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DEVELOPMENT OF COLD FORMED STEEL-TIMBER COMPOSITES FOR ROOF STRUCTURES: CONNECTION SYSTEMS

Ali Awaludin[1*], Anindha Dyah Danastri[2], Bambang Supriyadi[3]

[1] *Research Group of Innovative Sustainable Infrastructure Materials and Construction Technologies, Department of Civil and Environmental Engineering, Universitas Gadjah Mada, Grafika Street No. 2, UGM campus, Sleman, Yogyakarta 55281, Indonesia*
[2] *Former Graduate Student, Department of Civil Engineering and Environmental Engineering, Universitas Gadjah Mada, Street No. 2, UGM campus, Sleman, Yogyakarta 55281, Indonesia*
[3] *Laboratory of Structural Engineering, Department of Civil Engineering and Environmental Engineering, Universitas Gadjah Mada, Street No. 2, UGM campus, Sleman, Yogyakarta 55281, Indonesia*

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ABSTRACT

Cold formed steel has relatively high width-to-thickness ratio elements, which causes it to buckle easily. Combining it with timber laminas would be an effective solution for reducing this buckling problem. This research focuses on the connection system of a cold formed steel-timber composite, which was obtained by attaching several timber laminas to the web part of cold formed steel using screws. The connection used two bolts that were 8 mm in diameter, as well as two different kinds of side plates: steel and plywood. Cold formed steel 75Z08 and *Swietenia mahagoni* (moisture content 12.2%; specific gravity 0.77) were used for connections and were loaded in parallel and perpendicular directions. In addition, the connections of cold formed steel (without timber laminas) using self-drilling screw fasteners were tested until failure. Numerical analysis predicting the load-slip curve and apparent yield load of the composite joints was carried out using the DOWEL program and the European Yield Theory, respectively. The test results showed that the connection system with steel side plates is capable of accommodating the strength increase of composite member, as it has a maximum load carrying capacity and initial slip modulus of about 4.5 and 2 times larger than those of the cold formed steel connections, respectively. In the case of a connection system with plywood side plates, its joint properties are similar to those of the cold formed steel connection, except that it has larger joint deformation.

Keywords: Bolted connections; Cold formed steel; Composite; Load carrying capacity; Timber laminas

1. INTRODUCTION

Cold formed steel is a construction material that is currently very popular in Indonesia for roof structures as it has several advantages, including that it is easy to obtain, requires almost no maintenance programs, is quick to install, and has lesser weight than that of ordinary (hot rolled) steel. In practice, cold formed steel is generally used for roof structures with a span ranging from 6 to 20 m, depending on the type of roof cover materials and geometry complexities. Self-drilling screws are generally the only fastener types used in cold formed steel

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*Corresponding author's email: ali.awaludin@ugm.ac.id, Tel. +62-274-545675, Fax. +62-274-545676
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structures in order to ensure rapid construction assembly, especially at the site.

As cold formed steel members have relatively high width-to-thickness ratios, either in their webs or flanges, cold formed steel members buckle easily. A full-scale test of a cold formed steel roof structure conducted at the Structural Engineering Laboratory of Universitas Gadjah Mada, shown in Figure 1, found that this roof structure was brittle due to buckling of the diagonal chords; the collapse was instantaneous, following the achievement of the linear elastic response. The composites of cold formed steel-timber laminas, which were obtained by attaching timber laminas to the webs or flanges of cold formed steel members with screws or bolts, could potentially reduce this buckling problem in addition to enhancing its strength and stiffness. Moreover, this composite system would essentially increase the utilization of timber members with size limitations.

(a) (b)

Figure 1 Destructive test on cold formed steel roof structure: (a) before the test; and (b) after the test

Previous studies (Li, 2005; Winter et al., 2012) reported that a combination of cold formed steel and timber members increased the strength and stiffness of the system under bending tests. The strength increase of a cold formed steel-timber composite, in comparison to cold formed steel, therefore requires a different connection system, as well as different types of fasteners. Research on the connection system of cold formed steel has been conducted in previous studies (Chung & Lawson, 2000; Kwon et al., 2006; Wallace et al., 2001), but research on the connection system of cold formed steel-timber composites is not yet available, according to authors' knowledge. In this study, various connection models of cold formed steel-timber lamina composites were proposed and tested in order to verify their structural performance improvements.

