THIN-WALLED TIMBER STRUCTURES

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Abstract: Due to their efficiency, lightweight, ease of erection and low cost, steel and aluminium thin-walled structures have become very popular in the construction industry over the past few decades. Applications include roof and wall systems (purlins and girts), storage racks, and composite concrete and steel slabs. The effectiveness of these structures lies in the cross-sectional shape of the profiles which enhances their strength by controlling the three fundamental buckling modes: local, distortional, and global. However, despite the attractiveness of these structures, steel and aluminium are greenhouse gas intensive materials and do not produce sustainable structural products. This paper presents an investigation performed at the Griffith School of Engineering, Griffith University, which shows manufacturing these types of profiles in timber is possible. Short composite thin-walled timber Cee-sections (500 mm long) were fabricated by gluing together thin softwood (*Araucaria cunninghamii*) veneers (1 mm thick). Two types of Cee-sections were considered, one with a web stiffener to increase the local buckling capacity of the profile and one without. The profiles were tested in compression and the test results are presented and discussed in the paper in terms of structural behaviour and performance. Further research directions are proposed in order to provide efficient and lightweight sustainable structural products to the timber industry.

1. INTRODUCTION

Cold-formed steel and aluminium thin-walled profiles are commonly used in the construction industry due to their efficiency, lightweight, ease of erection and low cost [1]. The worldwide market for these products is significant and estimated at more than two billions dollars. Typical industrial and civil engineering applications include roof and wall systems (purlins and girts), steel storage racks and composite concrete and steel slabs. Typical cold-formed steel profiles are shown in Figure 1.

Due to the nature of the manufacturing process, consisting of bending a thin sheet of metal to a desired cross-sectional shape, open profiles such as Cee- or Zee-cross-sections are generally used. Three fundamental buckling modes, referred to as local, distortional and global (see Figure 2), can occur for these profiles and usually govern the compressive and bending strength. The section capacity for local buckling is typically enhanced by adding intermediate stiffeners to walls having significantly large width-to-thickness ratios (see Figure 1), while the section

capacity for distortional buckling can be enhanced by changing the cross-sectional shape, mainly by adding lip stiffeners.

Figure 1: Examples of cold-formed steel profiles, with and without intermediate stiffeners

Figure 2: Fundamental buckling modes for thin-walled open cross-sections in compression, (a) local, (b) distortional and (c) global,

However, despite the attractiveness of these types of profiles, steel and aluminium are greenhouse gas intensive materials and do not produce sustainable structural products [2]. Manufacturing profiles similar to the ones shown in Figure 1 in timber is possible, but to the authors' best knowledge, has not yet been investigated in the published literature. This paper presents the first step of a study aiming at developing extra light and structurally sound timber profiles for applications including amongst others (i) emergency shelters which can be rapidly assembled and disassembled, (ii) purlins for major timber buildings, as a substitute to steel purlins currently used and (iii) wall stubs.

Short composite thin-walled timber Cee-sections (500 mm long) were fabricated by gluing together thin softwood (Hoop pine - *Araucaria cunninghamii*) veneers. Two types of Cee-sections were considered, namely one with a web stiffener to increase the local buckling capacity of the profile and one without, and this paper introduces the steps involved in the manufacturing process. The sections were tested in compression and the test results are presented and discussed in the paper in terms of structural behaviour and performance. Further research directions are also proposed.

2. INVESTIGATED SECTIONS

2.1 General

Three sets of two 210 mm deep $\times$ 105 mm wide Cee-sections (with 50 mm lip stiffeners) were manufactured and investigated. The two cross-sections in a set are of different type. Type A cross-section had no intermediate stiffeners, as shown in Figure 3 (a), and the cross-section was designed so local buckling of the web would govern the strength of the profile. On the contrary, a web stiffener was added to the second type of cross-section (Type B), as shown in Figure 3 (b), so the local buckling capacity of the cross-section would theoretically be higher than the maximum compressive strength of the material. The nominal section properties of the cross-sections are given in Table 1.

