1.30	2.01	1.64
$F_{ax,k,U\&B}$ (N)	5555.9	11527.0	17155.9	13144.0	20138.0	22952.5
$\gamma_{sc}$	1.90	1.80	1.54	1.18	1.50	1.36
$\gamma_{an}$	0.85	1.12	1.03	0.85	1.12	1.02
$\gamma_{Rd}$	1.61	2.02	1.58	1.01	1.68	1.39

ductile performance of the connections. The validity of current analytical models to predict the strength and stiffness of tension and shear connections is examined. The following conclusions can be formulated based on the investigation presented.

- On single fasteners, a trend of increasing load with increased fastener length was observed when examining the maximum load, yield load and ultimate load. However, increasing fastener length resulted in a decrease in ductility.
- In multiple fastener connections, the fastener length was found to have a positive influence on the load carrying capacity when loaded both under tension and shear conditions.
- Under tension loading, the mean connection ductility values for all fastener lengths was found to be quite similar and there does not appear to be any influence due to the fastener length. On the other hand, for shear loading, the mean connection ductility values were found to reduce with increased fastener length.
- Two analytical strength models were presented, namely the Eurocode 5 [15] model and the Blaß & Uibel [19] model. Generally, the Eurocode 5 [15] model produced more conservative values. The model by Blaß & Uibel [19] produced higher values but also over-predicted the failure strength of 25 mm screws under all loading situations. In the absence of further tests, it is not recommended to use this model for screw lengths less than 50 mm on C16 grade CLT.
- When examining connection stiffness, the factor of 2.0, recommended in Eurocode 5, was shown to over-predict the experimental stiffness. This reduced stiffness is due to the ductility within the steel bracket, which is neglected in the analytical formula.
- Overstrength factors have been determined for multiple fasteners in bracket-fastened CLT connections subjected to tension and shear loading conditions. Based on the results, conservative overstrength factors of 2.5 and 2.1 are recommended for tension and shear connections utilising multiple screw fastened bracket connections

between CLT panels. These overstrength factors are specific to CLT manufactured from C16 grade material.

- Strength degradation factors have been calculated under different loading conditions and there does not appear to be any influence due to fastener length. The results indicate that the strength degradation behaviour is more heavily influenced by the loading mode. For screw-fastened bracket connections in C16 grade CLT, conservative values of 0.80 and 0.70 are recommended for tensile and shear connections, respectively.
- Further tests are required on floor-floor connections and wall-wall connections under tension and shear loading. Floor-floor and wall-wall connections, such as lap joint or splice connections, are generally achieved using large self-tapping screws and are an essential component in the construction of multi-storey timber buildings. While many studies have investigated their behaviour in CLT manufactured from C24 grade material, their behaviour on CLT manufactured from different grade materials, such as C16, should also be examined to assess the influence of density on the failure behaviour.

## Acknowledgement

This work has been carried out as part of the project entitled ‘Commercialisation of Irish Cross Laminated Timber’ (project ref. 15/C/694) funded by the Department of Agriculture, Food and the Marine of the Republic of Ireland. The contribution of the technical staff of the College of Engineering and Informatics, NUI Galway, in particular, Peter Fahy, Colm Walsh and Gerard Hynes, is acknowledged.

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