1.1. Literature Review

Cold formed steel products are fabricated at ambient temperatures from steel sheets, strip plates, or flat bars by roll-forming machines, press brakes, or bending brake applications. The effect of cold working is an enhancement of mean yield stress, as stated by Brokenbrough and Merritt (1999). When compared to hot rolled steel sections having with an equal cross-sectional area, cold formed steel sections had a greater element of width or height, due to its thinner section wall. This led to a significant increase of second moment of inertia and bending moment capacity. Consequently, cold formed steel is more economical compared to hot rolled steel.

Chung and Lawson (2000) investigated the connections of cold formed steel. They used 24 connection specimens divided into four different configurations: two configurations of beam-to-beam connections and two configurations of beam-to-column joints. In their report, they identified three types of failure modes: failure of connection, shear buckling, and lateral-torsional buckling. Wallace et al. (2001) also performed an experiment on cold formed steel connections, but they used bolts as the connector, and the test results were compared to S136 and AISI specifications. They also examined the presence of washers as a parameter and found that connections equipped with washers could reach higher lateral load capacities than those without washers.

Li (2005) studied the bending behavior of composite beams of cold formed steel-timber and Oriented Strand Board (OSB) using self-drilling screws. He found that adding timber as web reinforcement and OSB as flange reinforcement yielded the best performance, with high ductility ratio in the composite beams. In 2006, Kwon et al. (2006) performed experiments on the failure of cold formed steel connections as part of his portal frame test, and they found that one failure mode that generally occurred was local buckling of the members. Other possible failure modes were local failure, severe joint deformation, and fracture of clinching.

Recently, in 2012, Winter et al. (2012) conducted research on composites of double cold formed U steel section–glulam, or X–lam, using screws and gun-driven nails. They prepared four different composite arrangements, as shown in Figure 2, and tested them under a four-point bending configuration. Beams with the composite arrangement shown in Figure 2c were the weakest due to lateral-torsional buckling, while failure of the other beams was due to fracture in the tension area of the timber beams. Chung and Lawson (2000) investigated the connections of cold formed steel. They used 24 connection specimens divided into four different configurations: two configurations of beam-to-beam connections and two configurations of beam-to-column joints. In their report, they identified three types of failure modes: failure of connection, shear buckling, and lateral-torsional buckling. Wallace et al. (2001) also performed an experiment on cold formed steel connections, but they used bolts as the connector, and the test results were compared to S136 and AISI specifications. They also examined the presence of washers as a parameter and found that connections equipped with washers could reach higher lateral load capacities than those without washers.

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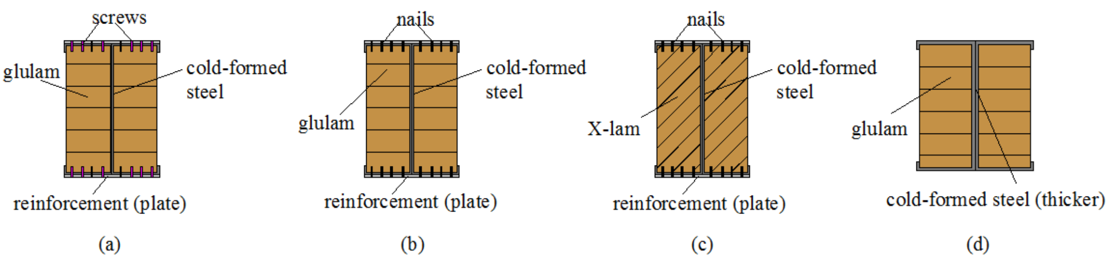

Figure 2 Four composite arrangements of double cold formed U steel section–glulam, or X–lam (Winter et al., 2012)

