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Contribution of OSB Sheathing to Racking Capacity of Cold-Formed Steel Frames

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Abstract

In recent years, new innovative systems to ensure high structural and environmental performance have emerged in constructional steel practice. Among others, modular construction using cold-formed steel (CFS) frames which offer some advantages over conventional structural system counterparts, such as high strength-to-weight ratio, controlled material quality and sustainability. Wood-sheathed shear wall is one of the lateral load resisting systems adopted in CFS modular construction. It is composed of CFS C-shaped framing members (chord studs, studs and tracks) attached to wood sheathing using fasteners. The resistance capacity of these shear walls, when subjected to in-plan lateral loads, is governed by the connections between the frame and the sheathing board. It is therefore crucial to quantify the shear capacity of sheathing-to-frame fasteners. In this context, a testing rig was developed consisting of two CFS lipped channels facing back-to-back connected on the flanges by oriented strand board (OSB) and loaded such that the 8 connecting fasteners experi-

1. Introduction

In cold-formed steel (CFS) framed structures, sheathed shear wall is one of the primary lateral load resisting systems; it is composed of CFS C-shaped framing members (chord studs, studs and tracks) attached to steel or wood sheathing using fasteners. The behaviour that develops in the connection zone between the CFS frame and the sheathing is the main mechanism of lateral load resistance. This structural component should be designed to provide adequate lateral shear strength and stiffness to the global structure. However, the current version of the Eurocodes does not provide any guidance for sheathed CFS shear wall systems, which hinders the use of this lateral load resisting system in constructional practice. The North American Standard code of practice for seismic design of CFS Steel Structural Systems AISI S400 (2015) [1] represents the main reference for lateral design of this type of structures. Therefore, it is deemed necessary to refer to design methods assisted by testing.

Several research activities on the shear capacity of the sheathing-to-frame fasteners have been carried out. A series of screw connection tests was performed by Iuorio (2007) [2] and Fiorino et al., (2008) [3] to investigate the shear behaviour of screw connections between CFS members and oriented strand boards (OSB), gypsum panels (GWB) and cement-based panels (CP). The studies investigated the effects of sheathing typology, loading direction (in case of OSB sheathings) and loaded edge distance (three different values of

