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Experimental investigation of the moment-rotation behaviour of beam-column connections produced using compressed wood connectors

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ABSTRACT

The use of timber in construction in medium–high rise construction has increased in recent years largely due to the significant innovation in engineered wood products and connection technology coupled with a desire to utilise more environmentally sustainable construction materials. While engineered wood products offer a low-carbon solution to the construction industry, the widespread use of adhesive and metallic fasteners often limits the recyclability of the structural components at the end of life of the structure and it may be beneficial to reduce this where possible.

To establish the possibility of an all-wood connection solution, this preliminary study examines a series of beam-column connections designs to evaluate the relative performance of the different designs, which are connected with modified or compressed wood (CW) connectors. The connection designs are formed between glued-laminated beam and column members in the first instance and later examined when connecting dowel-laminated timber (DLT) members.

The results show that significant moment capacity and rotational stiffness can be achieved for connections solely connected using CW fasteners. Furthermore, the all-wood solution utilising CW fasteners to connect DLT members has also demonstrated significant moment capacity and rotational stiffness capacity without the use of adhesive and metallic components.

1. Introduction

Recent advancements in timber engineering have led to the development of high-performance engineered wood products (EWPs) which allow for the construction of taller, safer and more economical timber buildings. In the last two decades, the EWPs commonly referred to as massive or mass timber solutions are garnering attraction across the globe. Mass timber products are EWPs with larger section sizes, typically with the smallest dimension greater than 75 mm [1]. They are layered products comprising sawn timber boards or laminates that are adhesively bonded/mechanically connected to produce thick panels or linear elements [1–3]. These products are being increasingly used in building construction as they offer excellent load carrying capacity, fire performance, durability and are easily customisable making them very suitable for construction purposes [1,3]. On the sustainability side, they offer lower embodied carbon solutions and provide excellent potential for recycling at the end-of-life of the buildings [1,3–7]. Due to their structural performance, environmental properties and cost-competitiveness, several studies [1,2,6,8] have shown their significant potential as an environmentally alternative to steel and concrete in construction.

Although mass timber products are bio-based and sustainable, there is scope to further improve their environmental and health benefits by minimising the use of energy-intensive synthetic adhesives and metallic connectors during the manufacturing and assembly processes of these products. Currently, the use of synthetic adhesives and mechanical fasteners cannot be eliminated in timber construction until new bio-based solutions emerge with comparable structural performance and cost. In the last two decades, there is a great interest in the development of modified wood and bio-based composites [9–17]. Research [9,11,15] has demonstrated that thermo-mechanically densified wood, which is a form of modified wood manufactured by subjecting timber to heat and pressure and often termed densified or compressed wood (CW), results in increased density, strength, stiffness, and hardness. Namari et al. [15] provides an exhaustive database on the structural strength and stiffness properties of CW in a number of different loading conditions. A series of

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studies has examined the use of CW in structural timber applications and indicated that CW could be an ideal choice for manufacturing wood-based connectors for structural applications [16–22]. Mehra et al. [17] examined beam-beam spliced connections using CW connectors and when compared to comparable steel connectors, demonstrated that the CW solution could achieve 80% of the steel solution. However, there are limited studies available on the applications of CW connectors in structural connections systems. Jung et al. [20] used CW of Japanese cedar as a substitute for high-density hardwood for making shear dowels. The CW with its annual ring perpendicular to the loading direction (0°), when tested in double shear, showed good properties as a dowel material in terms of its enhanced strength and ductility. Kitamori et al. [23] developed a friction joint system using CW wedges and metallic bolt-nut connections for timber buildings. The proposed joint system was shown to have adequate mechanical performance in terms of high stiffness, strength and ductile behaviour. Jung et al. [24] examined the use of CW dowels to improve the strength of glued-in-rod (GIR) joint systems. Their test results demonstrated that GIR joints with CW dowels could achieve approximately 1.6 times higher strength than those with maple dowels when subjected to pull-out tests. To make use of the high strength properties of CW connectors, Jung et al. [25–27] also utilised CW plates and dowels within column-sill and beam-column connections as an alternative to steel fasteners. The test results showed that good structural performance of the connection can be achieved by using CW dowels to resist high shear forces and using the CW plates to resist bending moment. However, these findings are based on the limited number of test configurations and smaller sample size, hence, further tests are required.

Further, CW is also subject to what is known as a shape memory or shape recovery effect whereby it attempts to partially recover its original shape with time. This effect is often characterised as an unstable feature of CW [28] however, the shape recovery of CW can be utilised as a beneficial trait for the development of connections and DLT products [7,10,16,29]. The stress relaxation of CW due to shape recovery ensures a tight fit and prevents loosening of joints during the service life of CW laminated timber products [30,31].

This preliminary study focuses on the use of CW as a connector material within beam-column connections. The study aims to experimentally characterise the moment-rotation behaviour of beam-column timber connections produced using CW dowels and CW plates to connect both glued-laminated and CW dowel laminated timber (CWDLT) structural members. The experimental programme was carried out in two phases. Each phase comprised the development of connection designs and associated experimental testing in accordance with EN 26891 [32]. In Phase-I, seven different beam-column connection designs were produced using glued-laminated structural members. In Phase-II, a novel all-wood beam-column connection system was developed and tested using CWDLT members and CW connectors and the moment capacity, rotational stiffness and ductility ratios from the structural tests were used to evaluate the relative performance of the different connection designs.

2. Experimental programme

2.1. Introduction

In Phase-I of testing, seven different semi-rigid type beam-column moment connection designs were developed using glued structural members and CW connectors (plates and dowels). The developed connection designs were classified into two main categories, namely "modern connection designs" and "traditional mortise and tenon designs". The classification was done based on the presence or absence of CW plates in the design. A typical example of each type is presented in Fig. 1. The modern connection design utilises the CW plates mimicking slotted-in steel plate connections, which are commonly used for timber structures. The traditional mortise and tenon type beam-column connection designs are solely connected with CW dowels which fasten the mortise and tenon connection. Each connection comprises a column and beam section connected at 90°. The top and bottom ends of the connected members are cut at 45° to allow for the connection to be subjected to a compression load between the steel platens of a testing machine.

