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EVALUATION OF THE STRUCTURAL BEHAVIOUR OF BEAM-BEAM CONNECTION SYSTEMS USING COMPRESSED WOOD DOWELS AND PLATES

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ABSTRACT: To support the transition to a bio-based society, it is preferable to substitute metallic fasteners and adhesives in timber construction with an eco-friendly alternative. Recent studies have identified compressed wood dowels and plates as a possible substitute for metallic fasteners in contemporary and mainstream applications. In this study, a spliced beam-beam connection system using compressed wood dowels and slotted-in compressed wood plates was examined under four-point bending. The study has considered specimens with compressed wood dowels of 10 mm diameter and compressed wood plates of 10 mm thickness. The load carrying capacity of connections using compressed wood dowels and plates were compared to connections utilising steel dowels and plates of equivalent capacity. Typical failure modes, moment resistance and rotational stiffness of both connection systems are evaluated on the basis of the experimental results. Tests have demonstrated similar failure modes when comparing steel-timber and compressed wood-timber connection systems. The mean failure load for the compressed wood-timber connection system is only 20.3% less than that achieved for the steel-timber connection system. The mean rotational stiffness of the compressed wood-timber connection system is 18.55% less than that achieved for the steel-timber connection system. These preliminary results demonstrate the potential for the use of compressed wood elements in the manufacture of timber connections.

KEYWORDS: Connections, compressed wood dowels, compressed wood plates, four-point bending

1 INTRODUCTION

The widespread use of metallic fasteners and adhesives in modern timber construction has negative implications for the end-of-life disposal or re-use of the structural timber components. Emission of volatile organic compounds during manufacture of synthetic adhesives may have human health impacts in addition to the environmental impact. To cope with the upcoming transition to a bio-based society, it is preferable to substitute metallic fasteners and adhesives with an eco-friendly alternative such as wood-based connectors.

The use of wood-based connectors is not new. In some of the early Egyptian and Polynesian boats, wooden pegs and treenails were used to fasten together the various pieces of the hull [1]. Treenails or trunnels have also been used as connectors in timber frame and covered bridge constructions [2]. Dense hardwood has traditionally been used for connectors in timber structures. However, the use of hardwood fasteners is limited by resource availability and the fact that hardwood connectors undergo stress relaxation, which causes loosening of the joint over time necessitating regular tightening [3-4].

The mechanical and physical properties of underutilised softwood species can be easily modified by chemical and thermal treatments. In recent years, densification of wood by compression, thermal and chemical treatments has been the subject of several research programmes. Examples include viscoelastic thermal compression wood, thermo-hydro-mechanical densified wood, oil-heated treatment and acetylated wood.

Compression of wood results in increased density, decreased porosity and improved material strength, stiffness, hardness and dimensional stability [5]. Compressed Sitka spruce (*Picea sitchensis*) has shown increased Young's modulus with increasing compression ratio in bending [6]. Compressed wood (CW) of Japanese cedar was used as a substitute for high density hardwood for making shear dowels [7]. When compressed radially, Japanese cedar has been shown to have good properties as a dowel material in terms of its enhanced strength and ductility [7].

CW friction joints were found to have a satisfactory high initial stiffness, load carrying capacity and ductility, where compressed wooden wedges were used together with a conventional bolt-and-bearing-plate joint [8]. Jung et al. [9] demonstrated that large moment resistance and ductility can be achieved in column-beam joints utilising CW plates and dowels. The high embedding performance

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of the CW plates contributed to the rotational stiffness, and the high shearing performance of the CW dowels to the axial stiffness.

Current deign codes do not adequately address the design of timber connections using wood-based connectors. The objective of this study is to investigate feasibility of CW dowels and CW plates as eco-friendly substitutes of metallic fasteners in moment-resisting connections. The study comprises experimental evaluation of CW and steel beam-beam moment connections to the determine typical failure modes, load carrying capacity and moment resistance.

2 EXPERIMENTAL STUDY

2.1 INTRODUCTION

This study investigates the use of CW dowels and plates in beam-beam moment connections and compares them to similarly loaded moment connections utilising steel dowels and plates. The beam-beam moment connection between two glued laminated beams is illustrated in Figure 1. The beams are spliced together using a total of two plates and twenty dowels.