2. MATERIALS AND METHODS

This study used cold formed steel 75Z08 (yield strength 592 MPa) with a thickness of 0.8 mm and height of 75 mm, as shown in Figure 3a. This steel section was combined with four 14 mm timber laminas, *Swietenia mahagoni*, using screws with a 4 mm root diameter, as illustrated in

Figures 3b-3d. In total, there were twelve composite joint specimens: six joints loaded in a parallel loading direction and another six joints loaded in a perpendicular loading direction. In both the parallel and perpendicular loaded joints, two different side members were used: 4 mm thick steel plates and 18 mm thick plywood, as shown in Figure 4. Each joint used two 8 mm diameter bolts. In addition to these composite joints, three specimens of cold formed steel joints with two self-drilling screws each (root diameter of 4 mm) were tested under a parallel loading direction.

The lateral load capacities of the connection specimens under a perpendicular loading direction were influenced by the distance of the two supports below the horizontal member (see Figure 4b). A greater distance reduced the stiffness of the horizontal member, as well as lowered the lateral load capacity of the connection. In this study, the test configuration shown in Figure 4b was adopted according to previous work (Awaludin et al., 2008). Evaluation of the dowel bearing strength of the timber laminas and the bending yield moment of the bolts were carried out according to ASTM D 5764-97a (ASTM, 2002) and ASTM F 1575-03 (ASTM, 2003), respectively.

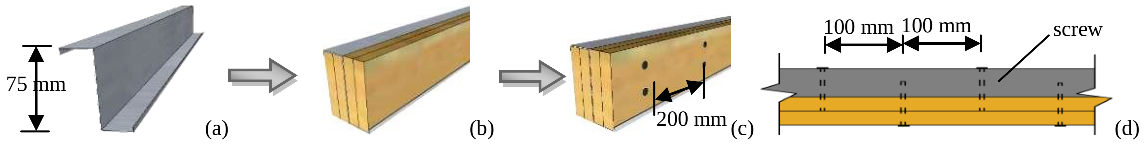

Figure 3 Composite timber laminated cold formed steel specimen

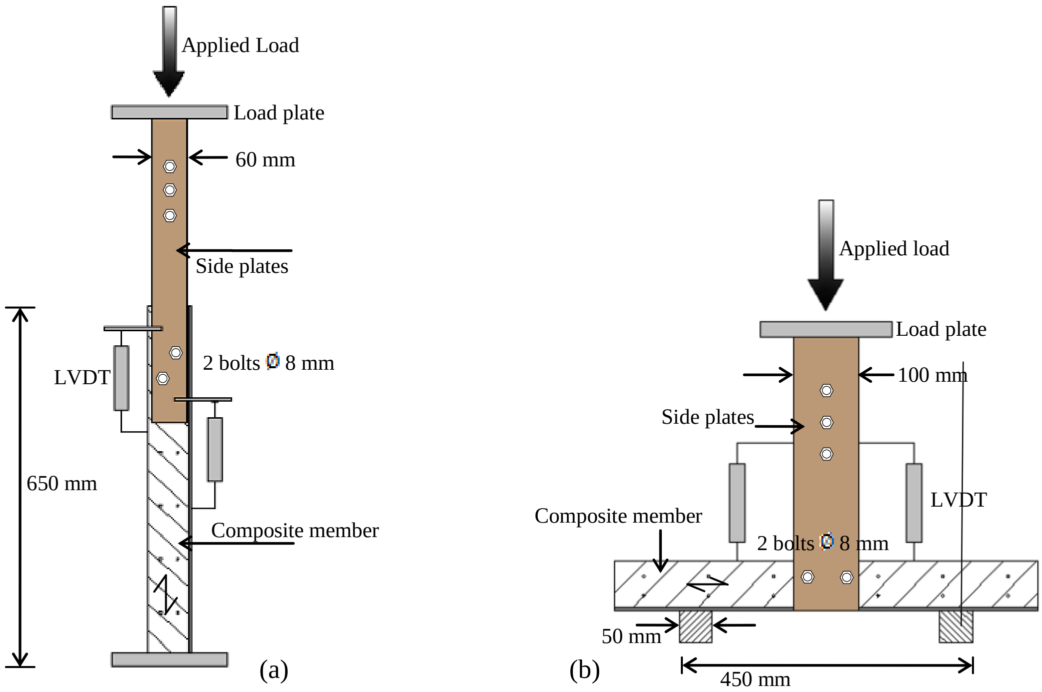

Figure 4 Setup of connection test: (a) parallel loading direction; and (b) perpendicular loading direction