the loaded edge distance (a) were adopted a=10 mm, a=15 mm, a=20 mm). The program included 96 specimens. Five displacement-controlled test procedures were adopted: monotonic tension, monotonic compression, and three types of cyclic loading history. As tests results, the sheathing material has a significant effect on the shear connection behaviour. OSB sheathings had larger strength (2.9 larger) and absorbed more energy (4.3 times larger) than connections with GWB. On the other hand, connections with GWB sheathings revealed larger stiffness (1.3 times larger) and ductility (2.30 times larger) than OSB sheathings. In the case of OSB panels, the perpendicular-to-grain loaded connections give lower strength, ductility and stiffness compared with parallel-to-grain loading, while the absorbed energy is almost the same for both cases. As far as the loaded edge distance (a) is concerned, larger edge distance values provide larger strength and absorbed energy, with an almost linear increase. Ductility does not vary significantly when increasing the loaded edge distance. Comparison between monotonic and cyclic response shows that, for both OSB and GWB sheathings, the cyclic loading produces a non-negligible reduction of strength (more significant for OSB sheathings) and absorbed energy (more significant for GWB sheathings). Cement based panels revealed larger stiffness than any other material, with on average, values 1.6, and 3.4 times larger than that showed by GWB and OSB panels, respectively. Moreover, the ductility revealed by CP was, on average 2.2 larger than that showed by OSB panels. CP panels showed less strength and absorbed energy than connections with OSB. At the same time, strength and absorbed energy were larger than that exhibited by GWB sheathings. Ye et al., (2015) [4] to investigate the shear behaviour of the screw connections between the steel frame and the sheathings (Gypsum Board and Bolivian Magnesium Board sheathing types). It was found out that increasing the steel plate thickness has negligible effect on the strength but an obvious effect on the deformability of the screw connections, and the peak strength of the screw connections can be significantly increased by increasing the edge distance to the screws. Peterman et al., (2014) [5] found that 152.4 mm (6 in) and 304.5 mm (12 in) screw spacings that were tested on Gypsum and OSB sheathing had only minor differences in backbone curves behaviour of screw fasteners. In both experiments, the loading was in the parallel direction. Li et al., (2020) [6] presented experimental research on the load bearing behaviour of self-drilling screw fasteners in CFS shear walls sheathed with engineered bamboo panels. The performance of the CFS bamboo fasteners was studied under various conditions with respect to differently engineered bamboo sheathing panels, screw features, end distances, and loading rates. The test results obtained in this study indicate that although the variation of shear capacity of fasteners was insignificant initially when considering 10 mm (2 times the fastener diameter "d") and 15 mm (3 times d) spacings with bamboo sheathing there was more prominent difference at higher loads leading to notable failure by tilting of screws with increasing spacings. This finding may however be inconsistent as Li noted that this phenomenon was not supported when comparing screw spacings of 60 mm with 15 mm and 10 mm edge distance. The types of screws are also a variable of note, in fact it was found that the types of self-drilling screws effect the lateral shear capacity of a section, for example the ductility coefficient ratio of the phosphating screw was found to be 10-20% greater than the ratio of the stainless steel self-screw when comparing screws of the same diameters and lengths. This leads to improved elastic stiffness and peak load for the ultimate shear capacity of the screw fastener set up. Load directions have been tested by Iuorio (2007) [2] and Abu-Hamd (2019) [7] where both perpendicular and parallel loads are applied to the CFS fasteners to compare the shear capacity, the conclusion of which is that the governing load direction is parallel to the free end; the parallel loading capacity is about 45% higher than that for perpendicular loading. These results are consistent with the thickness of the sheathing findings by (Karabulut and Soyoz, (2017) [8]) whereby the thicker the sheathing board, the higher the ultimate shear strength. The loading direction also leads to conflicting failure mechanisms, whereby both the parallel and perpendicular loading is governed by screw tilting, but screws loaded in the parallel direction are observed to pull through after tilting (Abu-Hamd (2019) [7]). Li et al., (2020) [6] also noticed difference in the directional orientation of the bamboo itself, for example axial loading of unidirectional flat pressing bamboo panels leads to increased peak loads and elastic stiffness compared to that of double-direction laminated bamboo. These results regarding bamboo sheathing material are assumed to be amplified when subjected to parallel and perpendicular loading scenarios. The lateral load resistance capacity of CFS structures is mainly dependent on the lateral capacity of shear walls, which in turn depends on the interaction between the sheathing-to-framing connections – the complete set up is therefore crucial. Tao et al., (2016) [9] developed fastener backbone models for fasteners after investigating the full connection behaviour under both monotonic and cyclic loading, this highlights the importance of the setup variables in light framed steel seismic analysis. Corner (2014) [10] proposed a model for predicting tilting angle and limit states of single-fastened cold-formed steel-to-steel shear connections. Predictions are validated through an experimental study considering ply configuration and a single washer head fastener. Although, some of these studies focused on the shear behaviour of CFS sheathed walls fasteners. However, more research work on the use of specific steel thickness and sheathing type and thickness, is deemed necessary.

In this paper, a testing rig was developed consisting of two CFS lipped channels facing back-to-back connected on the flanges by OSB and loaded such that the 8 connecting fasteners experience shear. The prevailing role for each sheathing thickness and fastener type is highlighted in the results. The experimental results are summarized for further use in the analysis of shear walls under gravity and lateral load. Subsequently, analytical study to assess the lateral strength and stiffness of full scale sheathed CFS walls have been completed based on the results obtained from the experimental campaign.

2. Experimental tests

2.1. Test setup

Twenty racking tests on built-up back-to-back CFS stud sheathed with OSB have been carried out according to BS EN 594 (1996) [11]. The experiments focused on the sheathing thickness, fastener type and spacing/layout. Figure 1 shows a specimen and the testing rig that has been adopted to apply monotonic concentric compression loading during the test series. This testing rig provides the response of eight stud-fastener sheathing combinations in shear. The direction of shear is parallel to the stud flange, this choice was adopted due to the fact that, in shear walls, the primary shear direction in the chord studs is parallel to the stud flanges.