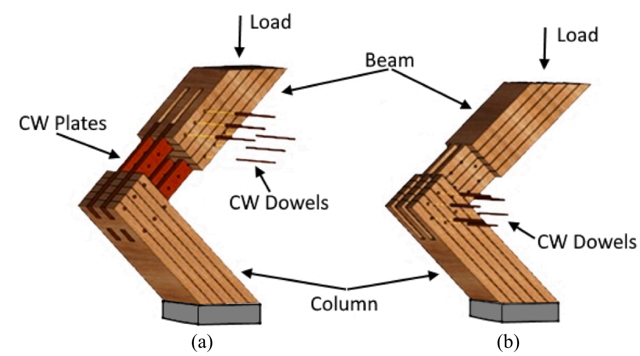


Fig. 1. Connection types and components, (a) Modern design connection type (Design -I-1) with CW plates and dowels and (b) Traditional mortise and tenon design connection type (Design-I-5) connected with CW dowels.

In Phase-II, connection designs were manufactured using CWDLT members connected with CW plates and dowels. The Phase-II designs were limited to modern type connections as the traditional mortise and tenon type connections were not compatible with CWDLT lamination due to the relatively thin tenon width which is just 28.5 mm wide.

2.2. Materials

2.2.1. Compressed wood (CW) connectors

CW in the form of CW plates and CW dowels were used as connectors for the beam-column connections. They were manufactured using a thermo-mechanical densification process, which involves the application of heat and pressure to the timber as described by Sotayo et. al. [11]. The CW dowels were produced using visually graded kiln dried uncompressed Scots Pine (*Pinus sylvestris*) boards. The mean density of the timber boards was 556 kg/m^3 with a standard deviation (Std. Dev.) of 77 kg/m^3. Clear specimens of 200 mm (L) × 9 mm (T) × 22 mm (R) were thermo-mechanically compressed in the radial direction to 10 mm (R). The final dowel diameter varied from 10 to 10.3 mm. The moisture content of the CW dowels was in the range of 5–8%, and their mean density was 1289 kg/m^3 (Std. Dev. = 92 kg/m^3) at this moisture content.

The CW plates were manufactured from clear and uncompressed timber boards of Western Hemlock (*Tsuga heterophylla*) that had an original mean mass density of 425 kg/m^3 (Std. Dev. = 99 kg/m^3). Western Hemlock was chosen to manufacture the CW plates as sourcing of defect-free Scots pine of the required dimensions was not possible. The initial dimensions of the uncompressed plates were 510 mm in the longitudianl (L) direction, 63 mm in the tangential (T) direction and 67 mm in the radial (R) direction. They were compressed in the radial direction to a final thickness of 21 mm (R). The mean oven-dry density of the CW plates was 1291 kg/m^3 (Std. Dev. = 96 kg/m^3).

2.2.2. Beams/Columns

The structural members used in this study were manufactured using kiln-dried timber boards of Irish-grown Douglas fir (*Pseudotsuga menziesii*). The boards were conditioned at a relative humidity of 65 ± 5% and a temperature of 20 ± 2C prior to manufacturing the structural members. The mean oven-dry density measured on small clear specimens (n = 40) was 572.1 kg/m^3 (Std. Dev. = 57.0 kg/m^3). There were two types of structural members, (1) glued-laminated and (2) CWDLT. For the glued-laminated beams, the timber laminates were glued together using a one-component PU adhesive and were clamped in a rig to a minimum pressure of 0.6 MPa in accordance with EN 14080 [33]. Each connection specimen was manufactured from beam and column elements each with a cross-sectional area of 180 mm (b) × 180 mm (h). The CWDLT members were manufactured using CW dowels to connect the timber laminates. To allow for comparison with the test results of connections with glued members, all geometric parameters (180 mm × 180 mm) remained constant. The holes were drilled in a staggered dowel arrangement with 50 mm spacing which allows for a relatively low reduction in the net cross-sectional area compared to the parallel dowel row arrangement [11].

2.3. Methodology

The connections tests were performed using a Denison hydraulic testing machine, model T42B [34], rated to 500 kN, under displacement control mode. All tests were carried out in accordance with EN 26891 [32] at a constant displacement rate of 3.2 mm/min and the ram displacement and load were recorded continuously throughout each test. Each specimen was placed between the loading head and machine platform to compress the connection as shown in Fig. 2. The base of the machine platform was fixed, and the loading head comprised a ball joint ensuring concentric loading throughout the test. Pairs of linear variable displacement transformers (LVDTs) with an accuracy of 0.1 %, were placed on each side of the column to measure the relative beam-column displacement of the connection at two points spaced 120 mm apart as shown in Fig. 2a.

The LVDT displacements are used to determine the relative rotation of the beam and column elements using Eq. (1). The rotation angle on each side of the connection was calculated using Eq. (1) [35–38].

$$\theta = \tan^{-1}\frac{\Delta_1 - \Delta_2}{s} \quad (1)$$

where Δ_1 and Δ_2 are the displacement readings from the corresponding LVDTs, and s is the distance between the LVDTs.

The moment on the connection is the product of the load and the lever arm distance or perpendicular distance between the load and the bearing point. The lever arm distance was approximately 244 mm and was the same for all connection designs (see Fig. 2b). The moment capacity (M) of the connection was calculated using the following expression:

$$M = F_{max} x l \quad (2)$$

where F_{max} is the maximum load, and l is the lever arm distance.