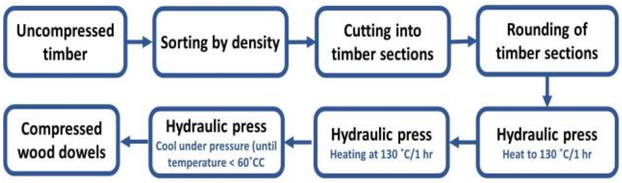
Figure 1: *Beam-beam moment connection*

2.2 MATERIALS

A total of four test specimens were produced comprising two replications of a steel fastened connection system and two of a compressed wood fastened connection system. The dimension lumber used in this study was Irish-grown Douglas Fir (*Pseudotsuga menziesii*). There were eight glulam beams manufactured. In order to minimise the variability among the glulam beams, timber laminates were selected based upon their density. The mean density of the laminates was 477.74 kg/m^3 with a standard deviation of 4.74. Each beam consisted three laminates of 1575 mm long and 52.5 mm thick. The cross-section area of each beam was 115 mm x 157.5 mm. The 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 [10]. All the beams were conditioned at a temperature of 20 ± 2˚C temperature and 65 ± 5 % relative humidity prior to testing.

The CW plates were manufactured using Scots Pine (*Pinus sylvestris*) wood compressed in the radial direction with a compression ratio of approximately 54% at the University of Liverpool, United Kingdom. The CW dowels were similarly compressed in the radial direction. A schematic of manufacturing process and the finished dowels are presented in Figure 2a and Figure 2b, respectively. The final density of the compressed wood dowels ranged from 1100-1500 kg/m^3 with the diameter of 10 $\pm$ 0.5 mm.

Each glulam beam in the test programme was routed at one end to accommodate two compressed wood or steel plates of 10 mm thickness. The routed slot was 11 mm in width. The grade of steel used in this study for plates and dowels was S275. The steel plates dimensions were 480 x 152 mm^2 and the compressed wood plates dimensions were 480 x 157.5 mm^2.



(a)

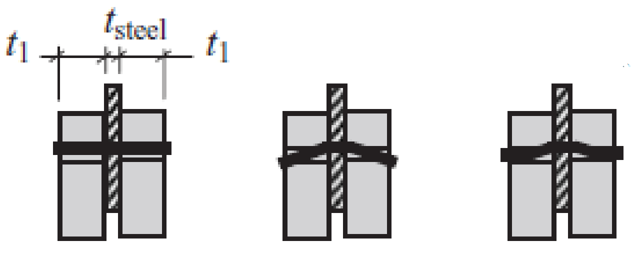
(b)

Figure 2: *Compressed wood fasteners, (a) Schematic of manufacturing of compressed wood plates and dowels, (b) finished compressed wood plates and dowels*

2.3 PRELIMINARY DESIGN OF CONNECTION SYSTEM

To assess the performance of CW fastened connections, beam-beam connections using equivalent capacity steel dowels and steel plates were produced as control specimens.

The Eurocode 5 does not adequately address the guidelines for designing of moment resisting dowel type connections. Thus, the recommendation for the minimum spacing criteria for both timber to steel and timber to timber moment resisting connection were followed as per the guidelines by Porteous and Kermani [11]. The possible failure modes under consideration for the steel-timber connection are illustrated in Figure 3 as defined in Eurocode 5 [12].



(1) (2) (3)

Figure 3: *Failure modes for steel-timber connection (Porteous and Kermani [11])*

The corresponding characteristic load carrying capacity per steel dowel per shear plane for each steel-timber connection failure mode is calculated using Equations (1)-(3).