All connections were loaded via a hydraulic hand pump until failure was observed or a connection slip larger than 20 mm occurred. The connection load and the slip between the main and side members were continuously measured by two attached LVDTs, as indicated in Figure 4. The connection tests were conducted at room conditions around 32°C and 64% RH. The connection load-slip curves obtained from the experiment were compared againt the numerical analysis done by DOWEL (Schreyer, 2002), and a finite element analysis code was developed based on the beam on elastic foundation theory. The bolt is represented by a two-node isoparametric beam element, which at each node has five degrees of freedom (DOF): axial displacement, axial strain, vertical displacement, slope, and curvature. Virtual work principle was applied to the equilibrium equation of the system, which was developed based on the assumption of the beam resting on elastic foundation in order to generate an element stiffness matrix, which was then assembled for the entire system. The general procedure of the Newton-Rahpson iteration was used in order to solve for the global displacement vector.

Predictions on the connection load carrying capacity was also carried out based on several possible connection yield modes, known as the European Yield Model (Blass et al., 2000), which is summarized in Appendix A. A detailed formulation is available upon request; however, due to space limitations, it has not been included in this paper.

3. RESULTS AND DISCUSSION

The results of bearing strength in parallel and the perpendicular-to-grain of *Switenia mahagoni* as well as the bending yield strength of the bolt are shown in Figure 5. Based on eighteen replicates (nine specimens for parallel-to-grain and nine specimens for perpendicular-to-grain), the bearing strength of *Switenia mahagoni* varied from 42 MPa to 49 MPa, with an average of 45.7 MPa for loading parallel-to-grain, and which varied from 34 MPa to 47 MPa with an average of 38.2 MPa for loading perpendicular-to-grain. The average moisture content and air-dry specific gravity of the timber members were 12.2% (min 12.0%, max 12.7%) and 0.77 (min 0.67, max 0.96), respectively. The average bending yield moment of the bolt derived from the two replicates was 24075 N.mm, and it was almost identical to the prediction given in a previous study (Blass et. al, 2000) based on the information about tensile yield strength and bolt diameter.