The rotational stiffness (k) of the connection was calculated based on 10% and 40% of the maximum moment and the corresponding rotational angles using Eq. (3).

$$k = \frac{M_{,40} - M_{,10}}{\theta_{avg,40} - \theta_{avg,10}} \quad (3)$$

The ductility ratio (D) of each connection was calculated as per EN 12512 [39] using Eq. (4):

$$D = \frac{V_u}{V_y} \quad (4)$$

where V_u is the ultimate displacement taken at a load level of $0.8F_{\max}$, and V_y is the yield displacement was calculated based on the 1/6 method [39].

3. Connection designs using glulam members and CW connectors- Phase-I

3.1. Connection configuration and assembly

The seven connection design types examined in Phase-I are presented graphically in Fig. 3 and a more detailed summary of each design type is presented in Table 1. For each modern connection type, four CW plates (two in the compression zone and two in the tension zone) were housed in slots routed in the connected members as shown in Fig. 3. In Design-I-1 and Design-I-2, the top surface of the CW plates in the tension zone was exposed in the column, whereas in Design-I-3, the CW plates are contained within the shouldered column which extended beyond the connection area. In all cases, the routed slot was 20.5 mm wide and 60 mm deep to accommodate the CW plates. The number of dowels used to connect the beam and column varied. Design-I-1 was fastened using 6 CW dowels in the column and beam areas, whereas Design-I-2 was fastened using 4 CW dowels in the column and 6 CW dowels in the beam. Design-I-3 has a similar dowel arrangement to Design-I-2; however; the two CW plates in the tension zone of the column remain unexposed due to the shouldered column.

The traditional mortise and tenon connections were manufactured with a varying pattern and number of dowels. Furthermore, the tenon geometry was varied from a full depth tenon (180 mm) to a reduced depth tenon (140 mm), which is accommodated in the mortises as shown in Fig. 3. In all cases, the tenons were 28.5 mm in width to fit

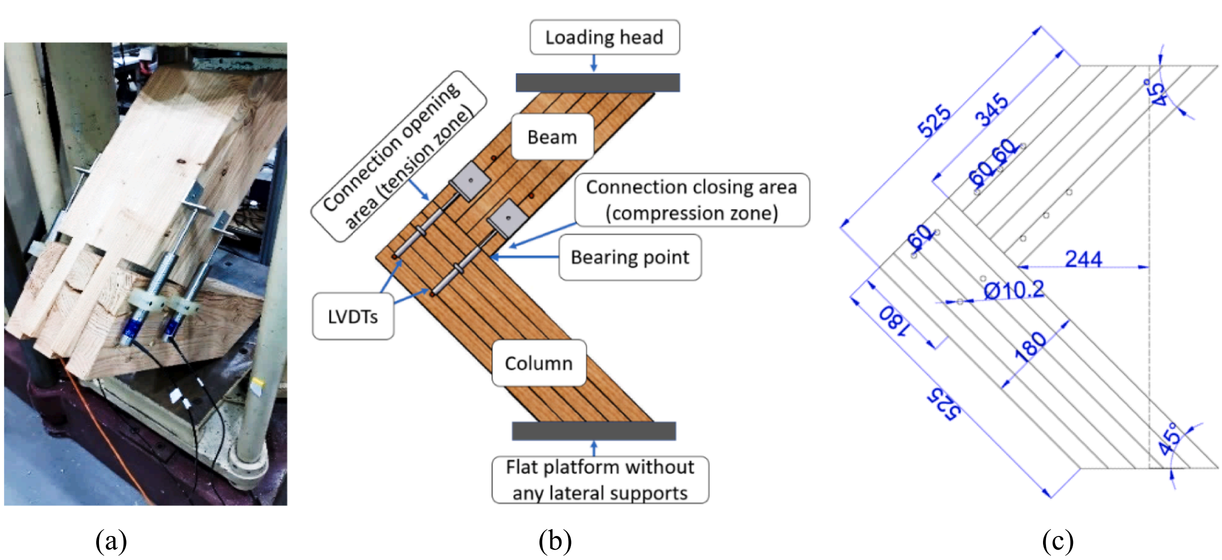


Fig. 2. Connection testing, (a) test set up, (b) connection test set up components, loading direction and position of the LVDTs, and (c) geometry of Design-I-II showing lever arm distance. (Dims. in mm).

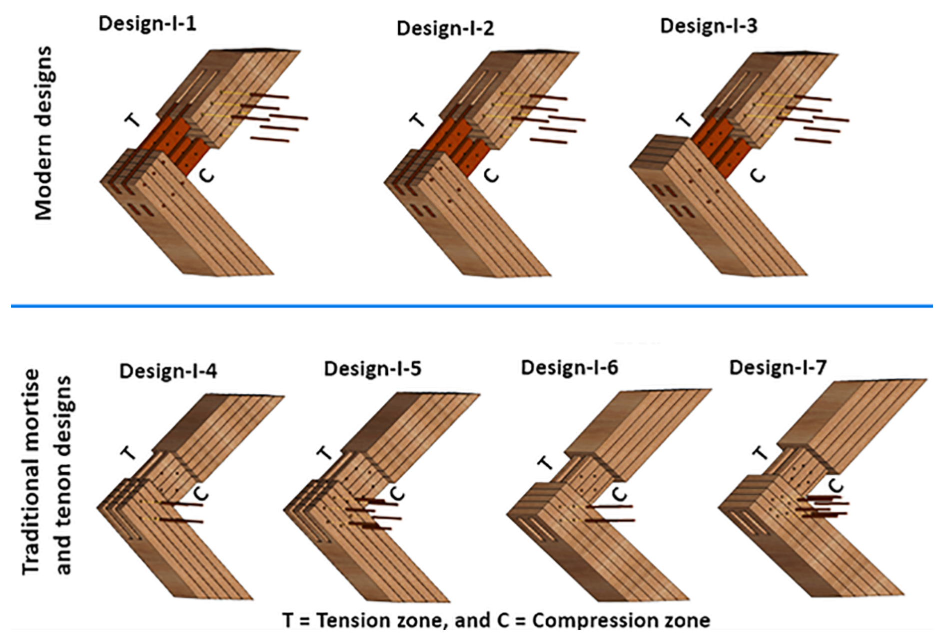


Fig. 3. 3D models of beam-column connection designs.