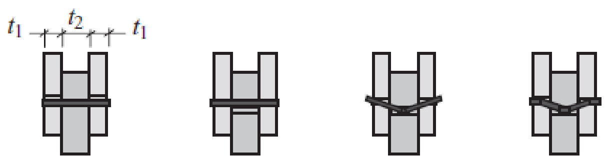
$$F_{v,Rk} = f_{h,1,k}.t_1.d \tag{1}$$

$$F_{v,Rk} = f_{h,1,k} \cdot t_1 \cdot d \left[\sqrt{2 + \frac{4M_{y,Rk}}{f_{h,1,k} \cdot t_1^2 \cdot d}} - 1 \right] \frac{F_{ax,Rk}}{4} \qquad (2)$$

$$F_{v,Rk} = 2.3\sqrt{M_{y,Rk} \cdot f_{h,1,k} \cdot d} + \frac{F_{ax,Rk}}{4} \qquad (3)$$

where:
$F_{v,Rk}$ = Characteristic load carrying capacity of per steel dowel per shear plane
t_1 = Timber board thickness
$f_{h,1,k}$ = Embedment strength
$M_{y,Rk}$= Yield moment of fastener
$F_{ax,Rk}$ = Withdrawal capacity of fastener.

The load carrying capacity of the CW-timber connection is calculated based on failure modes illustrated in Figure 4. The timber-timber connection has one more failure modes compared to the timber-steel connection.



(4) (5) (6) (7)

Figure 4: *Failure modes for the timber-timber connection [11]*

The corresponding characteristic load carrying capacity per CW dowel per shear plane for each failure mode is calculated using Equation (4)-(7).

$$F_{v,Rk} = f_{h,1,k} \cdot t_1 \cdot d \qquad (4)$$

$$F_{v,Rk} = 0.5 f_{h,2,k} \cdot t_2 \cdot d \qquad (5)$$

$$F_{v,Rk} = 1.05\frac{f_{h,1,k} \cdot t_1 \cdot d}{2+\beta}\left[\sqrt{2\beta(1+\beta) + \frac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k} \cdot t_1^2 \cdot d}} - \beta\right] + \frac{F_{ax,Rk}}{4} \qquad (6)$$

$$F_{v,Rk} = 1.15\sqrt{\frac{2\beta}{1+\beta}}\sqrt{M_{y,Rk} \cdot f_{h,1,k} \cdot d} + \frac{F_{ax,Rk}}{4} \qquad (7)$$

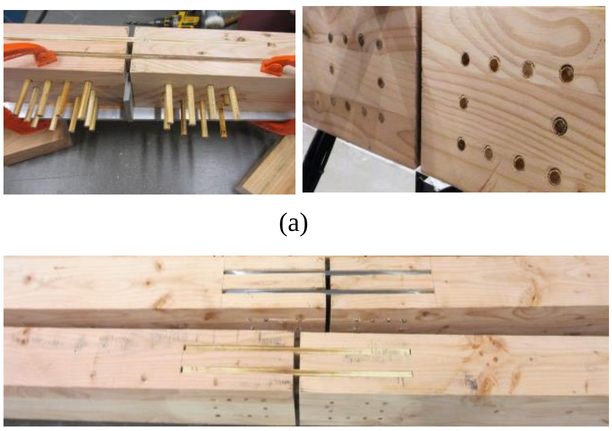
where,
$f_{h,2,k}$ = Embedment strength of central timber element
t_2 = timber board thickness of the central timber element
β = The ratio between the embedment strength of connected members
and the rest are as described previously.
The yield moment of the steel dowel was calculated as per Eurocode 5 [12]. The characteristic yield moment ($M_{y,RK}$) of the steel dowel was 27,412 N-mm. Whereas the characteristic yield moment of the compressed wood dowel was calculated from experimental three point bending tests in accordance with ASTM 1575 [13]. The characteristic yield moment ($M_{y,Rk}$) of the compressed wood dowels was 13.151 N-mm.
The assumed characteristic embedment strength of the compressed wood plate was 125.80 (N/mm^2) as reported by Jung et al. [7]. Whereas the embedment strength of the glulam was calculated as per Eurocode 5 [12]. Since the embedment strength is the function of the diameter and characteristic density of the material. Therefore, the embedment strength of glulam beam for a steel dowel of 8 mm diameter is 31 N/mm^2 and for a compressed wood dowel of 10 mm is 31.68 N/mm^2.
The failure mode is that associated with the minimum value from each set of equations, which is the characteristic load carrying capacity per fastener per shear plane. The connection parameters and calculated theoretical moment resistance of both connection systems are tabulated in Table 1.