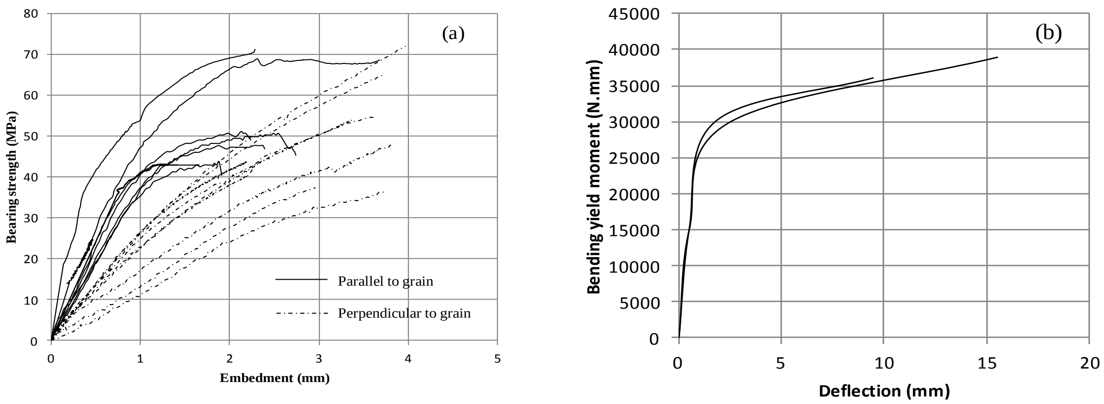

Figure 5 (a) Bearing strength of *Switenia mahagoni*; (b) bending yield strength

Load-slip curves obtained from the experiment and the numerical analyses where the composite joints with plywood side plates displayed a significant or large deformation are shown in Figure 6. The load was obtained solely from the hydraulic pressure, since the weight of the connection specimen was significantly small compared to the lateral load capacity of the connections. Each of the connection tests were completed within ten minutes, and this was about the same for all the specimens.

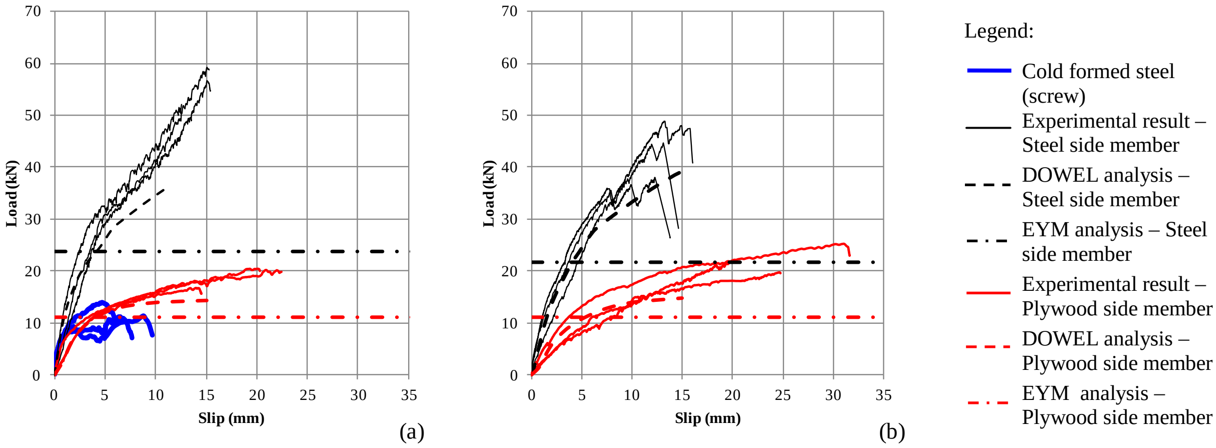

Figure 6 Load-slip curves of the connections: (a) parallel loading direction; and (b) perpendicular loading direction

In the beginning, the load-slip curves of the composite connections for both the experimental and numerical analysis showed agreement. After the elastic linear portion, however, there was variation in these curves: the curve given by the numerical analysis had a lower slope than that of the experimental curve. The lower slope of the numerical analysis is a result of the numerical model's limitation in that it was developed using the DOWEL program, where the frictional resistance between composite members and steel or plywood side members, associated with the secondary axial force development in the bolt (Awaludin et al, 2008), was not properly considered.

