Construction and Building Materials 122 (2016) 446–457

Contents lists available at ScienceDirect

Construction and Building Materials

journal homepage: www.elsevier.com/locate/conbuildmat

Experimental investigations and design provisions of steel-to-timber joints with annular-ringed shank nails for Cross-Laminated Timber structures

Matteo Izzi [a,b,*], Georg Flatscher [c], Massimo Fragiacomo [d], Gerhard Schickhofer [c]

[a] Department of Engineering and Architecture, University of Trieste, Piazzale Europa 1, 34127 Trieste, Italy
[b] National Research Council of Italy – Trees and Timber Institute (CNR IVALSA), Via Biasi 75, 38010 San Michele all'Adige, Italy
[c] Institute of Timber Engineering and Wood Technology, Graz University of Technology, Inffeldgasse 24/1, 8010 Graz, Austria
[d] Department of Civil, Construction-Architectural and Environmental Engineering, University of L'Aquila, Via Gronchi 18, 67100 L'Aquila, Italy

HIGHLIGHTS

- An experimental programme is carried out on nailed steel-to-timber joints in CLT.
- Average and characteristic values of the strength capacities are evaluated.
- Results are compared with the models given in current design codes and literature.
- The overstrength factor γ_{Rd} and the strength degradation factor β_{Sd} are calculated.
- Conservative values of γ_{Rd} and β_{Sd} are provided.

ARTICLE INFO

Article history:
Received 8 January 2016
Received in revised form 3 June 2016
Accepted 14 June 2016
Available online 9 July 2016

Keywords:
Annular-ringed shank nail
Steel-to-timber joint
Cross-Laminated Timber
Hysteretic behaviour
Calculation model
Capacity-based design
Overstrength factor
Strength degradation factor

ABSTRACT

This paper investigates the mechanical and the hysteretic behaviour of steel-to-timber joints with annular-ringed shank nails in Cross-Laminated Timber (CLT). These fasteners are used to anchor typical metal connectors, such as hold-downs and angle brackets, to the CLT panels. The experimental programme presented in the paper was carried out at the Institute of Timber Engineering and Wood Technology, Graz University of Technology (Graz, Austria). Average and characteristic values of the experimental strength capacities are evaluated and compared to the analytical predictions determined according to current structural design codes and literature. Furthermore, to fulfil the requirements of the capacity-based design, the overstrength factor and the strength degradation factor are evaluated and conservative values are recommended.

1. Introduction

Ensuring an adequate ductility and a sufficient energy dissipation are two key aspects when designing seismic resistant multi-storey timber buildings made of Cross-Laminated Timber (CLT) panels. As a structural product, CLT is characterized by high in-plane stiffness and a linear-elastic behaviour with tendency to fail with brittle mechanism (except for compressive stresses). Therefore, mechanical connections between adjacent walls and between wall and floor panels represent the ductile zones of CLT structures, supplying most of the building flexibility and providing the necessary strength, stiffness and ductility [1].

The hysteretic behaviour of single-joints and CLT wall systems (CLT wall panel and connections) was the focus of several experimental programmes. Shear and tension tests were performed on typical metal connectors, such as hold-downs and angle brackets, and on screwed panel-to-panel connections [2–6]. Furthermore, racking tests performed on CLT walls with several layouts of connections and openings [7–12] and full-scale shaking table tests [13–15] demonstrated significant energy dissipation.

* Corresponding author at: Department of Engineering and Architecture, University of Trieste, Piazzale Europa 1, 34127 Trieste, Italy.

E-mail addresses: izzimatteo@gmail.com (M. Izzi), georg.flatscher@tugraz.at (G. Flatscher), massimo.fragiacomo@univaq.it (M. Fragiacomo), gerhard.schickhofer@tugraz.at (G. Schickhofer).

http://dx.doi.org/10.1016/j.conbuildmat.2016.06.072
0950-0618/

Predicting the load-carrying capacity of joints with dowel-type fasteners in CLT is more complex than for traditional sawn timber or other engineered wood products (e.g. glued laminated timber). Blaß and Uibel [16] developed a calculation model for the prediction of the fastening capacity in CLT. Specific rules for joints in CLT, derived from the works of Blaß and his collaborators, are prescribed in the Austrian National Annex to Eurocode 5 [17]. However, design formulas were not included in structural design codes of any other European country.

The experimental programme presented in the paper aims at investigating the behaviour of steel-to-timber joints with annular-ringed shank nails in CLT. These nails are used in CLT buildings to anchor typical metal connectors (such as hold-downs and angle brackets) to the wall and floor panels. Monotonic and cyclic single fastener joint shear tests were carried out in parallel and perpendicular to the face lamination of the CLT panels while nail withdrawal tests were performed from the side face of CLT panels. Moreover, the tensile strength and the yield moment of the fastener were measured via tension and bending tests, respectively.

Mechanical properties such as strength, stiffness, ductility and equivalent viscous damping ratio were assessed as prescribed in EN 12512:2001/A1 [18] and ISO 16670 [19]. Characteristic values of the experimental strength capacities were derived according to EN 14358 [20] and were compared to the analytical predictions prescribed in the current standards [21,22,17] and recommended in literature [16]. Finally, the overstrength factor and the strength degradation factor were evaluated and conservative values were recommended for nailed steel-to-timber joints in CLT.