Table 1*: Theoretical moment resistance of both steel-timber beam and compressed wood-timber beam connection systems*

Design Parameter	**Steel-timber connection**	**Compressed wood-timber connection**
Dowel diameter	8	10
No. of dowels	10	10
No. of plates	2	2
Design moment capacity of glulam	5.48 kN-m	5.48 kN-m
Design Moment capacity of the connection	3.03 kN-m	2.90 kN-m

2.4 FABRICATION OF CONNECTION SYSTEM

The fabrication of the beam-beam spliced connection system can be seen in Figure 5. In Figure 5a, the beams are fixed in position using clamps while the CW plates are positioned prior to dowel insertion. Once aligned, the dowels were driven into rectangular pattern to form the connection.



(a)

(b)

Figure 5*: Spliced connection system, (a) fabrication process, (b) plan view of steel and compressed wood connection system.*

In Figure 5b, the completed timber-steel connection and the timber-timber connection systems can be seen. The designed spacing edge and end distances are summarised in Table 2.

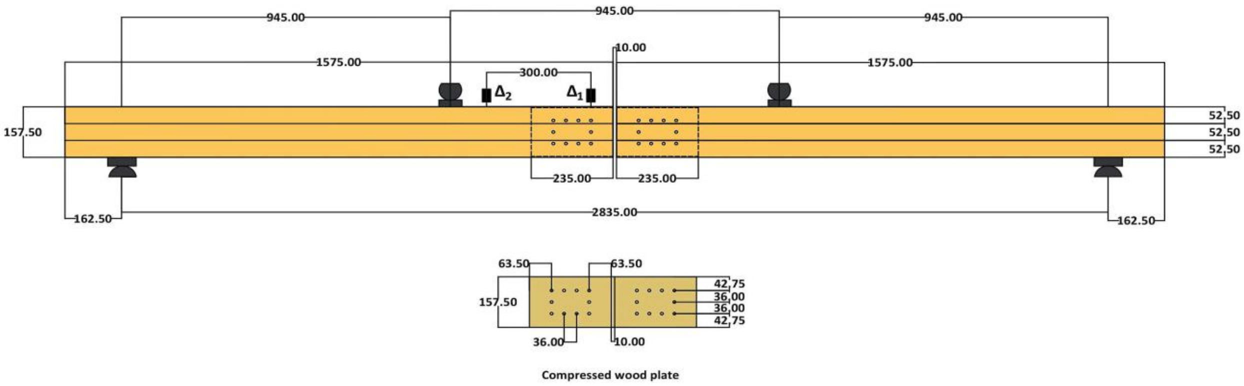


Figure 6: *Configuration of test set up for four-point bending as per EN408*

***Table 2**: Spacing, edge and end distances for designed connection system*

Spacing, edge and end distances		Spacing (mm)
Spacing	Parallel to grain	36
Spacing	Perpendicular to grain	36
End	Loaded end	63.5
End	Unloaded end	63.5
Edge	Loaded edge	42.5
Edge	Unloaded	42.5

2.5 STRUCTURAL TESTING

The structural tests were conducted at the laboratory of Timber Engineering Research Group (TERG) at the National University of Ireland Galway. The beam specimens were tested in flexure over a simply supported span in four-point bending in accordance with EN 408 [14]. Figure 6 illustrates the beam-beam connection test set-up with the spliced connection at mid-span. As recommended by Wang et al. [15], a gap of 10 mm was used to avoid friction between the beams. Simple lateral supports were placed at the end of the beams to avoid lateral movement. This ensures that the connection was subject to a pure bending load. The testing set up is illustrated in Figure 7.