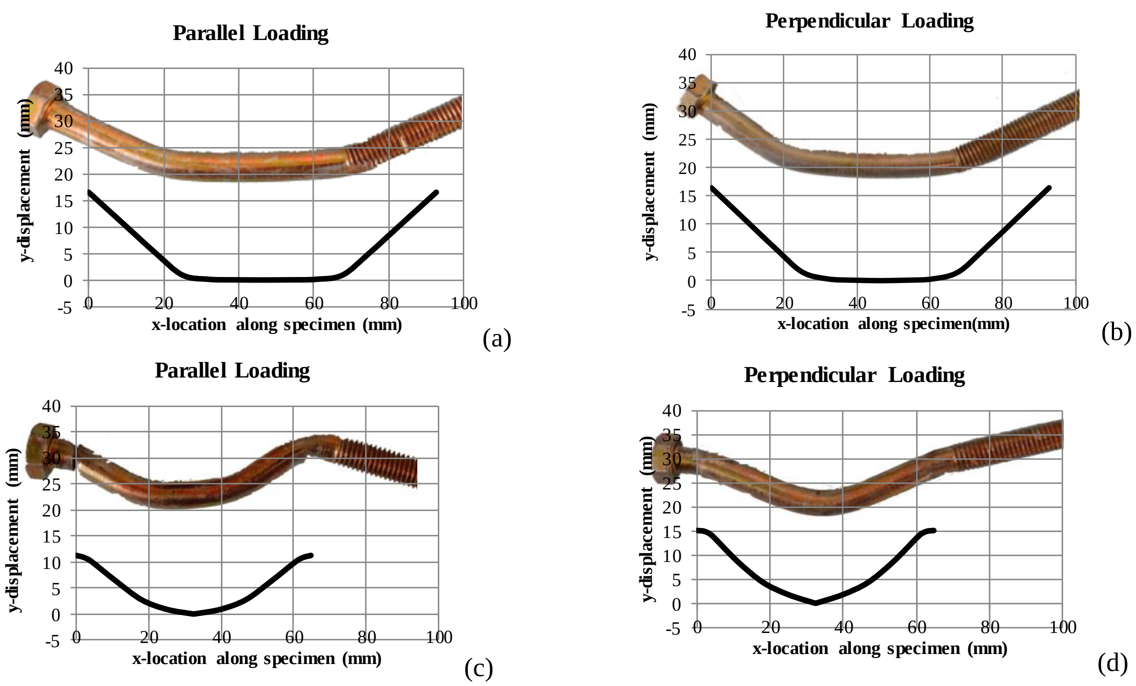

Figure 7 Bending deformation of the bolt and its prediction value given by the DOWEL program: (a)-(b) Composite connection with plywood side plates; (c)-(d) Composite connection with steel side plates

The failure mode of the composite connections was primarily due to the bending deformation of the bolts, especially in the composite connections with steel side plates, whereas the failure mode of the cold formed steel connection was solely caused by the large bearing deformation of the cold formed steel members beneath the screw fasteners. One of three composite connections with steel side plates loaded under parallel loading direction failed due to a fracture of the bolt at one shear plane, the interface between composite members and steel side plates. Figure 7 shows graphs of the bending deformation of the composite connection bolt obtained from the experiment, which well supported the findings of the DOWEL program. The bolts had several bending points, commonly called plastic hinges, because stress-strain relation at this point is no longer elastic. When plywood was used as side plating, the bolts had two plastic hinges located in the shear planes, as shown in Figures 7a and 7b, while in the case of composite joints with steel side plates, one or two more additional plastic points were developed in the bolt at the center (see Figures 7c and 7d). This evidence confirmed the prediction of EYM, where yield mode IIIs (for composite connections with plywood side plates) or IV (for composite connections with steel side plates) is the actual connection yield mode, as this mode had the lowest load carrying capacity, among other possible yield modes (see Appendix A).

Compared to cold formed steel connections, the composite connections had higher load carrying capacity and joint deformation. The load carrying capacity increase was tremendous in the case of the composite connection with steel side plates. For further discussion, some parameters summarized in Table 1 were obtained from the load-slip curves. The maximum load carrying capacity refers to the highest load reached before or at a slip of 15 mm; apparent yield load refers to the intersection between load-slip curve and a 5% diameter offset line that parallels the initial slip modulus; initial slip modulus refers to a slope in the linear portion of a load-slip curve between 10% and 40% of the maximum load; and ductility ratio refers to the ratio between the ultimate slip and the slip at apparent yield load, where the ultimate slip is the slip corresponding to the ultimate load before the connection fails with a load reduction and is not higher than 20% of the maximum load (International Organization of Standardization, 1983; AF&PA, 1999; Smith et al., 2006).