Table 1
Connection characteristics detailing connection type, components, no. of CW dowels and additional comments for specific connection designs.

Connection type	Design type	Connection Components	No. of CW Dowels		Comments
			Column	Beam	
Modern designs	Design-I-1	CW Plates & Dowels	6 Dowels	6 Dowels	–
	Design-I-2	CW Plates & Dowels	4 Dowels	6 Dowels	–
	Design-I-3	CW Plates & Dowels	4 Dowels	6 Dowels	Shouldered Column
Traditional mortise and tenon designs	Design-I-4	CW Dowels	4 Dowels	–	Full depth tenon
	Design-I-5	CW Dowels	6 Dowels	–	Full depth tenon
	Design-I-6	CW Dowels	4 Dowels	–	Reduced depth tenon
	Design-I-7	CW Dowels	6 Dowels	–	Reduced depth tenon

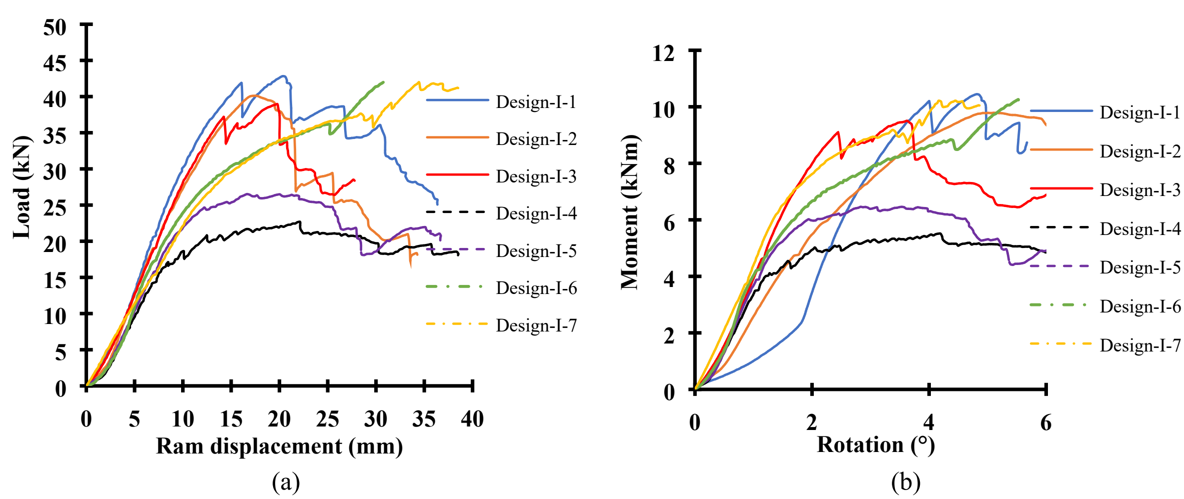


Fig. 4. Structural performance of beam-column connections using glued structural members and CW connectors (a) load-ram displacement behaviour, and (b) moment-rotation behaviour.

within the 30 mm wide mortises. It was expected that the reduced depth tenons may improve the rotational stiffness of the corresponding design as the tenons are not exposed along the top surface but fully housed within the mortise resulting in a more constrained tenon. Design-I-4 and Design-I-6 consisted of 4 CW dowels arranged in a cross-shape pattern, whereas Design-I-5 and Design-I-7 comprised 6 dowels arranged in a hexagonal pattern. The assembled connections were conditioned at 20 ± 2°C temperature and 65 ± 5% relative humidity for 30 days before testing.

3.2. Test results – Phase-I

The load-ram displacement behaviour for all the connection designs can be seen in Fig. 4a. In this study, the term "ram displacement" refers to the crosshead movement of the loading plate of the testing machine. All modern connection designs showed a relatively higher yield load and load-carrying capacity when compared to the traditional mortise and tenon designs. The load-carrying capacity of the modern connection designs ranged between 39.0 and 42.9 kN with the corresponding moment capacity ranging from 9.5 – 10.5 kNm. For the traditional designs, it was found that the load-carrying capacity ranged from 22.7 to 42.0 kN with the corresponding moment capacity ranging between 5.5 and 10.3 kNm. While the variation in results was low for the modern connection designs, the traditional designs resulted in a larger variation. It can be observed in Fig. 4a that Design-I-4 and Design-I-5, which were manufactured with full depth tenons, were shown to have the lowest moment capacities of the seven designs. While there was only one specimen of each connection tested, it is clear that the tenon depth affects the behaviour and the reduced depth tenon results in a higher load-carrying capacity and associated moment capacity when examining the traditional connection designs.

Fig. 4b shows the moment-rotation behaviour of all the connection designs and Table 2 summarises the test results for all connection designs. As seen in Fig. 4b, the typical moment-rotation behaviour of each design is similar, with the exception of Design-I-1, where there was a significant amount of initial slip within the design during the initial phase of loading. This is believed to be caused by a loose fit between the dowels and timber column during the manufacture of this specimen. This occurred during the initial phase of loading up to 10% of the maximum moment capacity and as a result, the rotational stiffness of Design-I-1 was calculated between 20% and 40% of the maximum moment capacity of this connection. When examining the modern connection designs, Design-I-2 was shown to have the lowest rotational stiffness when compared to other designs however the differences were not thought to be significantly different from the remaining designs with all connections ranging from 198 to 287 kNm/rad.

