Self-drilling dowel connections for ductile link beams in coupled CLT walls

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ABSTRACT

Coupled CLT walls with steel link beams can form a more efficient lateral load resisting system for tall timber buildings when compared to single CLT shear walls. To achieve adequate coupling between CLT walls and ensure energy dissipation occurs in the steel link beams, connections between the beams and walls must be: (1) strong enough to ensure ductile failure of the link beams and (2) stiff enough to engage the link beams at low inter-storey drift levels. This paper presents an experimental programme to assess the performance of one type of high-capacity link beam-to-wall connection. The connection used a group of self-drilling dowels and an inserted steel knife plate to connect 200UB18 steel link beams to 5-ply CLT wall elements. Capacity design was used to design the connection. The experimental results validated the capacity design method and showed damage concentrated in the ductile steel link beams. This preliminary study indicated that self-drilling dowel connections may be a feasible solution for the link beam-to-wall connections in coupled CLT shear walls. Further analysis and testing of coupled timber walls are required before this system can be used in new structures.

1 INTRODUCTION

Cross laminated timber (CLT) is a solid mass timber panel consisting of layers of sawn timber boards that are oriented at 90 degrees to adjacent layers, which forms a structural element with good dimensional stability and high in-plane strength and stiffness. Common uses include structural floors and wall elements in mass timber buildings. Construction using CLT allows a building to be prefabricated off-site which can lead to higher building quality, faster construction, and reduced on-site labour. CLT structures have been gaining global popularity as CLT becomes more readily available and recognized by structural engineers, architects, and owners. The Brock Commons (Think Wood 2017) and HoHo Vienna (Lightwood 2018) buildings are examples of recent, milestone CLT projects.

Mass timber walls made of CLT, and similar products like laminated veneer lumber (LVL), can be challenging for structural engineers to design because there is a lack of high-capacity connection types, design methods, research, and code acceptance for mass timber lateral load resisting systems. They are especially difficult to use in high seismic regions where a building's lateral load resisting system must resist greater forces and exhibit ductile performance to provide damping forces. The development of high-capacity connections to support the use of CLT wall systems is a rapidly developing area of research. Without adequate connections, CLT wall structures cannot take full advantage of the panel's in-plane strength and stiffness.

A coupled timber wall structure, with ductile link beams, is an effective lateral load resisting system only when high-capacity connections can be depended on to transfer forces between adjacent walls through the link beams. The two walls in a coupled wall system behave as cantilevers under lateral loads and impose large displacement demands on the link beams between them. Critical connections in this system are required at the ends of the link beams (also called coupling beams) and at the base of the walls to the foundation. The concept of coupled wall structures, with ductile coupling beams to provide hysteretic damping, was researched and popularized for reinforced concrete buildings by Paulay (1969) and Santhakumar (1974). This system is advantageous and commonly used for several reasons: (1) significantly greater stiffness than single walls, (2) ability to accommodate door and window openings, and (3) conventionally designed with relatively large ductility factors (i.e. $\mu=6$ in NZS3101 (2017)) which reduces design forces. For similar reasons, steel eccentric braced frames (EBF) (New Zealand Standard 2007) are also popular and work in a similar way to coupled walls, where ductile steel links are provided between adjacent braced bays and have large displacement demands imposed on the links to dissipate energy.

The coupled wall configuration can be also be adopted by mass timber walls, as shown in Figure 1. This was conceptualized by Karsh (Michael Green Architecture 2017) in the form of timber core walls (optionally paired with steel frames in a dual system) and was researched by Bhat (2013) and Zhang (2017). The research showed potential for the proposed coupled wall structure in some of the link beam connection tests and nonlinear time history analyses. However, poor performance of the proposed connection detail has, in part, limited its feasibility for use in new buildings.