Table 1 Connection test results

Specimen	Loading Direction	Ultimate Lateral Load Capacity* (kN)		Apparent Yield Load Capacity (kN)		Initial Slip Modulus (N/mm)	Ductility Ratio
		Experiment	Prediction (DOWEL)	Experiment	Prediction (EYM)		
Connection of cold formed steel (screw)	Parallel	12.1	-	7.2	-	5452	9.9
Connection of composite member with steel plates	Parallel	55.5	36.0	27.9	23.7	7430	3.6
	Perpendicular	43.8	39.1	24.2	21.7	5592	3.2
Connection of composite member with plywood plates	Parallel	17.6	13.2	9.6	11.2	4457	≥ 7.6
	Perpendicular	18.3	12.7	9.6	11.2	2127	≥ 5.5

* Maximum load which is reached before or at 15 mm slip

Table 1 shows that the average maximum load carrying capacity, the initial slip modulus, and the ductility ratio of the cold formed steel connections equal, respectively, 12.1 kN, 5452 N/mm, and 9.9. The maximum load carrying capacity of the composite connections with steel side plate was around 4.5 times and 3.5 times of the load carrying capacity of cold formed steel connections, respectively, for the connection under parallel and perpendicular loading directions. This increase was around 1.5 times in the case of composite connections with plywood side plates for both parallel and perpendicular loading directions. The composite connections with steel side plates had a higher ultimate slip than that of the cold formed steel joints, though they had smaller ductility ratio. Among the tested connections under loading parallel direction, the composite connections with steel side plates had the highest initial slip modulus, followed by the cold formed steel connections and the composite connections with plywood side plates. These facts indicate that the composite connection system using plywood side plates has not successfully accommodated the strength increase of a composite member, unlike the connection system using steel side plates.

4. CONCLUSION

This study evaluated the connection systems of a cold formed steel-timber composites using bolts and two different side members: steel and plywood plates. The evaluation was based on the test results of several composite connections loaded under parallel and perpendicular loading directions, in addition to numerical analysis performed using the DOWEL program and the European Yield Model. The results show that the connection system using steel side plates is capable of accommodating the strength increase of a composite member, as this connection system had a much higher maximum load carrying capacity and initial joint slip modulus than the cold formed steel connections.

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Apendix A: European Yield Models for the cold formed steel-timber composite

A.1. Composite connection with steel side plate

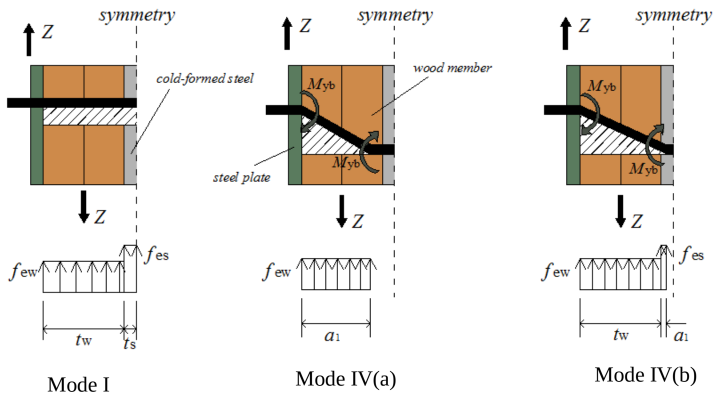

Figure A1 Yield mode of composite connection with steel side plate
(where shaded area represents the part of member material that experienced bearing failure)

Yield modes:

$$Z_I = f_{ew}.t_w.d + f_{es}.t_s.d$$

$$Z_{IV(a)} = f_{ew}.d.\sqrt{\frac{4M_{yb}}{f_{ew}.d}}$$

$$Z_{IV(b)} = f_{ew}.t_w.d + f_{es}.a_1.d$$

Coefficient a_1 is equation root of $Aa_1{}^2 + Ba_1 + C = 0$ where: $A = \frac{1}{2}f_{es}.d$; $B = f_{es}.d.t_w$; $C = \frac{1}{2}f_{ew}.d.t_w{}^2 - 2M_{yb}$