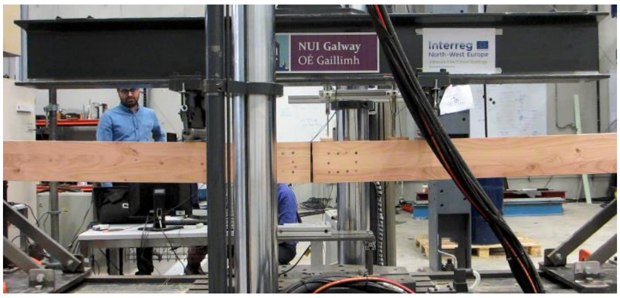


***Figure 7**: Four-point bending test set up*

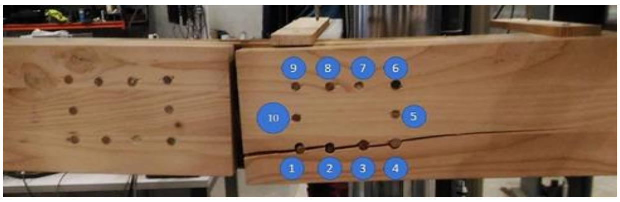
The continuous load was applied at the rate of 0.15 mm/second using a Dartec 500 kN Servo hydraulic testing machine. The vertical displacement at the mid-span of the connection was measured by a micro-epsilon optoNCDT1420 laser with an accuracy of 8µm. Two linear variable differential transformers (LVDTs), Δ_1 and Δ_2 were placed at a fixed spacing of 300 mm on one side of the connection as illustrated in Figure 6.

The respective movement of the LVDTs allowed the rotation angle (θ) of the connection to be calculated by $(\Delta_1\text{-}\Delta_2)/300$. Each connection system was initially preloaded up to 40% of maximum load and unloaded in accordance with EN 408 [14]. The vertical load was continuously applied until significant failure took place. Each specimen failed within 300 ± 120 seconds of commencing the test in accordance with EN408 [14]. The flexural stiffness, rotational stiffness, maximum failure load and maximum bending moment of the connections were determined.

3 TEST RESULTS

3.1 TYPICAL FAILURE MODES

Initially, when the vertical load is applied, the connection at mid-span begins to rotate and the beams began to move closer to each other at the top of the connected beams and begin to move apart at the bottom of the connection. The 10 mm gap at the splice connection ensured no additional friction or embedment effects occurred at this point. Loading continued until tension splitting took place along the bottom row of one of the connected ends. The tension splitting initiated at dowel number 1 as illustrated Figure 8. With increasing load, splitting propagated along the bottom row of the connection. The same failure mode was also observed in the connection systems fabricated using steel fasteners.



(a)

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***Figure 8**: Typical failure modes (a) Connection system using compressed wood fasteners (b) Connection system using steel fasteners*

3.2 LOAD-DISPLACEMENT RESPONSE

In Figure 9, the load-displacement behaviour of the beam-beam connections can be seen. The behaviour can be seen to be linear elastic until failure when the splitting on the timber beam occurs. Additional capacity can be seen after the initial failure of dowel number 1 as presented in Figure 8.

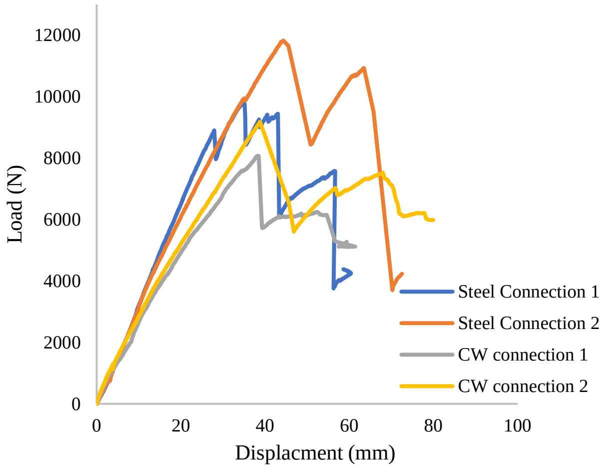

***Figure 9**: Load v/s displacement curve for both the connection systems: steel and compressed wood.*

The steel-timber beam connections achieved a greater overall load carrying capacity than that of the CW timber-timber beam connection. The maximum load carrying capacity of each connection is tabulated in Table 3.

***Table 3**: Maximum load carrying capacity of steel-timber beam connection and the CW-beam connection*

Connection ID	Maximum load (kN)
Steel connection 1	9.8
Steel connection 2	11.8
CW connection 1	8.1
CW connection 2	9.2

As shown in Table 4, the mean failure load of 10.8 kN was achieved for the steel-timber beam connections and the mean failure load of 8.6 kN was achieved for the CW timber-timber beam connections. The is an overall percentage decrease of 20.3% for the CW dowel connection.