When examining the traditional connection types, it was found that there was no significant difference between the rotational stiffness of the different designs and although the tenon depth appears to be significant when examining the moment capacity, the tenon depth did not appear to have an impact on the rotational stiffness. When examining the ductility ratio, the variation in the modern connection designs was low but there were no significant trends when examining the traditional connection designs and further testing is required to perform a statistical analysis. While it is acknowledged that there is only one specimen of each connection design, Design-I-1 demonstrated the highest load-carrying capacity, Design-I-6 achieved the highest rotational stiffness, and Design-I-4 resulted in the highest ductility ratio of the examined connection designs.

Results have shown that the mean yield load of the modern connection designs was approximately 33% higher than that of the traditional connection types, which is probably due to the relatively higher number of CW dowels and significantly higher embedment strength of the CW plates. The mean moment capacity of the modern connection designs was 22% higher than the carpentry connection designs. When comparing the mean rotational stiffness values, the traditional connection types showed a 9% increase when compared to the modern connection designs. The mean ductility ratio of traditional connection types was 45% higher than the modern connection designs.

When examining the failure modes of the different connection types tested in Phase-I, the commonly observed failure types were tension perpendicular the grain failure in the structural members, splitting of CW plates along the row of dowels and bending and shear failure of the CW dowels. The modern connection designs (Design-I-1, I-2, and I-3) demonstrated high moment capacity due to the high embedment strength of the CW plates whereas the ductility was impacted due to the deformation of the CW dowels. The commonly observed failure types were tension perpendicular the grain failure in structural members, bending and shear failure of the CW dowels and splitting of CW plates along the row of dowels as shown in Fig. 5. The typical sequence of the failure modes observed during the testing initiated with splitting on the CW plates, followed by a combination of bending and shear failure of the CW dowels. This was then followed by tension perpendicular to the grain in the connected timber members.

Focusing specifically on the traditional mortise and tenon connection designs, Design-I-4 and Design-I-5 showed higher ductility due to the deformation of CW dowels and crushing of timber around dowel holes. The tenon relish (single or block shear failure behind the peg hole) and mortise wall failure were observed in addition to shearing and bending of CW dowels as shown in Fig. 6.

Design-I-6 and Design-I-7 showed high ductility and moment capacity largely due to the constrained tenon movement and deformation of dowels. These connections demonstrated shear and bending failure of the dowels and embedment failure due to compression perpendicular to the grain in the beam at the bearing point as shown in Fig. 7.

Table 2
Phase-I results summary of beam-column connections using glued structural members and CW dowels.

Connection type	Design type	Load-carrying capacity (kN)	Moment capacity (kNm)	Rotational stiffness (kNm/rad)	Ductility ratio
Modern designs	Design-I-1	42.9	10.5	264*	2.2
	Design-I-2	40.1	9.8	198	1.9
	Design-I-3	39.0	9.5	256	1.7
Traditional mortise and tenon designs	Design-I-4	22.7	5.5	233	3.8
	Design-I-5	26.5	6.5	269	2.9
	Design-I-6	42.0	10.3	287	2.7
	Design-I-7	42.0	10.2	255	1.8

* based on 20% and 40% of the maximum moment and the corresponding rotational angles.

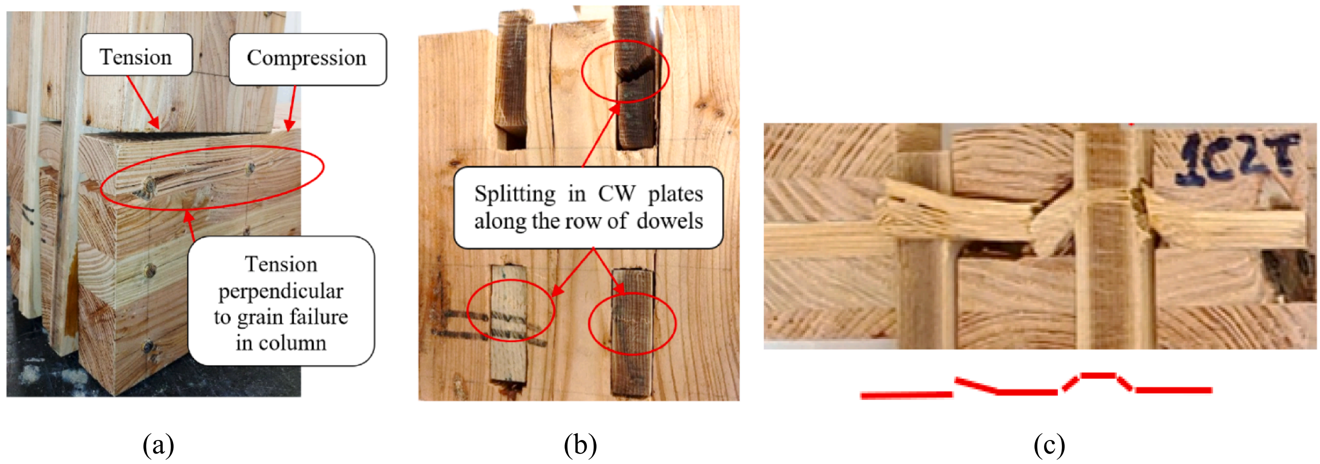


Fig. 5. Typical failure modes in modern connection designs (a) tension perpendicular to the grain in the column, (b) splitting in CW plates and (c) bending and shear failure of the CW dowels.