A.2. Composite connection with plywood side plate

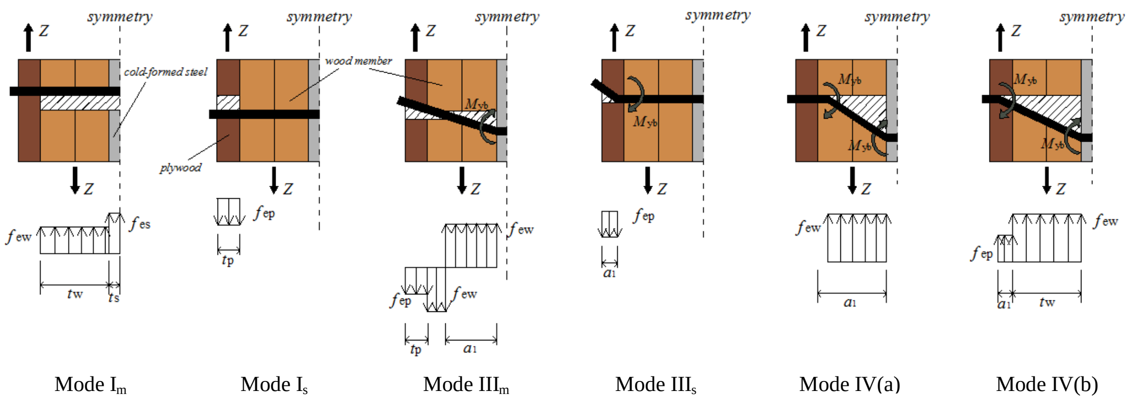

Figure A2 Yield mode of composite connection with plywood side plate
(where shaded area represents the part of member material that experienced bearing failure)

Yield modes:

$$Z_{Im} = f_{ew}.t_w.d + f_{es}.t_s.d$$

$$Z_{Is} = f_{ep}.t_p.d$$

$$Z_{IIIm} = 2.f_{ew}.d.\sqrt{\frac{M_{yb}}{f_{ew}.d} + \frac{f_{ep}}{f_{ew}}\left(t_p.t_w + \frac{1}{2}t_p{}^2\right) + \frac{1}{2}t_w{}^2} - f_{ew}.d.\left(t_w + t_p\right)$$

$$Z_{IIIs} = f_{ep}.d.\sqrt{\frac{2M_{yb}}{f_{ep}.d}}$$

$$Z_{IV(a)} = f_{ew}.\,d.\sqrt{\frac{4M_{yb}}{f_{ew}.d}}$$

$$Z_{IV(b)} = f_{ew}.\,t_w.\,d + f_{ep}.\,a_1.\,d$$

Coefficient a_1 is equation root of $Aa_1{}^2 + Ba_1 + C = 0$ where: $A = \frac{1}{2}f_{ep}.\,d$; $B = f_{ep}.d.t_w$; $C = \frac{1}{2}f_{ew}.\,d.t_w{}^2 - 2M_{yb}$

Table A1

Loading Direction	Steel Side Plate Connection (kN)				Plywood Side Plate Connection (kN)						
	Z_I	$Z_{IV(a)}$	$Z_{IV(b)}$	**Minimum**	Z_{Im}	Z_{Is}	Z_{IIIm}	Z_{IIIs}	$Z_{IV(a)}$	$Z_{IV(b)}$	**Minimum**
Parallel	56.1	23.7	-	**23.7**	56.1	11.7	14.0	11.2	23.7	-	**11.2**
Perpendicular	49.3	21.7	-	**21.7**	49.3	11.7	14.9	11.2	21.7	-	**11.2**

Where:

t_s (cold formed steel thickness) = 0.8 mm
t_w (timber thickness) = 28 mm
t_p (plywood thickness) = 18 mm
f_{ew} (wood bearing strength) = 45.7 MPa for parallel-to-grain; 38.2 MPa for perpendicular-to-grain
f_{es} (cold formed steel bearing strength) = 591.7 MPa
f_{ep} (plywood bearing strength) = 20.3 MPa
d (bolt diameter) = 8 mm
M_{yb} (bending yield moment of bolt) = 24075 N.mm