Figure 3: Cee-sections manufactured and tested, (a) Type A and (b) Type B

Each cross-section was composed of 5 layers of nominal 1 mm thick Hoop pine rotary sliced veneers. The grain of the three inner layers was orientated along the member longitudinal axis, while the grain of the two outer layers was perpendicular to the inner layers and orientated in the member transverse direction (see insert in Figure 3). In this configuration, the three inner layers would mainly resist the compressive load while also providing resistance against bending of the walls in the longitudinal direction, and the two outer layers would mainly provide resistance against bending of the walls in the transverse direction, therefore enhancing the local buckling capacity of the profile.

Table 1: Nominal section properties

Type	Area (mm ²)	$I_{\text{major axis}}$ (mm ⁴)	$I_{\text{minor axis}}$ (mm ⁴)	Warping (mm ⁶)	J (mm ⁴)
A	2375	1.58×10^7	3.92×10^6	4.97×10^{10}	1.98×10^4
B	2466	1.58×10^7	4.64×10^6	5.03×10^{10}	2.06×10^4

2.2 Manufacturing process

Veneers were delivered in sheets of $1.2\text{ m} \times 1.3\text{ m}$ and sheets with a minimum number of or with no natural defects (knots, resin veins, etc...) were selected. Different veneer sheets were used for each layer constituting a profile. Yet, all cross-sections in a set were manufactured from the same sheets, glued in the exact same order, allowing comparison between the two types of cross-sections in a set.

Each cross-section was manufactured as:

- Step 1: Sheets were cut to size and veneers were soaked in water for 48 hours.
- Step 2: The veneers were laid flat on a bench, with the grain in the appropriate orientation (see section 2.1), heated using a steamer and bent around a jig to form the cross-section.
- Step 3: Each flat side of the cross-section was clamped to the jig, as seen in Figure 4 for Type B cross-section.
- Step 4: The jig was placed in an oven at 40°C for about 12 hours, till the bends were dried and hold their shapes.
- Step 5: The veneers were unclamped and removed from the jig, further left to dry in the oven for about 4 hours, and stored in an air-conditioned room until their moisture content reached equilibrium.
- Step 6: The veneers were glued around the jig at ambient temperature with Resorcinol formaldehyde structural adhesive. Similar to Step 3, each flat side of the cross-section was clamped to the jig. Rubber sheets were inserted between the veneers and the clamping plates to uniform the applied pressure. The glue was left to set for a minimum of 48 hours before unclamping the cross-section.
- Step 7: The excess wood in the lip stiffeners were cut to form the final cross-sections.

Figure 4: Type B Cee-section during manufacturing Step 3

Figure 5: Final Cee-sections

To simplify the manufacturing process and obtain more reliable products, especially for the bends, a vacuum press is planned to be used in the future. Figure 5 shows a photo of the final products.

To determine the mechanical properties of each set of cross-sections, flat panels were also manufactured using the same sheets as their associated cross-sections glued in the exact same order and orientation. Before testing, cross-sections and flat panels were left in an air-conditioned room until their moisture content reached equilibrium.

3. MATERIAL TESTING

3.1 MOE parallel to grain

The Modulus of Elasticity (MOE) parallel to the grain of the veneers used to manufacture the cross-sections was estimated using a non-destructive resonance method [3]. A 300 to 500 mm long $\times$ 30 mm wide sample of each veneer was simply supported on rubber bands and impacted with a hammer. The natural frequency of the tested sample was recorded using a microphone and analysed using the software BING[®] (Beam Identification by Nondestructive Grading) [4]. Table 2 gives the measured MOE parallel to the grain of each veneer used.