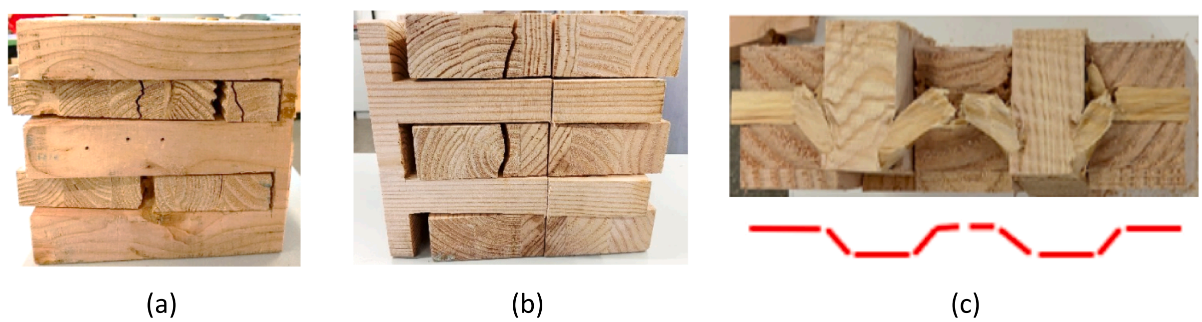


Fig. 6. Typical failure modes in the traditional connection designs (a) tenon relish failure, (b) mortise wall failure, and (c) bending and shearing of CW dowels.

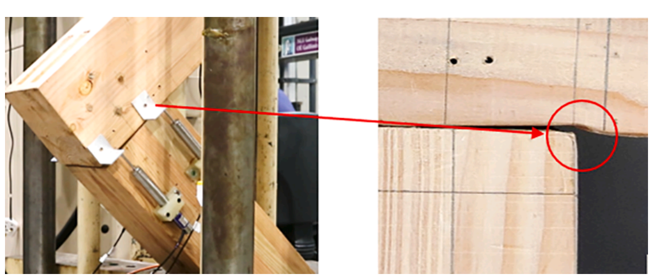


Fig. 7. Embedment failure compression perpendicular to the grain in Design-I-6 and Design-I-7.

4. Connection designs using CWDLT members and CW connectors -Phase-II

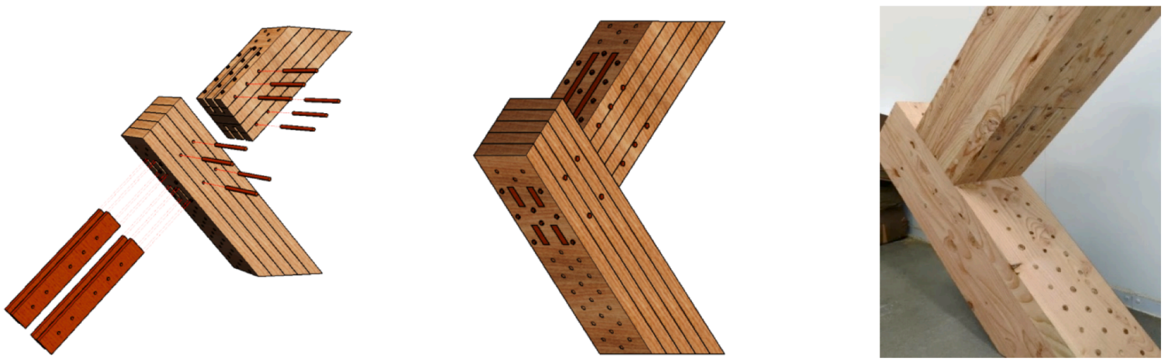
4.1. Connection configuration

In Phase-II, beam-column timber moment connections were developed using CWDLT structural members and CW connectors. When comparing and contrasting the structural response of the connections in Phase-I, it was found that there was no significant difference in the moment capacity, rotational stiffness and ductility ratio of the different modern connection designs. Based on its structural performance and ease of manufacturing, modern connection Design-I-3 was chosen as the basis for the designing of beam-column connections using CWDLT members and CW connectors for Phase-II testing. While it can be said that the traditional Design-I-6 and Design-I-7 designs could be an appropriate choice for a connection between CWDLT members to form an all-wood connection, the selection of these designs raises issues associated with the formation of the laminations in the CWDLT members. These designs cannot be adopted with their existing geometries due to the relatively thin tenon width which is just 28.5 mm wide and therefore too narrow to accommodate the 10 mm diameter laminating dowels.

As a result, the structural members in Phase-II were connected in a similar fashion to Design-I-3 which has a shouldered column using four CW plates and ten CW dowels. In this phase, two connection configurations were investigated, which were categorised based on (1) thickness of the CW plates and (2) inter dowel and edge spacing of the CW dowels. For these two designs, all other connection properties such as the number of CW dowels/CW plates within the connection remained constant as previously manufactured in Design-I-3. Fig. 8 presents the all-wood configuration of the beam-column connection between CWDLT members.