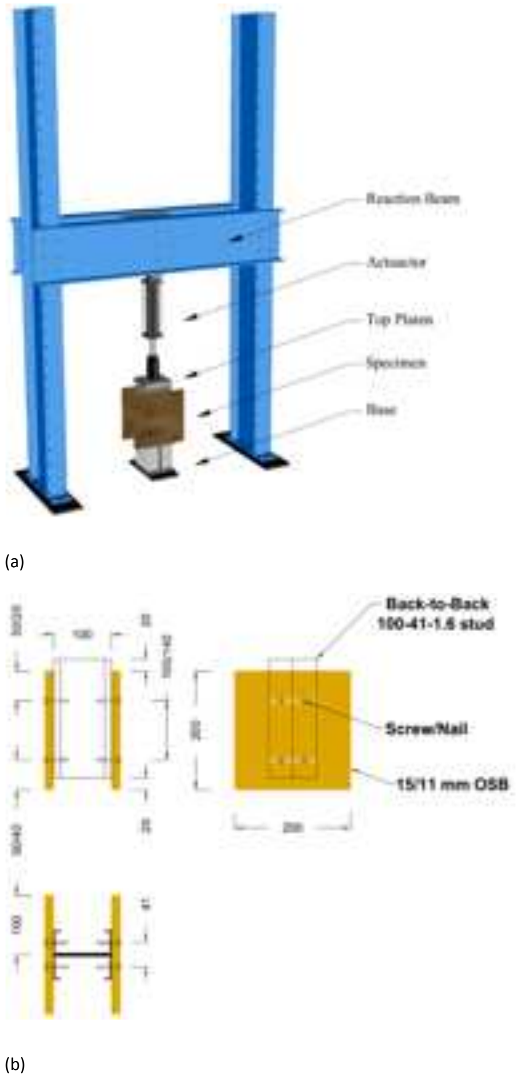


Figure 1 (a) 3D view of specimen in rig and (b) Front view of loaded specimen, dashed lines indicate hidden stud (dimensions in mm)

The specimens were rested on fixed platen supports. One lipped channel size S100-41-1.6 was adopted to build 200 mm long back-to-back studs. The S100-41-12-1.6 has the following out-to-out nominal dimensions: a web depth of 100 mm, a flange width of 41.3 mm, a lip length of 12 mm, and a thickness of 1.60 mm. This is the same stud utilized in a larger experimental campaign on sheathed walls that is being undertaken by the authors. OSB sheathing of 200 x 200 mm width x length having 11 and 15 mm thickness was attached on both sides of the studs and in contact with the stud flanges as shown in Figure 1b. Detailed description of the different test options is provided in Section 2.2.

All tests were displacement-controlled quasi-static loading. The load rate did not exceed 4.00 mm/min. Loading platens were made of low-carbon steel with an appropriate hardness and yield strength as required for the tests; they were installed parallel (+/- 0.05° off the horizontal plane). The dimensions and setup are shown in Figure 1. Measurements of load are made through the load cell on the rig (Figure 1a), and the LVDT measures the applied displacements.

As shown in Figure 1, the back-to-back section is offset from the base by 20 mm and is loaded through a steel plate. The fixings are placed at 50 mm transverse spacing and 100 mm longitudinal spacing (Figure 1b). The applied shear force is therefore resisted by 8 screws. The deflection may be measured as the relative movement between the flanges of the C section and the OSB on both sides of the back-to-back section. Two LVDTs were placed on both web faces of the back-to-back section in order to measure the relative displcament between the built-up stud and OSB.

Loading in the tests was in accordance to the BS EN 594 (1996) [11]. The required full test cycle is shown in Figure 2. The test method was divided into a stiffness test followed by a strength test. Following a racking preload causing a loading of 10% peak capacity (for bedding in test) of the specimen, the stiffness test consists in applying a racking load up to 40% peak capacity of the specimen, and the racking load was then removed. This loading cycle was repeated two times and on the seconde cycle the racking load was increased until failure of the specimen, ensuring that the racking displacement did not exceed 4 mm every 1 minute. Each racking test required about 45-50 minutes to be completed.

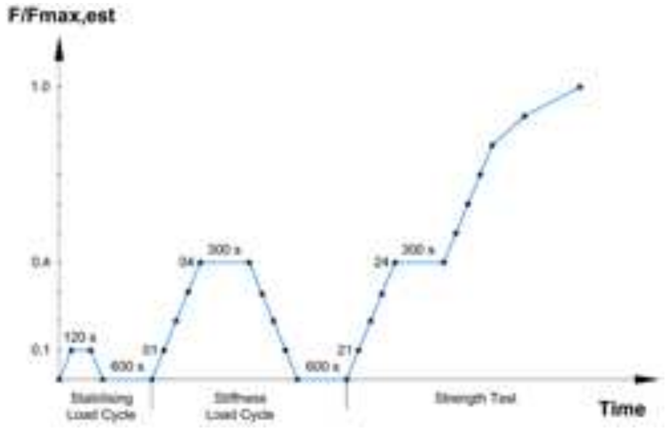


Figure 2 BS EN 594 (1996) loading protocol for racking monotonic tests

From the EN 594 (1996) [11] test procedure, racking strength and stiffness is determined. The racking stiffness (, in kN/mm) is determined from the following equation:

(1)

Where is the racking load from the test load cycle (in kN) and is the racking displacement under (in mm). As for the racking strength, it is represented by the peak capacity of the tested assembly.

2.2. Test parameters

Specimens were configured to represent typical sheathing-to-frame fasteners used in shear wall assemblies that will subsequently be tested by the authors. For this purpose, the tests covered the following parameters: sheathing thickness, fastener type and spacing.

Testing were conducted with fasteners in the field of the OSB as shown in Figure 1 and also repeated with top pair of fasteners (each side) near the top edge of the OSB to reflect the edge distance used in practice. The edge distance can significantly influence the ultimate resistance and stiffness of the assembly. The test parameters are summarized in the test matrix of Table 1. A minimum of three replicates of each test were performed.

Table 1 Test matrix for characterizing fastener behaviour in shear

Test Option	Test ID	Steel thickness (mm)	Loading	Fastener spacing (mm)	Fastener type	OSB thickness (mm)
1	1-4				Screw	15
2	5-8			100		11
3	9-12	1.6	Monotonic			15
4	13-16				Nail	11
5	17-20			140		15

Four specimens of 5 options were prepared for testing as follows:

Test Option 1

200 mm long, 100 mm x 40 mm x 12 mm x 1.6 mm thick back to back specimens of CFS C-section with two pieces of 15 mm thick 200 mm x 200 mm OSB (sheathing board) attached using eight wood-to-steel screws *i.e.*, four for each piece of OSB. The screws were installed 50 mm from the top of the OSB and 30 mm from the bottom of the OSB with a 100 mm space between screws.

Test Option 2

200 mm long, 100 mm x 40 mm x 12 mm x 1.6 mm back to back thick specimens of CFS C-section with two pieces of 11 mm thick 200 mm x 200 mm OSB attached using eight wood-to-steel screws *i.e.*, four for each piece of OSB. The screws were installed 50 mm from the top of the OSB and 30 mm from the bottom of the OSB with a 100 mm space between screws.

Test Option 3

200 mm long, 100 mm x 40 mm x 12 mm x 1.6 mm back to back thick specimens of CFS C-section with two pieces of 15 mm thick 200 mm x 200 mm OSB attached using eight nails *i.e.*, four for each piece of OSB. The nails were installed 50 mm from the top of the OSB and 30 mm from the bottom of the OSB with a 100 mm space between screws.

Test Option 4

200 mm long, 100 mm x 40 mm x 12 mm x 1.6 mm back to back thick specimens of CFS C-section with two pieces of 11 mm thick 200 mm x 200 mm OSB attached using eight nails *i.e.*, four for each piece of OSB. The nails were installed 50 mm from the top of the OSB and 30 mm from the bottom of the OSB with a 100 mm space between nails.

Test Option 5

200 mm long, 100 mm x 40 mm x 12 mm x 1.6 mm back to back thick specimens of CFS C-section with two pieces of 15 mm thick 200 mm x 200 mm OSB attached using eight nails *i.e.*, four for each piece of OSB. The nails were installed 50 mm from the top of the OSB and 30 mm from the bottom of the OSB with a 160 mm space between nails.

3. Material properties

To quantify basic material properties of the CFS used for the test specimens, a series of 32 coupon tests, using CNC milled longitudinal cuts of the webs and flanges for the channel sections and of the webs and lips of the track section, were performed. Figure 3 shows the locations of the coupons. Testing was completed in accordance with BS EN ISO 6892-1 (2016) [12].

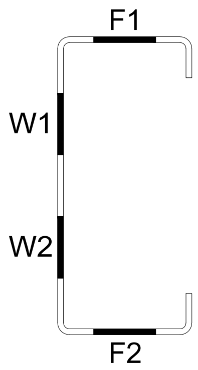


Figure 3 Location of coupon taken from the lipped channel

To remove the zinc coating, both ends of all coupons were put in a 1M HCl solution until the coating was removed; uncoated steel measurements (namely uncoated thickness) could then be made. Figure 4 shows the BS EN ISO-dictated coupon dimensions for steel sheet thicknesses used in the tests herein. Yield (at 0.2% offset) and ultimate tensile strengths for the 100S-41-1.6 section were recorded with a mean of 472.4 MPa and 495.5 MPa, respectively. All yield stress values are considerably above the nominal 450.0 MPa. Young's modulus was not estimated from the linear data in the test results and is assumed to be 203400 MPa as prescribed in EC3 Part 1.3 [13]. Additional material properties testing on the fasteners, or sheathing, was not conducted.

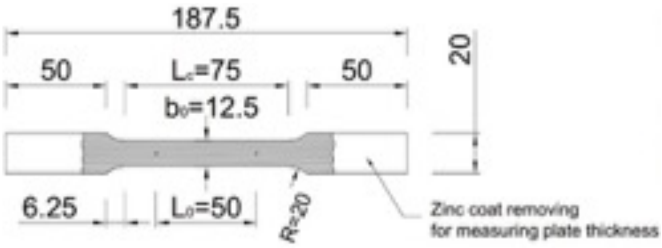


Figure 4 Tensile coupon dimensions in mm

Complete stress-strain curve is plotted in Figure 5, and Table 2 summarizes the basic material properties: yield stress, ultimate stress, and maximum ductility.

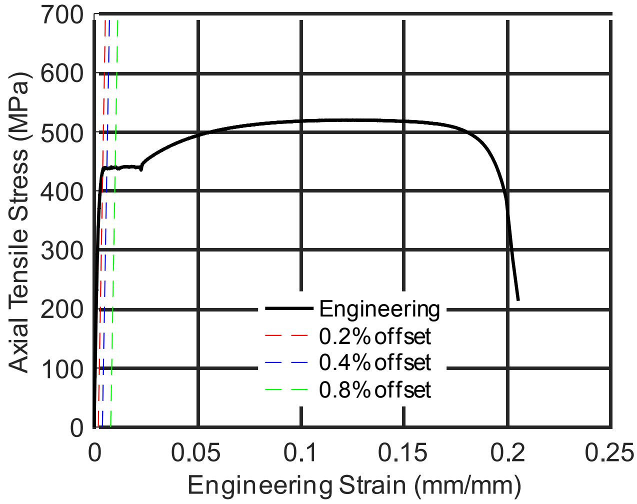


Figure 5 Stress vs. strain of coupons cut from the web of the 100S-41-1.6 section

Table 2 Tensile coupon test results

Specimen	Base metal thickness t (mm)	Gauge length elongation ΔL_g (%)	Yield stress[a] $F_{y,0.2}$ (MPa)	Yield stress[b] $F_{y,\,auto}$ (MPa)	Upper yield stress $F_{y,\,upper}$ (MPa)	Tensile strength F_u (MPa)	Strain at tensile strength ε_u (mm/mm)	Strain at rupture ε_r (mm/mm)
100S-41-1.6- G275-W1	1.55	14.68	492.9	492.8	493.4	516.5	0.0725	>0.12
100S-41-1.6- G275-W2	1.55	9.27	478.4	477.9	478.7	501.2	0.0673	>0.10
100S-41-1.6- G275-F1	1.57	11.06	472.4	471.8	472.5	495.8	0.0641	>0.10
100S-41-1.6- G275-F2	1.57	11.67	445.9	445.6	446.3	468.3	0.0579	>0.09
Mean	1.56		472.4	472.0	472.7	495.5		
STDEV	0.01		19.65	19.70	19.68	20.11		

[a]The 0.2% offset method is used here
[b]The autographic method used was the averaging of the stress levels at the 0.4% and 0.8% offset intercepts

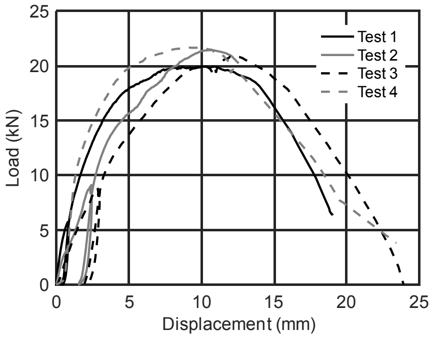
4. Results and discussion

Typical load-deflection results, of four nominally identical specimens for five test options, under monotonic loading are provided in Figure 6. Each plot contains the curves representing the nominally identical specimens that belong to the same test option.

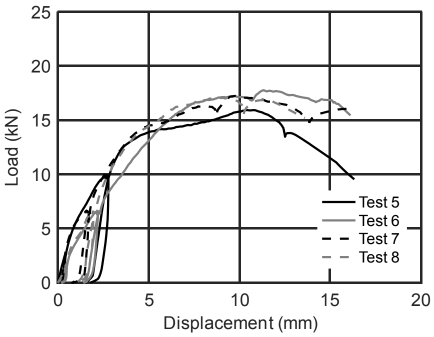
The key parameters i.e., peak racking strength and stiffness at the serviceability condition from all the conducted tests (twenty in total) were determined in accordance with the BS EN 594 (1996) [11] guidance and are provided in Table 3. Conversion of the full test results, on eight fasteners, to single fastener values is carried out by assuming a total peak racking strength of and the peak racking strength of the individual fastener equal to /8. Assuming all deflections occur at the fastener locations implies that the deflection at the fastener, , is determined from the total deformation, as equal to .

Although significant scatter exists in the test results some basic findings are immediately clear: racking strength and stiffness for screws are greater than nails (Options 1-2 vs. options 3-4, respectively). The OSB thickness is strongly correlated with the racking strength and stiffness of the assembly (thicker OSB implying stiffer response). The loaded edge distance is not influential in determining the racking strength and stiffness per nail as witnessed through test results of Options 3 and 5. This was mainly due to the fact that the minimum loaded edge distance (20 mm in Option 5) was enough to prevent breaking of the sheathing edge. The loaded edge distance was not considered as a parameter of investigation for screw connected specimens (i.e., Options 1 and 2). Figure 7 shows the typical failure mode witnessed during all the test options.

From the test results, a comparison of the peak racking strength and racking stiffness at the serviceability condition between identical specimens tested to BS EN 594 (1996) [11] is shown in Figure 8.



(a)



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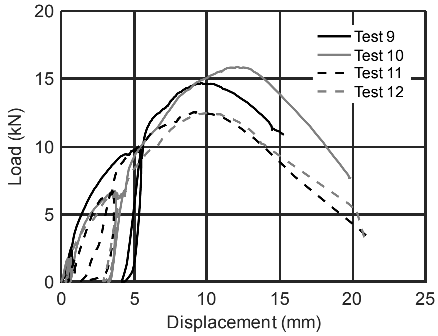
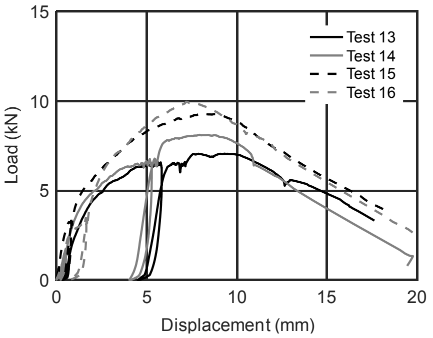


(c)



(d)

(e)

Figure 6 Load vs. vertical displacement curves for: (a) Option 1, (b) Option 2, (c) Option 3, (d) Option 4 and (e) Option 5

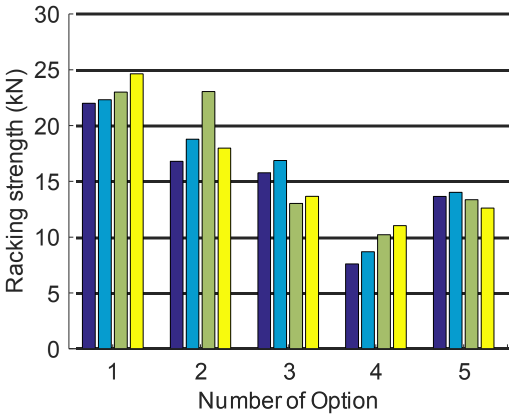
(a)

(b)

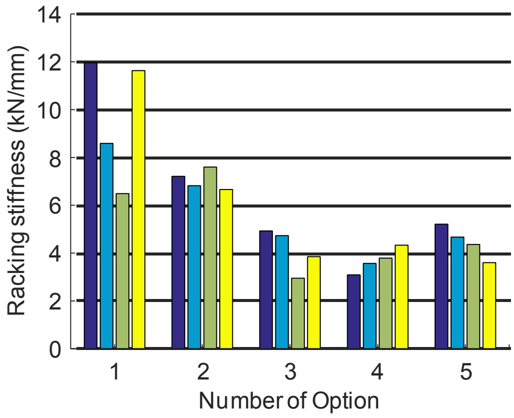
(c)

(d)

Figure 7 Typical sheathing-to-frame fastener failure modes: (a) tilting, (b) pulling-through OSB, (c) OSB detachment from the steel profiles and (d) complete disassembly of the specimen



(a)



(b)

Figure 8 Comparison of peak racking strength (a) and stiffness at the serviceability condition (b) for identical samples of different test options

For strength behaviour, as shown in Figure 8a, the peak racking strength is similar to some extent for the four specimens within each test option with an average variation from the mean value of just 9.43%. On the other hand, for stiffness behaviour, shown in Figure 1b, there is a clear difference between the results of Option 1 specimens where the maximum variation from the mean value is about 23.81%.

5. Analytical assessment of the shear wall lateral strength and stiffness

Steel Construction Institute (SCI) publication ED002 [14] presents design methods for CFS walls with sheathing boards. The test results described in the previous section which enabled the assessment of the fasteners racking stiffness and strength (and , respectively) have been used to calculate the design racking resistance, , of sheathed stud walls by the means of the following equation:

(2)

Where:

g: is given in SCI ED002 [14]

: is the fastener spacing at the edge of the shear wall (assumed constant)

: is the experimentally determined maximum fastening strength

: is the proposed material safety factor when characteristic fastening strengths are used is the width of the panel

: is the width of the shear wall

According to SCI ED002 [14], and having calculated the lateral strength of a sheathed CFS shear wall, the lateral displacement of the wall at the ultimate lateral load level can be conveniently evaluated as:

(3)

Where:

: is given in SCI ED002 [14]

: is the stiffness of the fastener

: is the height of the shear wall

: is the shear modulus of the sheathing board

: is the thickness of the sheathing board

The deflection of the sheathed CFS shear wall panel should be limited to . For further details on the anatical method adopted to assess the strength and stiffness of the sheathed CFS walls, the reader is refered to SCI ED002 [14].

Table 4 shows the results of the assessment of the racking strength and stiffness of wood-sheathed shear walls using the above-described analytical method. The results show that the racking strength and stiffness of the shear wall is very sensitive to the height-to-width aspect ratio where slender shear walls are characterised by relatively low racking strength and stiffness compared to shear walls having lower height-to-width aspect ratio (*i.e.*, stocky walls). In addition, a steady increase in rakcking strength and stiffness is associated with screw spacing reduction.

Table 3 Racking test results carried out on a range of sheathing-to-frame fastener combinations

Test Option	Test ID	Stiffness		Strength			Deflection	Failure mode
		Stiffness (kN/mm)	Mean (kN/mm)	Peak strength (kN)	Mean (kN)	Peak strength per fastener (kN)	Δ (mm)	
1	1	11.96	9.66	22.00	22.99	2.75	9.43	Tilting + pull-through OSB
	2	8.59		22.32		2.79	10.77	
	3	6.49		23.00		2.88	12.14	
	4	11.63		24.64		3.08	8.44	
2	5	7.21	7.07	16.80	19.15	2.10	11.23	
	6	6.82		18.78		2.35	13.53	
	7	7.60		23.05		2.88	10.77	
	8	6.66		17.98		2.25	8.55	
3	9	4.93	4.12	15.77	14.83	1.97	9.08	
	10	4.73		16.87		2.11	11.99	
	11	2.95		13.02		1.63	9.84	
	12	3.85		13.66		1.71	9.22	
4	13	3.09	3.69	7.60	9.39	0.95	9.00	
	14	3.57		8.69		1.09	10.34	
	15	3.79		10.22		1.28	8.96	
	16	4.33		11.03		1.38	7.87	
5	17	5.21	4.46	13.65	13.41	1.71	10.76	
	18	4.67		14.02		1.75	10.42	
	19	4.36		13.36		1.67	11.73	
	20	3.60		12.61		1.58	11.53	

Table 4 Assessment of the racking strength and stiffness of wood-sheathed shear walls

Sheathing type and thickness (mm)	Frame thickness (mm)	Wall size (mm x mm)	Fastener spacing (mm)	Lateral strength (kN)	Lateral stiffness (kN/mm)
		935 x 2550	100	14.51	0.71
		1900 x 2550	100	35.58	2.50
		1690 x 2550	100	28.71	2.00
		2205 x 2550	100	40.87	3.33
		2400 x 2550	100	44.22	3.33
		1665 x 2550	100	28.30	2.00

OSB 15	1.6	720 x 2550	75	14.95	0.56
		1393 x 2550	75	31.77	2.00
		1463 x 2550	75	33.31	2.00
		1835 x 2550	75	41.43	3.33
		2400 x 2550	75	58.96	5.00
		700 x 2550	75	14.54	0.53
		1096 x 2550	75	22.61	1.11
		750 x 2550	75	15.57	0.59

6. Conclusions and future work

In CFS structures, shear wall is one of the primary lateral load resisting systems; it is composed of C-shaped framing members (chord studs, studs and tracks) attached to sheathing using screw/nail fasteners. The behaviour of the connections between the CFS frame and the sheathing gouverns the performance when these walls are subjected to increasing lateral loading (*e.g.*, wind). This structural component should be designed to provide adequate lateral shear strength and stiffness to the global structure. Given the fact that the current version of the Eurocodes does not provide any guidance on the design of this lateral load resisting system, a design method based on testing is deemed necessary to evaluate the peak racking strength and the racking stiffness at the serviceability condition of wood-sheathed CFS shear walls. The main conclusions drawn from this study are listed as follows:

- Overall, the test results are satisfactory expect those of Option 1 where the racking stiffness values are so varied. Therefore, and from a design stand-point, adopting the lowest value for that specific set of tested specimens (*i.e.*, Option 1) could be an alternative and it would lead to a conservative design.

- Moreover, and in order to reduce the above-mentioned variation, additional specimens of Option 1 would shortly be tested.

- Selecting data from tests described in Section 3 in which web-cut coupons were used and the average yield stress was attained as shown in Figure 5. Raw data was recorded as engineering stress and engineering strain, and therefore conversions to true stress and true plastic strain at 20 discrete points from the plotted curves will be determined in order to to be used in detailed numerical finite element modelling.

- From this study, determination of strength and stiffness backbones from the experimental results of steel-to-OSB screw fasteners will be carried out in order to model the screw fastener behaviour in detailed numerical finite element modelling.

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