Table 2: Modulus of Elasticity for each veneer used

Set	MOE parallel to the grain (MPa)				
	Layer 1 (Inside)	Layer 2	Layer 3	Layer 4	Layer 5 (Outside)
1	12969	13279	11955	12471	14648
2	20399	19317	16811	19164	15113
3	17187	16186	16725	18776	14401

3.2 MOR parallel to the longitudinal axis

The flat panels of each set of cross-sections were cut in four and reglued to form 20 mm nominal thick panels. Nominal 100 mm long $\times$ 46 mm wide samples were then tested in compression parallel to the longitudinal axis of the cross-sections, in a 500 kN capacity MTS testing machine, following the method proposed in the Australian and New-Zealand Standard AS/NZS 4357.2 [5]. The apparent Modulus of Rupture MOR_{app} is calculated as,

$$MOR_{app} = \frac{P_{max}}{bd} \quad (1)$$

where P_{max} is the maximum applied load, and b and d are the measured width and depth of the sample, respectively. Table 3 gives the average apparent MOR for each set of cross-sections.

Table 3: Average apparent Modulus of Rupture for each set of cross-section

Set	Number of tests	Average MOR_{app} (MPa)	CoV
1	2	29.1	0.014
2	3	36.3	0.034
3	3	34.8	0.023

4. STUB-COLUMN TESTS

4.1 Test set-up

The profiles were tested in compression in a 500 kN capacity MTS testing machine in which the lower platen was fixed, while the upper platen was mounted on a half sphere bearing which could rotate so as to provide full contact between the platen and the specimen, as shown in Figure 6. The tests were performed in displacement controlled at a stroke rate of 0.6 mm/min. Load was applied through the theoretical centroidal axis of the cross-sections.

This test arrangement, i.e. consisting of testing short columns of lengths typically less than twenty times their least radius of gyration, aims at determining the effect of local buckling on the column performance and allows the determination of the column strength (or section capacity for local buckling) [6].

For Type A cross-sections only, three Linear Variable Displacement Transducers (LVDT) recorded the horizontal displacements of the centreline of the web at heights of 175 mm (LVDT 2), 250 mm (mid-height – LVDT 1) and 325 mm (LVDT 3), and one LVDT recorded the horizontal displacement of the centreline of the north flange at mid-height (LVDT 4), as shown in Figure 6.

Figure 6: Stub-column test set-up (shown for Type A cross-section), elevations: (a) flange view, (b) web view

4.2 Test results

Table 4 gives the recorded section capacity N_s of each cross-section (defined as the maximum applied load) and the associated squash load N_{squash} , representing the upper bound capacity of the cross-section, defined as,

$$N_{squash} = MOR_{app} \times A \quad (2)$$

where MOR_{app} is given in Table 3 and A is the nominal cross-sectional area given in Table 1.

Table 4: Stub-column test results

Set	Type A			Type B		
	N_s (kN)	N_{squash} (kN)	N_s/N_{squash}	N_s (kN)	N_{squash} (kN)	N_s/N_{squash}
1	41.3	69.1	0.60	54.7	71.8	0.76
2	47.5	86.2	0.55	63.5	89.5	0.71
3	49.8	82.7	0.60	61.3	85.8	0.71
Average	46.2	79.3	0.58	59.8	82.4	0.73

It can be seen from Table 4 that the addition of the web stiffener in Type B cross-section resulted in an increased average section capacity of 29% for an increased cross-sectional area of 3.8%. Type A and B cross-sections reached an average of 58% and 73% of their upper bound capacity N_{squash} , respectively.

Figure 7 and Figure 8 plots the recorded displacements of the four LVDTs and shows the failure mode, respectively, for the Type A cross-section of the third set. It can be noticed in Figure 7 and Figure 8 a large post-buckling behaviour, similar to the one encountered in steel structures.