***Table 4:** Mean failure load of the steel-timber beam connection and the CW-timber beam connection.*

Connection type	Mean failure load (kN)
Steel-timber	10.8
CW-timber	8.6

3.3 BENDING STIFFNESS

The mean bending stiffness results of the spliced beams, calculated in accordance with EN 408 [14]. As observed in Figure 9, both the CW timber-timber and steel-timber connection systems behave in a linear elastic manner until brittle failure. The bending stiffness results for spliced beams are presented in Table 5 and the mean results are presented graphically in Figure 10.

Table 5: Bending Stiffness of the spliced beams

Connection ID	Bending Stiffness (Nmm^2)
Steel connection 1	1.38×10^{11}
Steel connection 2	1.08×10^{11}
CW connection 1	0.68×10^{11}
CW connection 2	0.83×10^{11}

The bending stiffness of the beams is greater for the steel-timber beam connections. The percentage decrease in mean stiffness between the steel-timber beam connections and the CW timber-timber beam connections is 38%. This is a promising result for the CW system. Further planned testing will examine this initial finding.

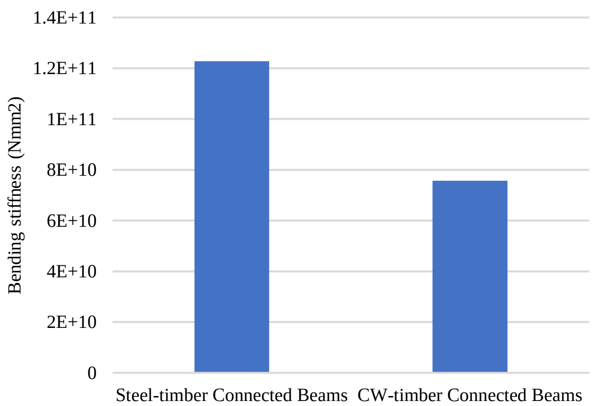

***Figure 10**: Mean bending stiffness results of steel-timber connected beams and the CW-timber connected beams*

3.4 MOMENT RESISTANCE

Usually, multiple-fastener connections often fail in splitting mechanism due to non-uniform load distribution and the concentration of stress around the fastener's hole [16]. Figure 9 validates the brittle failure of the designed connection systems. The maximum moment of each connection specimen was calculated based on the peak point of the moment vs rotation curves. The maximum moment capacity of the tested connection systems can be seen in Table 6. The steel-timber connections achieved a greater moment capacity when compared to the CW timber-timber connections.

Table 6*: Moment resistance (kN.m) for both the connection systems:*

Connection ID	Maximum Moment (kN.m)
Steel connection 1	9.28
Steel connection 2	11.17
CW connection 1	7.63
CW connection 2	8.67

The mean maximum moment for both the connection systems were tabulated below in Table 7.

Table 7: *Mean maximum moment capacity for both connection systems*

Connection type	Maximum Moment (kN.m)
Steel-timber	10.2
CW-timber	8.1

The mean maximum moment of the steel-timber connection was 10.2 kN.m and the mean maximum moment of the CW timber-timber connection was 8.1 Kn.m. This represents a percentage increase of 20.5%. The mean values presented are greater than the design values tabulated in Table 1.

3.5 MOMENT-ROTATIONAL ANGLE

The bending moment (M) and corresponding rotational angles (θ) were calculated based on the load and displacement measured from load cell and displacement transducers, respectively. Figure 11 illustrates the M-θ relationship for both the steel and compressed wood connection systems until the first point of failure as seen in Figure 9.