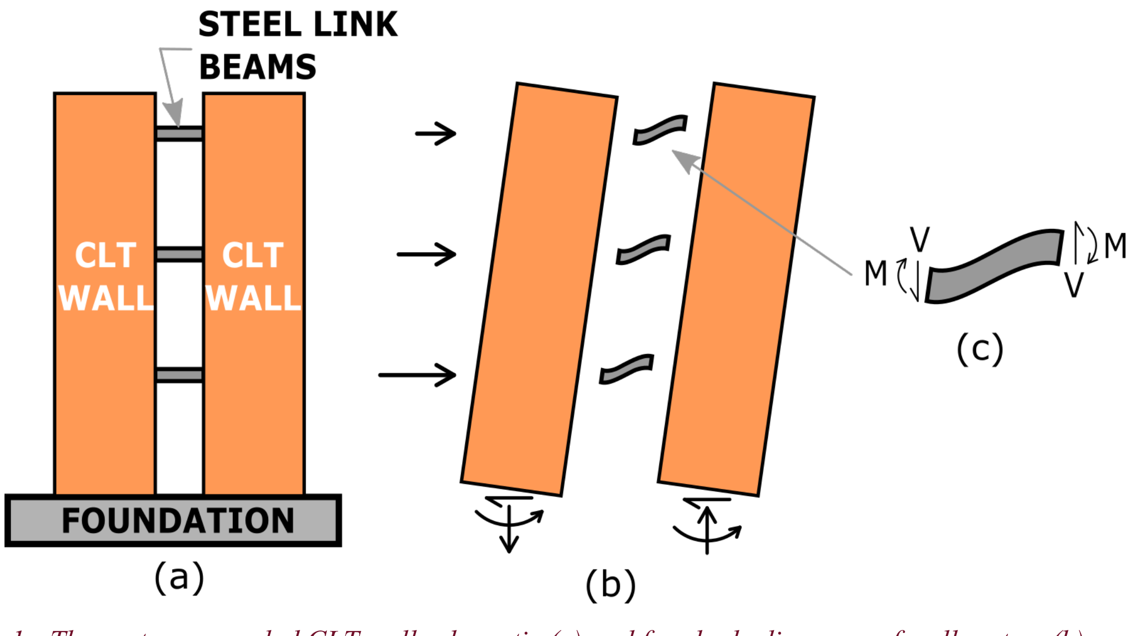

Figure 1 - Three-storey coupled CLT wall schematic (a) and free body diagrams of wall system (b) and link beam (c) when subjected to lateral loading.

2 LINK BEAM TO WALL CONNECTIONS

The behaviour of the proposed coupled timber wall system relies on the performance of the connection between the steel link beams and CLT walls. These connections are critical because they transfer high shear and moment between the walls, creating the coupling action, and are responsible for engaging the ductile link beams in the system. The designer of a coupled timber wall structure must pay attention to three basic properties of the connections: (1) strength, (2) stiffness, and (3) initial displacement (i.e. the amount of deformation required before an element provides a resisting force).

The most intuitive requirement is for the connection strength to exceed the overstrength demand of the steel link beam, following well-established capacity design methods (T. Paulay and Priestley 1992), to ensure ductile failure occurs in the link beam. Overstrength factors for active EBF links are available in NZS3404 and should be used to determine the demand on the link beam connection.

In addition to strength, the stiffness and initial displacement of the connections are also critical to ensure the steel link beams are effectively providing coupling action between adjacent walls. When the connections are not sufficiently stiff, significant displacement demands will not be imposed on the link beams until large inter-storey drifts are experienced by the building. When initial displacement is present, whether translation or rotation, the system will behave as two independent walls before fully engaging the link beams. The effectiveness of a coupled wall system can be compromised if the connections are adequately stiff and tightly fit.

2.1 Connections with self-drilling dowels

Self-drilling dowels (SDD) (Rothoblaas 2021), shown in Figure 2c, can be used to create high-capacity, semi-rigid connections in timber structures. The Ø7 mm metal fasteners have a drill tip that can create holes through steel plate up to 10 mm thick and form tight dowel connections between wood elements and steel knife plates. A threaded portion below the head provides additional force to pull the dowel into the timber element. The capacity of a SDD group connection can be varied significantly by changing the amount and size of dowels, group geometry, and amount of knife plates. Experiments by Dong et. al. (2021) have demonstrated the ability to create strong, stiff, and tight connections with self-drilling dowel groups between glulam beams and columns.

The primary benefit of SDD, compared to conventional dowels, is the absence of any initial displacement, leading to full engagement of the dowel group and high initial stiffness. In contrast, conventional dowels require oversized holes (typically by 1-2 mm) in the steel knife plates, which introduce some initial displacement before the full dowel group is engaged. The SDDs are also advantageous over smaller fasteners, like nails and timber rivets, because they have a greater shear strength (due to larger diameter and greater tensile strength), are long enough to penetrate through the full thickness of a CLT panel, and the strength is independent of the loading direction (unlike timber rivets).

The purpose of this study was to assess the performance of a SDD group connection between steel link beams and CLT walls. The connection, shown in Figure 2a, between a 200UB18 steel link beam (grade 300) and a 5-ply CLT wall panel (45/35/45/35/45 layup) was made using an 8mm knife plate and a group of Ø7x170 mm SDD. The CLT was composed of grade SG8 (45mm boards) and SG6 (35 mm boards) Radiata Pine timber (New Zealand Standard 2005). The steel link beam was stiffened with 6mm web stiffeners on both sides, spaced at 150 mm, to meet the requirements of an active EBF link per NZS 3404.

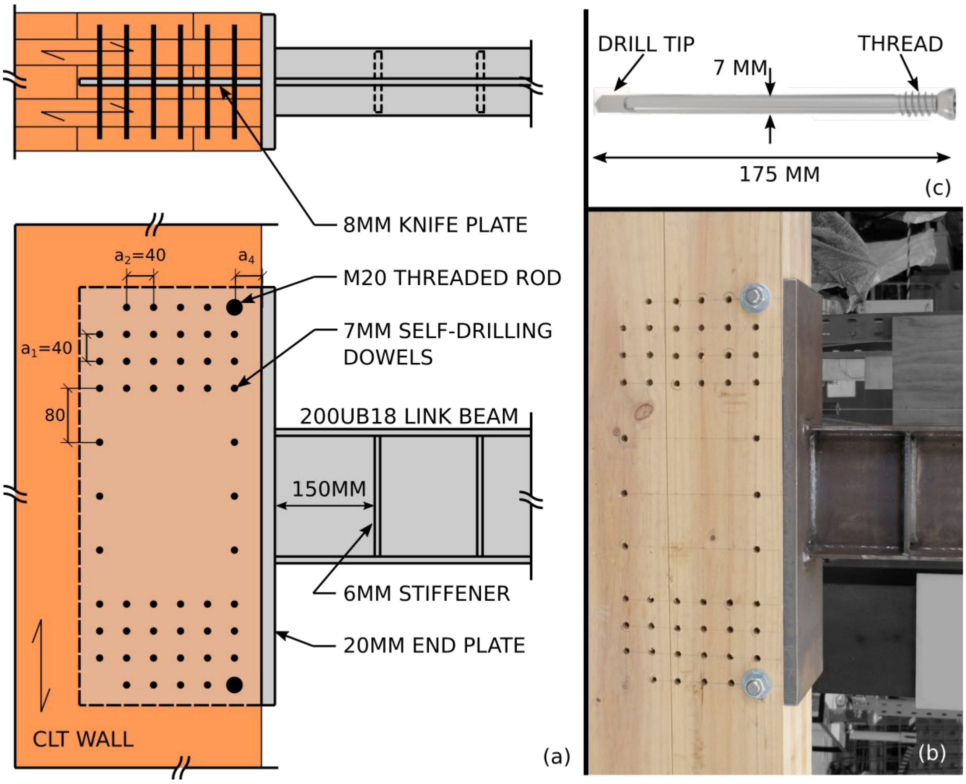

Figure 2 – Connection between steel link beam and CLT wall (a, b) with self-drilling dowels (c).

A typical minimum spacing (a_1, a_2 in Figure 2) and edge distance of 40 mm (a_4) was used between the dowels. The edge distance was less than the minimum value (80mm) required by the product specification (Rothoblaas 2021) for sawn timber but was ignored in recognition of the cross layers in CLT reducing the tendency for splitting failures near the edge. In addition, M20 threaded rods were installed, with nuts and washers, at the corners of the dowel group nearest to the CLT edge following recommendations from *Timber Engineering – Principles for Design* (Blaß and Sandhaas 2017) to prevent splitting in the thickness of the CLT panel.

2.2 Capacity design of self-drilling dowel group

The nominal strength of a SDD group can be calculated using a simplified approach based on previous research by Dong et. al. (2021) which presented a comprehensive moment-rotation prediction method. For design simplicity, this method has been condensed to only find the strength of the connection and modified to eliminate the influence of contact friction.

The nominal strength of a single dowel, F_d, is first found based on the European Yield model equations (Blaß and Sandhaas 2017). A centre of rotation (O'), shown in Figure 3, is assumed and forces in each dowel are found with the following equations:

$$f_{x\_i,j} = \frac{-y'_{i,j}}{r'_{max}} * F_d \tag{1}$$

$$f_{y\_i,j} = \frac{x'_{i,j}}{r'_{max}} * F_d \tag{2}$$

$$f_{i,j} = \sqrt{f^2_{x\_i,j} + f^2_{y\_i,j}} \tag{3}$$

where r' is the distance from the centre of rotation (O') to a given dowel location ($x_{i,j}$', $y_{i,j}$').

Equilibrium of the connection requires the following equations to be satisfied:

$$\sum f_x = F_c = \frac{l_c b_{eff} f_{c,0}}{2} \tag{4}$$

$$\sum f_y = V_{os} \tag{5}$$

$$\sum f_{i,j} * r_{i,j} + F_c * \frac{2l_c}{3} = M^* + V_{os} * d' = M^{*\prime} \tag{6}$$

where F_c is the contact force,

l_c is the contact length,

b_{eff} is the thickness of CLT layers oriented parallel to the contact force,

$f_{c,0}$ is the compressive strength of the CLT layers oriented parallel to the contact force,

d' is the distance from the centre of rotation to the geometric centroid, and

M*' is the moment about the centre of rotation (O').

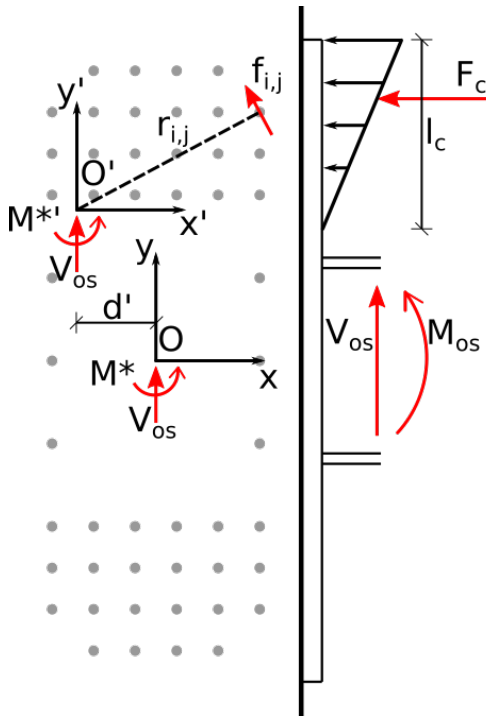

Figure 3 – Analytical model and notation for determining the strength of a dowel group connection.

If the equilibrium conditions are not satisfied, then a new centre of rotation is assumed, and the process is repeated.

The proposed method is inherently conservative because it determines the force at which the first dowel (with a maximum value of $r_{i,j}$) yields. However, the experiments by Dong et. al. (2021) demonstrated that a group of SDD can reach a higher ultimate strength and exhibit ductile behaviour beyond its yield strength when timber splitting is prevented. Therefore, instead of using the yield strength for capacity design, the ultimate strength of the connection may instead be used if minor damage in the connection will not compromise the performance of the wall system. This allows a more economical design to be achieved. To calculate the ultimate strength, the proposed method is modified to assume a plastic force distribution where all the dowels have reached their yield strength. Therefore, equation (3) is replaced with the following:

$$f_{i,j} = F_d \quad (7)$$

The connection in this study was designed to ensure its nominal strength was greater than the bending overstrength and associated shear force of the 200UB18 link beam (M_{os}=91 kNm, V_{os}=240 kN). An overstrength factor of 1.7 was used for the link beam according to NZS 3404. The moment demand at the centroid of the dowel group (M*) was 130 kNm, which accounts for the 160 mm offset from the end of the steel link beam.

The two M20 threaded rods were not considered in the strength calculation. Their installation requires oversized holes in the steel and timber elements and therefore they cannot be relied upon to resist load under small rotations.

A yield strength of V_{ny}=193 kN (M*=104 kNm at the dowel group centroid) and an ultimate strength of V_{nu}=261 kN (M*=141 kNm) were determined with the proposed method. The ultimate strength satisfied the capacity design requirements. All calculations were based on a single dowel nominal strength of 12.5 kN using the European Yield Model.

3 EXPERIMENT SETUP

As shown in Figure 4, a connection specimen between a portion of CLT wall and one side of a steel link beam was loaded by an actuator through a pinned connection on the beam. For test setup convenience, the specimen was rotated by 90 degrees from its orientation in a real building. Three replicates were tested, one under monotonic loading and two cyclic.

The CLT wall panel was tied down to the concrete strong floor to resist overturning and shear keys placed on either side to resist sliding. Guide rails with Teflon sliding surfaces were placed above the loading point on either side of the beam flanges to prevent out-of-plane deflection and twisting about the beam's length. A hydraulic actuator (400 kN capacity) loaded the specimen through a welded tab 400 mm above the CLT wall edge, which represents the inflection point and mid-span of a link beam in a coupled wall system.

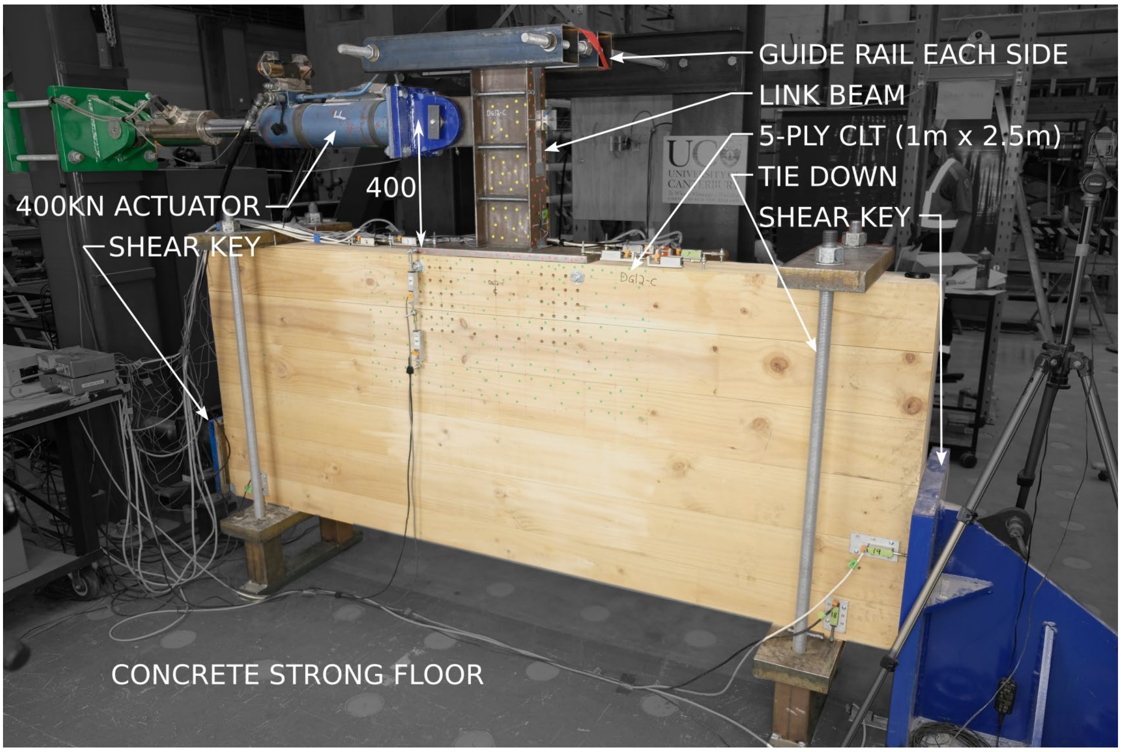

Figure 4 – Experimental setup for cyclic testing of steel link beam and SDD connection to CLT wall.

The cyclic tests used the cyclic protocol from AISC 341 Section K2.4c (American Institute of Steel Construction 2016), which is used to evaluate the cyclic performance of eccentric braced frame (EBF) link-to-column connections. This is a displacement-controlled protocol that imposes a series of increasing chord rotations and decreasing amounts of cycles as the rotation level increases, as shown in Figure 5.

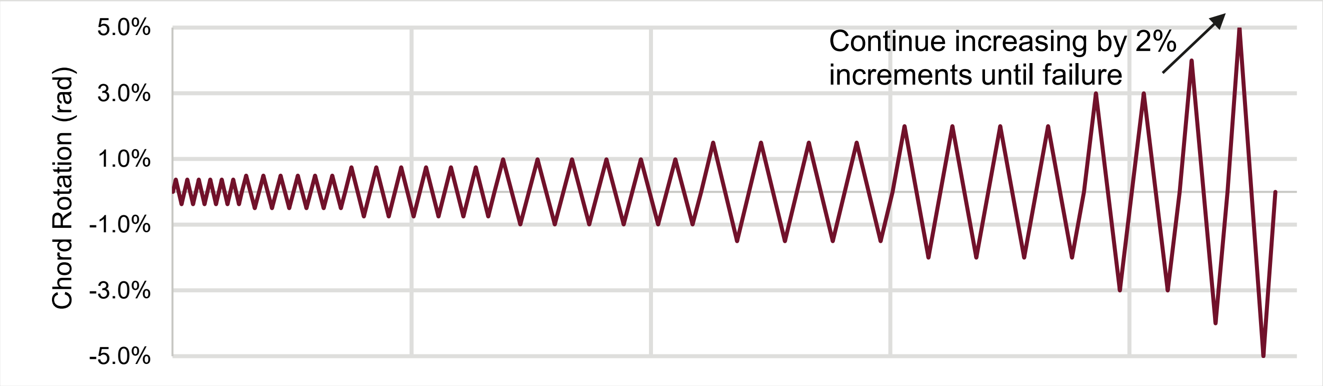

Figure 5 - Cyclic loading protocol from AISC 341 Section K2.4c used for cyclic loading.

4 RESULTS AND DISCUSSION

Preliminary results from the experiments are shown in Table 1, including peak forces in the steel link beam, chord rotations, and connection stiffness values. Total displacement of the beam and connection assembly is represented by chord rotation (Θ) (a useful design quantity used in coupled concrete wall and steel EBF link design) which is found by dividing the relative displacement between the loading point and the CLT wall by the distance to the end plate, shown graphically in Figure 6. The 20 mm end plate was considered as a rigid offset from the wall edge and therefore the effective chord length was 380 mm.

Table 1 - Results from link beam and connection testing.

	Link Beam Force		**Link Chord Rotation**				**Connection Stiffness**	
Test	V_{peak} (kN)	M_{peak}[a] (kNm)	Θ_y (rad)	$\Theta_{y,\,con}$ (rad)	Θ_{peak} (rad)	$\Theta_{peak,\,con}$ (rad)	K_v[b] *Initial / Secant* (kN/mm)	K_m[b] *Initial / Secant* (kNm/rad)
1-M	219	83	0.027	0.0092	0.12	0.013	129 / 104	17,700 / 16,400
2-C	250	95	0.025	0.0066	0.080	0.011	158 / 122	29,900 / 20,200
3-C	247	94	0.026	0.0072	0.092	0.012	145 / 115	22,500 / 17,300

[a]Calculated at the end of 200UB18 beam.
[b]For cyclic tests, the average value of positive and negative cycles is presented.

The chord rotation at yield (Θ_y) was found at the intersection of two linear functions that were fit to the pre-yield and post-yield regions of the force-displacement data. The contribution of deformation from the connection is also reported ($\Theta_{y,\,con}$).

The connection stiffness is represented by translational (K_v) and rotational (K_m) springs at the centroid of the dowel group, as shown in Figure 6. Initial and secant stiffness values are presented because the connection exhibited nonlinear behaviour when loads beyond ~60% of the peak load were applied to the specimen. The initial stiffness was found by fitting a linear function the load-displacement data of the connection between 5% and 60% of the peak load. The secant stiffness was determined by dividing the peak force (or connection moment) by the corresponding connection displacement (or rotation).

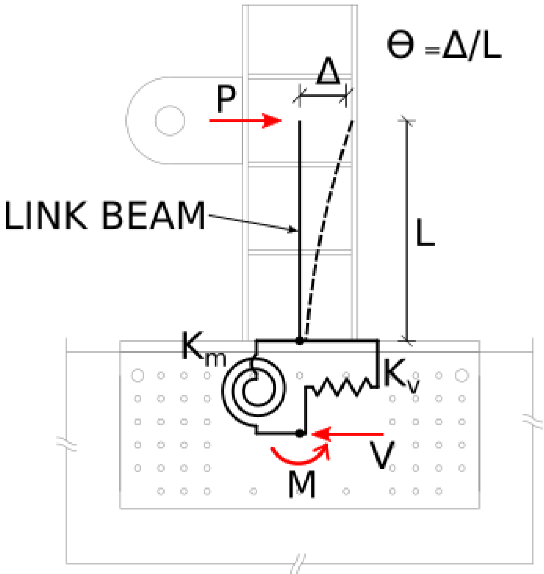

Figure 6 – Analytical spring model of beam to wall connection and calculation of chord rotation.

The chord rotation includes deformations from the CLT wall, steel link beam, and dowel group connection. The contribution of the dowel group connection to the total chord rotation is shown for the yield ($\Theta_{y,con}$) and peak strength ($\Theta_{peak, con}$) points in Table 1. The connection's flexibility contributes a mean value of 29% at yield and 13% at peak strength to the total chord rotation experienced by the beam and connection assembly. Therefore, it is critical to consider the connection's flexibility when analysing a coupled timber wall system.

The connection stiffness is quite variable between the 3 test iterations. This is partly attributed to the inherent variability of timber properties but is also influenced by the initial gap between the steel end plate and the CLT edge. The rotational behaviour of the connection, captured by a single rotational spring, is governed by the dowel group and contact between the steel end plate and CLT edge. The connection is subjected to quite small rotations and can therefore be influenced by small changes in the initial gap between the end plate and CLT edge. This initial gap is governed by the construction method and fabrication tolerances for the CLT and steel elements.

Moment versus chord rotation plots are shown in Figure 7 for the 3 replicates. The energy dissipation of the link beam was significant, as seen by the large hysteresis loops in Figure 7b. A small amount of initial displacement developed during the cyclic tests and can be seen at low displacement levels in the hysteretic plot. Although the SDDs created a tight connection when they were installed, minor embedment damage of CLT around the dowels occurred (Figure 8b) and caused the small amount of initial displacement. However, the chord rotation due to this initial displacement is considered negligible (maximum of 0.0025 radians observed in Test 3-C).

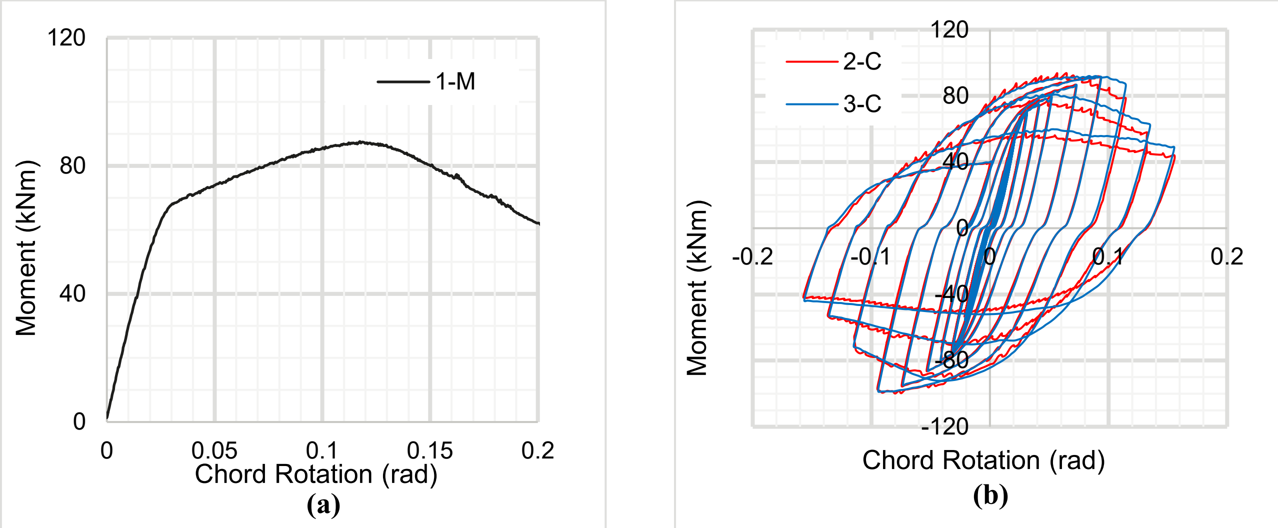

Figure 7 – Moment vs. chord rotation response for the (a) monotonic and (b) cyclic tests.

Damage to the specimen, shown in Figure 8, was concentrated in the link beam's panel zone closest to the end plate where the web and flanges yielded and buckled inelastically. Fracturing in the beam's web (Figure 8c) also occurred but not until large displacements were experienced after which the specimen's strength loss exceeded 20% of its peak strength. No permanent bending of the dowels was evident after deconstructing the specimen. Additionally, a small region of embedment damage was observed in the CLT edge due to bearing of the steel end plate.