Figure 7: Test results for set n°3, Type A cross-section

Figure 8: Failure mode for set n°3, Type A cross-section

4.3 Comparison with cold-formed steel structures

Using typical densities of 550 kg/m^3 for hoop pine [7] and $7,850 \text{ kg/m}^3$ for steel, the cross-sectional area required for a steel profile to equal the linear weight of the studied cross-sections is in the range of 166 (Type A cross-section) to 173 mm^2 (Type B cross-section). This range of cross-sectional area would lead to a maximum section capacity N_s of 74.7 kN to 77.9 kN for 450 MPa graded steel, i.e.

about 39% and 22% greater than the average observed capacities of cross-sections Type A and B, respectively. Yet, these steel profiles would likely have slender wall elements leading to low local buckling capacities and therefore reduced column strength.

For instance, the C7510 cold-formed steel Cee-section (75 mm deep and 1.0 mm wall thickness) from the Australian manufacturer Lysaght [8] has a nominal cross-sectional area of 162 mm^2 and a stub-column capacity of 46.2 kN [9] for 450 MPa graded steel. This capacity is equal and 29% less than the average observed capacities for cross-sections Type A and B respectively. Moreover, the bending stiffness EI about the major axis of bending of the C7510 is equal to $2.88 \times 10^{10} \text{ N.mm}^2$ ($E = 200,000 \text{ MPa}$ and $I = 1.44 \times 10^5 \text{ mm}^4$), while the bending stiffness of Type A cross-section is equal to $1.23 \times 10^{11} \text{ N.mm}^2$ ($E = 13,000 \text{ MPa}$ at 12% moisture content [7] and $I = 9.45 \times 10^6 \text{ mm}^4$ when only considering the three inner layers resisting bending), i.e. 4.3 times greater than the C7510 steel profile.

Similarly, Yap and Hancock [10] performed stub-column tests on 128 mm deep and 0.42 mm wall thickness complex high strength cold-formed steel open profiles. The nominal yield stress of the profiles was equal to 550 MPa, the cross-sectional area to 158 mm^2 and the average tested stub-column capacity to 45.7 kN, i.e. 1% and 31% less than the average observed capacities for cross-sections Type A and B respectively. The profiles tested in [10] had a bending stiffness about the strong axis of bending equal to $4.74 \times 10^{10} \text{ N.mm}^2$ ($I = 2.37 \times 10^5 \text{ mm}^4$), i.e. about 2.6 times less than the studied timber profiles.

5. FUTURE STUDIES

By advancing the structural and mechanical knowledge on thin-walled timber structures, the final aim of this project is to eventually develop design rules, so engineers can safely use the proposed products. Current researches involve developing a Finite Element model to reproduce the test results presented in this paper. Moreover, the ultimate strength and post-buckling behaviour of thin-walled structures is significantly influenced by geometric imperfections [11] and understanding the influence of the moisture content on these imperfections, and therefore on the capacity of the cross-section, is important. Additional proposed future research also includes (i) developing joining details to connect manufactured lengths of cross-sections, (ii) testing and understanding the structural behaviour of long columns and beams, (iii) extend the Finite Strip Method [12] currently used to determine the buckling curves of thin-walled isotropic and orthotropic materials to composite materials, and (iv) extend the Direct Strength Method [13] currently used to design cold-formed steel profiles to thin-walled timber profiles.

6. CONCLUSION

This paper presented an experimental investigation on thin-walled timber structures. 210 mm deep, 105 mm wide and 5 mm wall thickness composite timber Cee-sections were fabricated by gluing together thin softwood (Hoop pine - *Araucaria cunninghamii*) veneers. Two types of cross-section, with and without inter-

mediate web stiffener, have been investigated. The manufacturing process is detailed in the paper. Stub-column tests were performed and results show that (i) manufacturing structurally sound thin-walled timber structures is possible, (ii) the investigated cross-sections have significant post-buckling behaviour and (ii) are able to compete with cold-formed steel products with similar or greater weight to compressive capacity ratio and significantly higher bending stiffness.

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