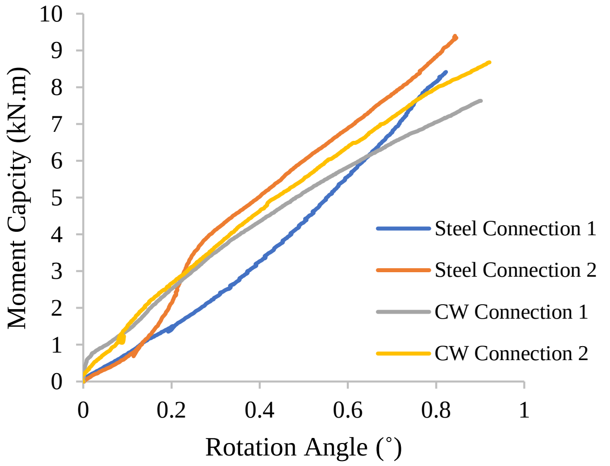

Figure 11: *Moment v/s rotational angle curve for both the connection systems: steel and compressed wood*

The initial rotational stiffness for both the connection systems, calculated based on 20% and 40% of the maximum moment and corresponding rotational angle [17]. The initial rotational stiffness of both the connection systems were summarised in Table 8.

Table 8: *Initial rotational stiffness (kN.m/rad) of steel-timber beam connections and CW-timber beam connections*

Connection ID	Initial rotational stiffness (kN.m/rad)
Steel connection 1	548.5
Steel connection 2	966.3
CW connection 1	630.2
CW connection 2	603.6

Table 9*: Mean initial rotational stiffness of both the steel-timber and CW-timber connection*

Connection type	Initial rotational stiffness (kN.m/rad)
Steel-timber	757.4
CW-timber	616.9

At the failure moment in Figure 11, the compressed wood-timber connections have shown greater connection rotation when compared to steel-timber connection. The mean initial rotational stiffness of the steel timber is 18.55% higher than that of CW-timber connection. This is as expected due to the lower stiffness of the CW-timber connection.

Future tests will further examine the moment capacity and rotational angle of such connections. The connections in this test failed due to splitting of the timber along the bottom row and there was no ductility observed in the connection. In an attempt to increase the connection ductility, future tests on spliced beams utilising a reduced number of fasteners and greater fastener spacing will be examined.

4 CONCLUSIONS

The bending test results delivered insights into the effects of CW dowels and plate configurations on the load carrying capacity, bending stiffness, maximum moment capacity and rotational stiffness of the connection system. The performance compared favourably with the equivalent steel connections. The results obtained have substantiated CW fasteners as potential green alternative to adhesives and metallic fasteners.

Tests have demonstrated similar failure modes when comparing steel-timber and CW timber-timber connection systems. The mean performance of the CW-timber connection is less than that of the steel-timber connection system, when comparing ultimate failure load, bending stiffness, moment carrying capacity and rotational stiffness however this is to be expected due to the mechanical properties of the CW elements compared to that of steel. The mean failure load for CW-timber connection is only 20.3% less than that achieved for the steel-timber connections. The mean rotational stiffness of CW-timber connection is 18.55% less than that achieved for the steel-timber connections. These preliminary results demonstrate the potential for the use of CW elements in the manufacture of timber connections.

Both connection systems demonstrated brittle failure. Further testing is proposed to induce ductile failure and to understand the effect of connection geometry, dowel diameter, number of dowels, spacing of dowels, number and spacing of CW plates.

5 FUTURE WORK

Eurocode 5 is the current harmonised design standard for timber structures in Europe. Currently, there are no rules governing the use of timber-timber connections and by extension, timber-timber connections utilising CW elements. The current equations for the lateral load carrying capacity of fasteners in Eurocode 5 are known as the Johansen equations. Such equations can be modified to allow the use of hardwood and CW timber fasteners. Additional equations must be considered to utilise CW plate elements within such connection systems. As a result, a series of material characterisation tests are proposed to establish be properties of CW fasteners and CW plates. To utilise CW fasteners, the bending moment capacity, must the established. To utilise CW plates, the embedment strength must be considered, and this must be examined parallel and perpendicular to the grain for use in moment resisting connections.

The tests performed on connections utilising CW fasteners and plates may then be compared to design values from Eurocode 5. The current Eurocode values for CW connections, presented in Table 1, are based on a small number of tests on CW dowels and assumed properties of CW plates sourced within the literature. While the mean experimental test results are in excess of the calculated Eurocode design values, further testing is required to allow for comparisons to characteristic values.

ACKNOWLEDGEMENT

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