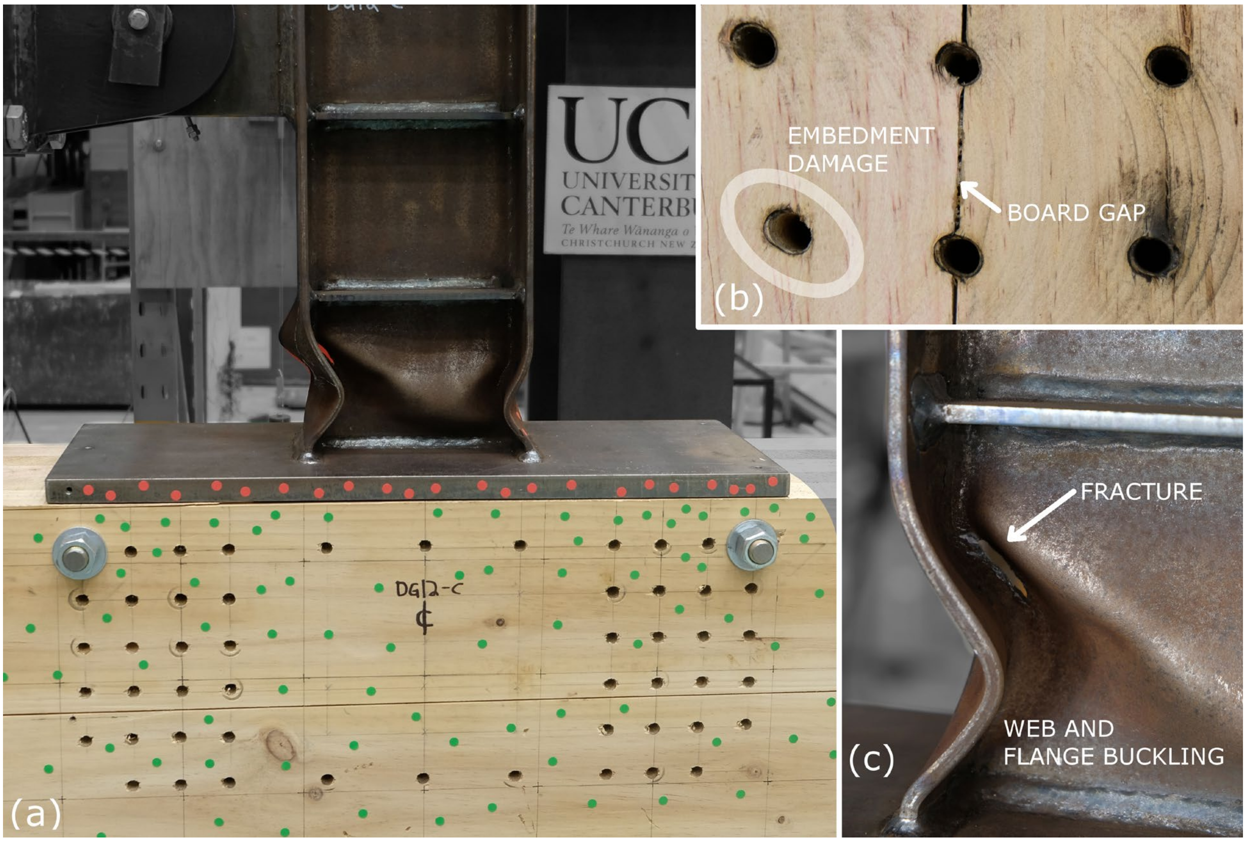

Figure 8 - Damage to steel beam flange and web (a, c) and absence of damage to self-drilling dowels (b).

No damage near the CLT edge was observed where the edge distance (a_4) was less than the minimum value recommended by the product's specification. A reduced minimum edge distance may therefore be justified for similar connections. However, more testing should be completed before this reduced edge distance may be generally acceptable in CLT applications.

The tests confirmed that the connection strength exceeded the steel link beam's overstrength. The maximum shear observed in the test was slightly higher than the expected overstrength demand (250 kN vs. 240 kN) but the difference was small (+4%). Therefore, the overstrength factor used for capacity design was appropriate.

4.1 Limitations and Future Work

The scope of this experimental programme was limited to a single connection configuration which was used to validate an analytical approach for strength prediction. Although the sample size was small, base material properties of the timber, dowels, and steel beam could be used to further validate the reliability of the SDD connection.

Future work could include predicting the connection stiffness, analysing different variations of the connection components (dowel size, beam size, geometry, CLT layup, multiple knife plates, etc.), evaluating the influence of an axial force in the beam, and investigating dowel block tear-out failure modes. The connection stiffness and link performance data will be used to model the performance of coupled timber wall systems under seismic loading.

5 CONCLUSIONS

Three experiments were completed to assess the structural performance of a SDD group connection between steel link beams and CLT wall panels for coupled timber wall structures. The proposed system uses ductile steel link beams to provide hysteretic damping during an earthquake while protecting the CLT and connections from damage through capacity design principles. A simplified analytical method was proposed and used to design a SDD group connection for the overstrength of 200UB18 steel link beams and specimens were constructed based on the designed detail. Preliminary results led to the following conclusions:

1. All 3 test replicates validated the connection's overstrength design approach.
2. Steel link beams showed ductile performance and damage was concentrated in the region of link beam closest to the wall edge. Minor embedment damage was observed in the CLT around some of the dowels.
3. Self-drilling dowel connections can be capacity designed to concentrate damage in the steel link beams to promote ductile performance of a coupled CLT wall system.

Future analytical and experimental work, including a 3-storey coupled wall test, will provide further research of coupled timber wall systems to provide a technical basis for application in new buildings.

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