2. Calculation models

The current version of Eurocode 5 [21] prescribes design rules for traditional structural products (solid timber, glued laminated timber, etc.) and fasteners (smooth nails, dowels, bolts, etc.). However, the same standard does not include any design provision for CLT and typically used metal connectors (such as angle brackets and hold-downs) requiring the use of harmonized technical specifications like the European Technical Assessments (ETAs). Some specific rules for joints in CLT were included in the Austrian National Annex to Eurocode 5 [17]. Moreover, Blaß and Uibel [16] proposed a calculation model for joints with dowel-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. It should be noticed that this model was validated on CLT panels made of board layers thinner than what are used nowadays and has not been included in structural design codes of any European country to date.

The calculation models considered in this study are described in the following sub-sections. The design rules included in Eurocode 5 [21] divide the steel-to-timber joints into two groups: joints with thin metal plates (i.e. plates with thickness less than 0.5d, with d diameter of the fastener) and joints with thick metal plates (i.e. plates with thickness greater than d). The thickness of the metal plate influences the failure mechanism of the joint. Joints with thick plates have a ductile failure mechanism where the bending capacity of the fastener is attained with two plastic hinges together with embedding of timber. Joints with thin plates have a less ductile failure mechanism where the bending capacity is attained with one plastic hinge together with embedding of timber. It must be noticed that, due to their conical-shaped cap, annular-ringed shank nails do not have such strict distinction. For instance, ETA-13/0523 [23] (Rotho Blaas nails) takes into account a similar distinction between thin and thick plates; however, compared to Eurocode 5 [21], the condition of thick plate is satisfied with a much thinner plate (1.5 mm thickness if d = 4.0 mm and 3.0 mm thickness if d = 6.0 mm). On the contrary, the design provisions included in ETA-04/0013 [22] (Simpson Strong-Tie nails, like those used in this experimental programme) refer only to thick plates and can be applied to any joint regardless the thickness of the metal plate. Therefore, for the sake of clarity, the following discussions are all referred to steel-to-timber joints with thick plates, whereas joints with thin plates were not included in the study.

2.1. Capacity-based design approach

The application of a capacity-based design procedure to CLT structures requires the definition of specific regions that must withstand large cyclic deformations and provide a stable energy dissipation. When it comes to ductile failure of timber structures, this is achieved with proper connection design and by ensuring that no other part (less ductile or brittle) exhibits anticipated failure. However, results of past experimental programmes on metal connectors (i.e. angle brackets and hold-downs) and CLT wall systems have highlighted some inappropriate mechanisms at the connection level that may be associated to an incorrect design of the nailed steel-to-timber joints. In particular: (a) in wall-to-floor connections with angle brackets, failure under tensile loads due to withdrawal of the nails connected to the floor panel; (b) in wall-to-foundation connections with angle brackets, failure due to pull-through of the anchoring bolts; and (c) in wall-to-floor connections with hold-downs, tensile failure of the net cross-section of the metal sheet.

Such failure mechanisms can be avoided by applying a capacity-based design approach, both at the connection level and at the wall level. Using force-based design methods, the load flow is followed from the top to the foundation of the building and design values of the action effects are determined (E_d). At the connection level, those values are used as inputs for the ductile design of the dissipative elements. In particular and again focusing on commonly used angle brackets and hold-downs, capacity-based provisions may be employed to avoid the afore-mentioned failure mechanisms and to ensure the plasticization of laterally loaded steel-to-timber joints. Once inappropriate failures at connection level are prevented, similar provisions are applied at the wall level. Here, the strength of the CLT panel (around the connections and of the entire panel considering, e.g., openings) is designed for the overstrength of the dissipative connections considering their strength degradation for cyclic loading.

As discussed in Follesa et al. [24], a structural element designed in accordance with the concept of dissipative behaviour is verified at the Ultimate Limit State if:

$$E_d \leqslant \beta_{Sd} F_{Rd,ductile} \tag{1}$$

with E_d design value of the action effects, $F_{Rd,ductile}$ design strength of the ductile element and β_{Sd} reduction factor for strength degradation for cyclic loading. The design strength of the ductile element is defined as $F_{Rd,ductile} = k_{mod} F_{Rk,ductile} / \gamma_M$, where $F_{Rk,ductile}$ is its characteristic value while k_{mod} and γ_M represent the modification factor for duration of load and moisture content and the partial factor for material properties, respectively. Values of $F_{Rk,ductile}$ should be determined either by theoretical considerations or from experimental results in monotonic conditions. It should be noticed that Eurocode 8 [25] sets the partial factor for material properties γ_M equal to 1.0 for ductile elements designed in accordance with the concept of dissipative behaviour.

Once the dissipative elements are verified at Ultimate Limit State, ductile failure mechanisms can be ensured by designing the strength of the brittle part ($F_{Rd,brittle}$) so that it is greater than or equal to the strength of the ductile part ($F_{Rd,ductile}$) multiplied by an overstrength factor γ_{Rd} and divided by a reduction factor for strength degradation due to cyclic loading β_{Sd} [24]: