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Numerical optimisation of novel connection for cross-laminated timber buildings

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ABSTRACT

This paper presents Finite Element Analyses (FEA) on the performance of a novel hold-down connection, suitable for mass-timber buildings, consisting of hollow circular steel tubes, inserted in Cross-laminated Timber (CLT) panels. The FEA models, which account for nonlinear material properties of both timber and steel components, were validated against experimental results. The primary objective of this research was to optimise the connector geometry and material properties to achieve a target yield capacity of 85 kN. Considering a capacity-based design approach, where all the wood components remain elastic and only the steel tube deforms, a sensitivity analysis was performed to quantify the input-performance correlations. It was shown that the diameter (d) and thickness (t) of the tube, as well as the coupler diameter (c) were the main parameters influencing the yield capacity and overall performance of the connection. The subsequent optimisation resulted in an optimum detailing with $d = 155$ mm, $t = 9.5$ mm and $c = 30$ mm, and a required steel yield stress of approx. 800 MPa. With a load-carrying capacity of 98 kN, an elastic stiffness of 30 kN/mm, and a ductility of 9, the connection was deemed suitable for the design of resilient CLT buildings. Finally, it was shown that optimised detailing exhibited a robust performance in presence of uncertainties in the timber and steel material properties, as well as geometry of the detailing.

1. Introduction

1.1. Background

Timber, described as a zero-carbon material [1], is seen as a viable option to address the concerns of sustainable construction for the world's future skylines. The combination of engineered wood products, such as Cross-Laminated Timber (CLT), with innovative structural systems and connectors, is among the factors behind the resurgence of mid-rise timber buildings [2]. Although the majority of constructed all-wood buildings are mid-rise, between six and ten storeys, different concepts for taller timber buildings have been proposed [3]; and the construction of buildings like Origine, a twelve-storey CLT building in Quebec City, Canada [4], is to proof of the feasibility. CLT panels, composed of a number of lumber layers glued together crosswise on their wide faces to form a dimensionally stable panel, can be used as floor or wall segments, with its production in North America being regulated through ANSI PRG 320-2018 [5].

CLT systems have gained popularity due to the high level of prefabrication and flexibility in planning, which reduces construction time, hence total cost of projects [6]. Previous research demonstrated the excellent performance of CLT structures and their connections when subjected to seismic loads [7–15]. In addition to meeting the strength and stiffness demands, ductility is a main requirement for resilient connection design [16,17]. The significance of this property is highlighted when the joints need to undergo deformations to dissipate the energy released during seismic events. Providing ductility in connection detailing changes the seismic performance of the building and leads to a reduction of the design forces. Furthermore, post-earthquake considerations require a quick rehabilitation of the building, characterised by minor damage or an easy replacement of failed structural components.

1.2. Capacity-based design of CLT buildings

Recent analytical and experimental studies [7–9] provided design guidance for platform-type mid-rise CLT buildings. Assuming that CLT panels in a shear wall behave as rigid body, the performance depends on the connections to the floor below, and between shear wall segments. To comply with the capacity-based design approach, ductile

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behaviour can be attributed to: (i) the joint resisting shear between the floor and the wall above, (ii) anchor or hold-down connectors resisting uplift and overturning, and (iii) the connection against vertical shear between two wall panels. These components are considered as 'fuse', as they are designed for first failure mechanisms.

Capacity-based design considerations, as recommended by the Canadian engineering design in wood standard CSA-O86 [18], allows that failure occurs in ductile components while brittle elements are overdesigned to remain elastic. This aims to ensure that the building survives high seismic loads with acceptable damages. According to CSA-O86 [18], a connection is considered as moderately ductile if: (i) yielding governs its resistance; (ii) its ductility ratio (μ), estimated from dividing the ultimate deformation by the yield deformation, is at least 3.0; and (iii) it possesses sufficient deformation capacity to allow wall rocking, or a combination of rocking and sliding. The CLT lateral load resistance system connections designed for over-strength are: (i) the joints against shear between the floor panel and wall underneath, (ii) joints between adjacent CLT floor panels, and (iii) joints between perpendicular walls.

1.3. Hold-downs for CLT buildings

The low weight of timber structures makes them more prone to uplift and overturning forces under lateral loads, compared to concrete buildings. These forces are mitigated by installing hold-downs of adequate strength and stiffness at the bottom corner of shear walls, to anchor the building to its foundation. These connections, when designed to act as fuse, according to the capacity-based design approach, also help in supplying the required system ductility.

Traditional commercially available hold-downs, such as straps emerging from the foundation and nailed to the shear walls, can provide the required strength, stiffness, and ductility for low-rise timber buildings. Traditional hold-down systems are usually not suited for taller mass-timber buildings, where both gravity and lateral loads become significantly larger [19]. Fig. 1a shows typical failure of commercially available non-resilient bracket connectors under reversed cyclic loading [17], idealising seismic forces on a building. The connection was designed for the required strength according to CSA-O86 [18]. However, the lack of resilience resulted in localised brittle wood failure. In different scenarios, the deformation of the bracket connector inevitably damaged the wood as shown in Fig. 1b and c. With such designs, the connection cannot be restored, and replacement could become uneconomic. Since the performance of the connections, and as a consequence the whole structure, depends on the connection's failure mechanisms, there is a need for hold-downs to meet the target performance in terms of strength, stiffness, as well as ductility and deformations.

Rod systems, with a capacity of up to 100 kN [20], have become popular for mid-rise timber structures up to six storeys. Loo et al. [21,22] studied symmetric slip-friction connectors as an energy dissipative device that can be used as hold-down for rocking shear walls. This detailing has tension bolts to mobilise friction between the plates; and nonlinear behaviour is triggered by the plates' sliding movements. Experiments demonstrated excellent performance with a stable hysteresis behaviour characterised by little stiffness degradation and no damage in the timber members. Hashemi et al. [23] developed numerical models of the slip-friction connector and confirmed the efficient energy absorption. The symmetric slip-friction detailing was further improved to the resilient slip friction (RSF) connector to provide self-centring [24].

With the idea of obtaining nonlinear behaviour in pre-defined ductile zones, Zhang et al. [19] investigated the Holz-Stahl-Komposit (HSK™) system as a hold-down for CLT shear walls. The system was based on adhesively bonded perforated steel plates, inserted in timber where the capacity was governed by the minimum strength between of steel plate, adhesive bond, and CLT panel itself. With proper design, yielding occurred within the perforated steel plate, and all timber elements remained undamaged. The system achieved a yield force (F_y), elastic stiffness (k_e), and ductility (μ) higher than 250 kN, 250 kN/mm, and 10, respectively.

For mid-rise CLT buildings, however, both the HSK™ and RSF connectors have limitations. For the former, although high strength, stiffness, and ductility have been obtained during testing, the feasibility study of a twelve-storey timber building performed by Zhang et al. [25], which showed that about two plates of 7.5 m long would be needed in order to resist the imposed uplift forces, questions the practicality. Repair after damage would be impossible as the plates are glued into the CLT panels. If the RSF was to be considered, at least ten connectors would be required to carry the similar uplift force.

1.4. Steel tube hold-down connector

Preliminary experimental investigations indicated that a steel tube connector [17,26] could overcome the limitations discussed before as a hold-down for CLT shear walls. A circular hollow steel section (CHSS), herein referred to as steel tube, is the main component of the connection, see Fig. 2. The detailing is simple; the CHSS is placed inside the CLT panel into a hole of the same diameter. The components are easy to

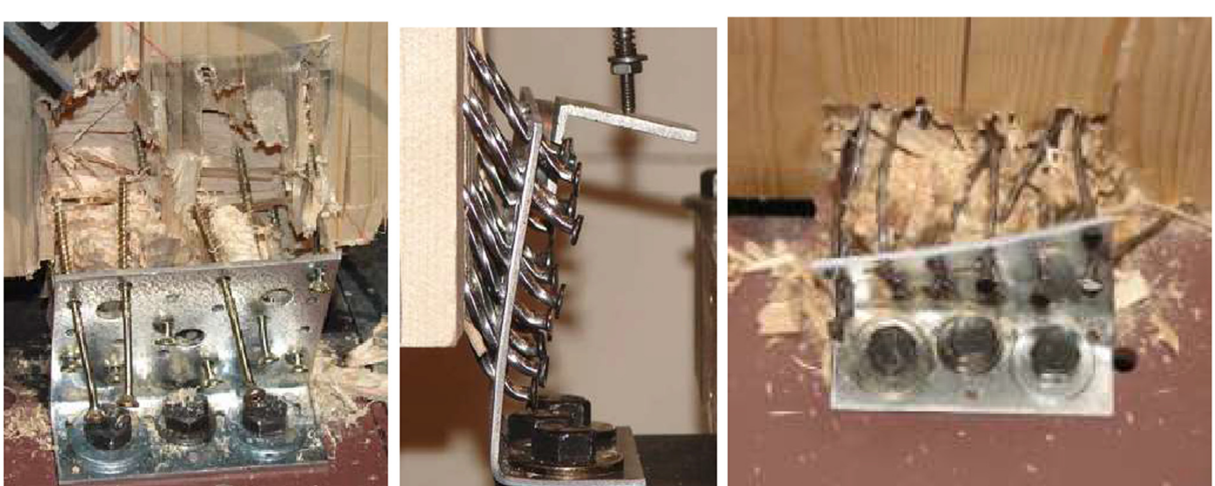

Fig. 1. Damages of traditional non-resilient connectors under reverse cyclic loads: brittle wood failure (a); nails yielding (b), and bracket connector yielding [17] (reprinted with permission).