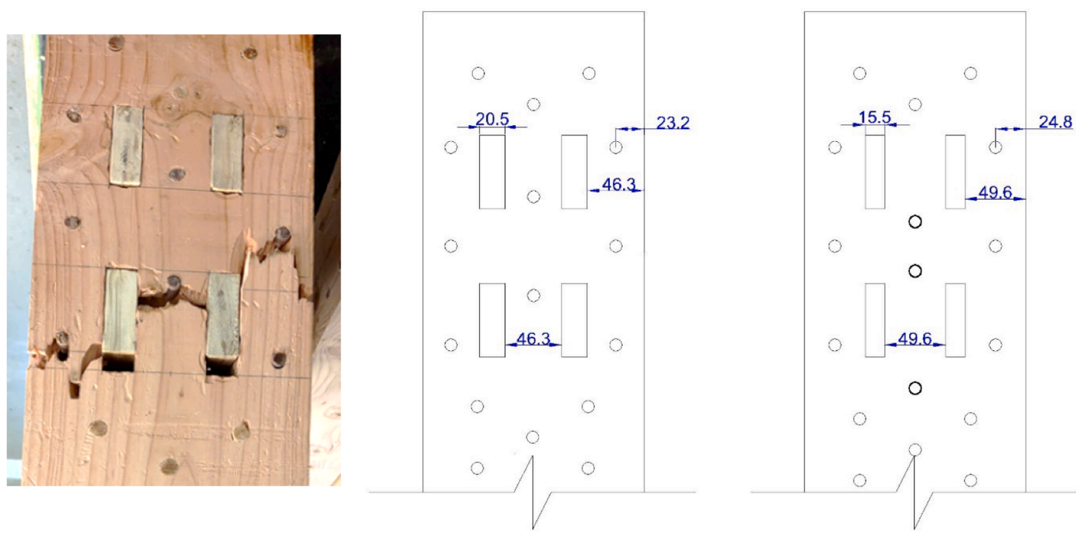
A total of four beam-column connections were manufactured and tested in Phase-II. Only a single specimen of Design-II-1 which comprised 20 mm thick CW plates was developed and tested. This is because of a brittle failure mechanism of the column which occurred due to the placement of CW dowels in the connection zone of the CWDLT members which is discussed in the following sections. To avoid this type of premature failure in the connected members, the thickness of the CW plates was reduced to 15 mm and the position of the laminating dowels was changed in the subsequent design. This new refined design is referred to as Design-II-2. The Design-II-1 geometry is presented in Fig. 9a and Fig. 9b and the Design-II-2 geometry is presented in Fig. 9c which highlights the key geometric dimensions that were adapted to avoid premature brittle failure.

To assemble the connections, both columns and beams were routed



(a) (b) (c)

Fig. 8. Beam-column connection for CWDLT members (a) exploded view, (b) assembled connection utilising CWDLT members and (c) manufactured connection prior to testing**.**



(a) (b) (c)

Fig. 9. Failure mode and plan view of columns showing the difference between the thickness of the slots and spacing of the CW dowels for different designs, (a) Failure mode of CWDLT member (Design-II-1) (b) Design-II-1 (initial design), (c) Design-II-2 with refined configuration and relocated dowels highlighted in bold (Dims. in mm).

at one end to accommodate the CW plates of 20/15 mm thickness. The routed slots were 20.5/15.5 mm wide and 60 mm deep. In both designs, it was ensured that the laminating dowels used to create the CWDLT members would not intersect with the CW dowels which were used to connect the beam to the column. The assembled connections were conditioned at 20 ± 2°C temperature and 65 ± 5 % relative humidity for 30 days prior to testing.

4.2. Test results- Phase-II

The load-ram displacement behaviour of the Phase-II beam-column connections is presented in Fig. 10a. This figure also shows the behaviour of a reference connection using glued-laminated members (Design-I-3). All connections with CWDLT members showed greater ram displacement at failure compared to connections using glued-laminated members. The mean load-carrying capacity of the refined Design-II-2 was approximately similar to the reference connection Design-I-3 and approximately 10% higher than Design-II-1. The ductility of the Design-II-1 specimen was found to be lower than the refined Design-II-2 specimens. This was primarily due to the premature failure of the Design-II-1 column without any significant deformation in the connection area.

Fig. 10b shows a comparison between the moment-rotation behaviour of the beam-column connections produced using CWDLT members and the reference connection configurations using glued-laminated members. It is clear from Fig. 10b that there was a limited rotation of Design-II-1 (approx. 1.5°) at the peak moment. This limited rotation was due to the premature failure mechanism in the column member. Replications of the refined Design-II-2 demonstrated a mean rotation of 3.2° which is approximately two times higher than the Design-II-1 specimen. The failure mechanism of both connection designs is described in the following section.

Table 3 presents a comparison between the beam-column connections using CWDLT members and the reference design using glued structural members (Design-I-3).

It should be noted that there was only one replication of Design-I-3 and Design-II-1 and this has been considered when interpreting the results as it may not give a statistical comparison for the whole population.

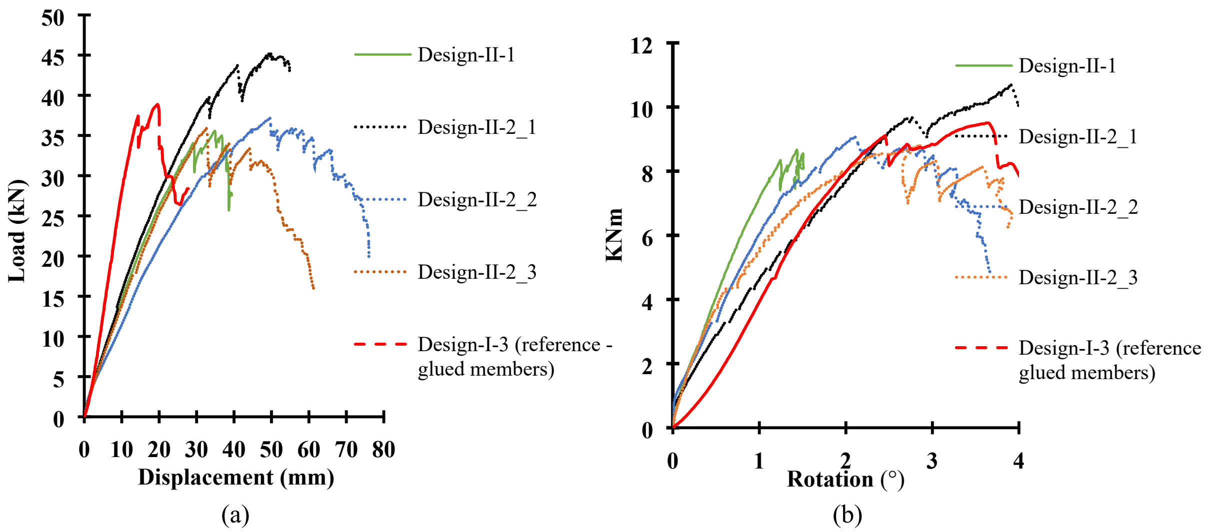


Fig. 10. Comparison between beam-column connections using CWDLT members and glued members, (a) load–displacement graph and (b) moment-rotation graph.

Nevertheless, the mean moment capacity of the refined Design-II-2 specimens was approximately similar to the reference connection Design-I-3 and approximately 10% higher than the Design-II-1 specimen. There was a significant difference in the stiffness when examining Design-II-1, which utilised thicker 20 mm CW plates and had a rotational stiffness approximately 26% higher than the mean rotational stiffness of the refined Design-II-2 specimens but it is difficult to draw any significant conclusions as only 1 repetition of Design-II-1 was tested. It was clear from the results that the reduction in the thickness of the CW plates from 20 mm to 15 mm did not significantly change the moment capacity of the connection indicating that the CW plates were not a governing factor in the connection capacity and that the number of dowels and dowel spacing may be more influential in the design, which creates scope for further refinement.

Splitting was observed between the slots of the compression zone of the CWDLT column member, which is largely due to the compact spacing between the slots and laminating dowels. As a result of the connection testing, the following observations are recommended to avoid premature brittle failure mechanism of the structural members. Increasing the spacing between the laminating dowels and the slots of the column will reduce the risk of brittle failure. Increasing the cross-sectional area of the structural member so that the laminating dowels can be spaced at larger distances will further reduce the risk of splitting failure of the member. As observed in Design-II-1, there was limited deformation in the CW plate and dowel components of the connection and the capacity of the structural column member was surpassed. Therefore, it was assumed that reducing the thickness of the CW plates may activate deformation in the connection area and avoid the brittle failure mechanism in the structural members.

5. Summary and conclusions

This preliminary study has demonstrated the potential to utilise CW connectors in the manufacture of an all-wood solution for beam-column timber connections. In Phase-I, seven different designs of beam-column timber moment connections were developed using glued-laminated members connected with densified CW dowels with and without CW plates. Modern type connections which were connected using CW dowels and CW plates housed within routed grooves in the connected members demonstrated significant moment capacity and rotational stiffness. Traditional mortise and tenon connections connected solely with CW dowels also achieved significant structural loads. It was shown that a reduced-depth tenon housed within the connected member performed better than an exposed full-depth tenon in terms of load-carrying capacity and associated moment capacity. The mortise and tenon connections with the exposed full-depth tenon did have a more ductile response but relatively lower moment capacity when compared to the modern connection types with CW plates and dowels.

As part of Phase-II, the connection types examined were applied to CWDLT members to manufacture an all-wood connection. While the traditional mortise and tenon connection designs are well suited to solid timber and glued timber members, connections between CWDLT members proved problematic for the laminating process and are not well suited given the tenon dimensions examined in this study. As a result, in Phase-II, two modern connection designs were developed, which were designated as Design-II-1 and the refined Design-II-2. Design-II-1 was similar to Design-I-3 with 20 mm thick CW plates. Only one specimen of Design-II-1 was tested due to brittle failure of the column caused by the dowel arrangement used in the lamination process in the production of the CWDLT elements. Therefore, a refined design (Design-II-2) was

Table 3
Phase-II result summary showing a comparison between beam-column connections using CWDLT members and reference design using glued structural members (Design-I-3).

Design type	Type of structural members	CW plate thickness (mm)	Load carrying capacity (kN)	Moment capacity (kNm)	Rotational stiffness (kNm/rad)	Ductility ratio
Design-I-3 (reference)	Glued	20	38.9	9.5	256	1.7
Design-II-1	CWDLT	20	35.7	8.7	413	1.6
Design-II-2_1		15	45.3	11.0	223	2.3
Design-II-2_2	CWDLT	15	37.1	9.1	298	2.1
Design-II-2_3		15	36.1	8.8	396	2.0
Mean (Design-II-2)			39.5	9.6	306	2.1
Std. Dev. (Design-II-2)			5.0	1.2	86	0.2

produced with an improved dowel arrangement and using 15 mm thick CW plates allowing for larger dowel spacing to avoid brittle failure of the structural members. This design produced favourable results with similar load-carrying capacity, moment capacity and rotational stiffness as the connection design with glued-laminated members.

Although there is a limited sample size, this preliminary study demonstrates that CW dowels and CW plates can be successfully used to connect both glued-laminated members and CWDLT members. The results have shown that a 100% wood solution, free from metallic connections or the use of adhesive, can achieve significant moment capacity, rotational stiffness and ductility values. The refined version of the CWDLT connection (Design-II-2) outperformed the comparable connection between glued laminated elements demonstrating that there may be further scope for optimisation of the geometry. Based on the above conclusion, it can be said that the CW connectors could be a suitable replacement to metallic connectors and synthetic adhesives both in the manufacturing of layered structural members and timber-timber connections.

Author Contributions:

S.M contributed to the conceptualization, experimental investigation and reviewing and editing of the final article. C.O.C. contributed to the conceptualization, methodology, experimental investigation, supervision and data analysis of the project in addition to writing, reviewing and editing of the final article. A.S. was involved in the experimental activities and reviewing and editing of the final article. Z.G was involved in the acquisition of funding for this project, involved in the experimental activities and reviewing and editing of the final article, A.M.H. was involved in the acquisition of funding for this project, supervision of all experimental activities and writing, reviewing and editing of the final article.

CRediT authorship contribution statement

Sameer Mehra: Conceptualization, Investigation, Writing – review & editing. **Conan O'Ceallaigh:** Methodology, Investigation, Formal analysis, Writing – original draft, Writing – review & editing, Visualization, Supervision. **Adeayo Sotayo:** Methodology, Investigation, Writing – review & editing. **Zhongwei Guan:** Methodology, Investigation, Writing – review & editing, Funding acquisition. **Annette M. Harte:** Conceptualization, Writing – review & editing, Supervision, Funding acquisition.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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