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LATERAL LOADING RESPONSE OF CFS FRAMED SHEAR WALLS SHEATHED WITH CEMENT BOARD PANELS

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Abstract

The University of Trento has recently been involved in a research project focusing on the development of an innovative industrialised housing system composed of cold-formed steel profiles. This paper describes the laboratory testing phase of the research project comprising the development of lateral design information for cold-formed steel framed walls that are sheathed with cement board panels. A summary is provided of the shear wall test program, as well as the ancillary characterization tests on the sheathing and the sheathing connections, in addition to the results of the application of existing hand calculation methods to determine shear wall resistance to lateral loads and lateral stiffness.

Introduction

The adoption of cold-formed steel (CFS) profiles for residential buildings started in USA. Combining the positive and consolidated experience in the field of timber framed structures with the advantages typical of CFS profiles such as lightness, shapes versatility, ease of assembly, etc., has led to the

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development of competitive industrialised CFS systems also for residential purposes. Positive experiences in this field took place in other countries such as Australia, Canada and Japan. Also in Europe, experience in this regards can be found in Scandinavian countries, the United Kingdom and Romania. In Italy, where traditionally steel in residential buildings has a rather limited application, this technique of construction has not yet found significant use.

The University of Trento has recently been involved in a research project focusing on the development of an innovative industrialised housing system composed of cold-formed steel profiles. In this framework, an extensive experimental programme was planned with the objective of investigating the response of single profiles and of substructures (walls and trusses). The experimental study of walls comprised tests on the bare steel skeleton and on framed walls sheathed with cement board panels. The monotonic and reversed cyclic testing of representative walls subjected to in-plane lateral and vertical load was carried out. Ancillary tests were also performed to provide a better understanding of the walls' response concerning the shear behaviour of the sheathing and of the sheathing connections.

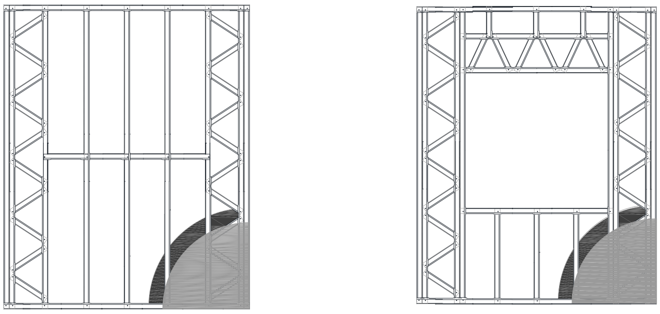
This paper focuses on the study of the sheathed shear walls. The main features of the test set up and test protocols for both the walls and the ancillary tests are described. The test results are hence presented and discussed. The paper at the end summarises the results of the application of existing hand calculation methods to determine shear wall resistance to lateral loads and lateral stiffness.

Test Program

An evaluation of the response of trussed frame systems was initially carried out by means of linear elastic structural analyses. These evaluations, however, were not able to take into account key issues such as the local elastic buckling of the stud members, the stiffness of the riveted connections used to fasten the wall members or the contribution of the sheathing and its connections. Due to the challenge in creating a numerical model that could capture these aspects tests on walls subjected to combined gravity loading and in-plane lateral displacements were planned, as well as tests on both the sheathing and steel member-sheathing connections.

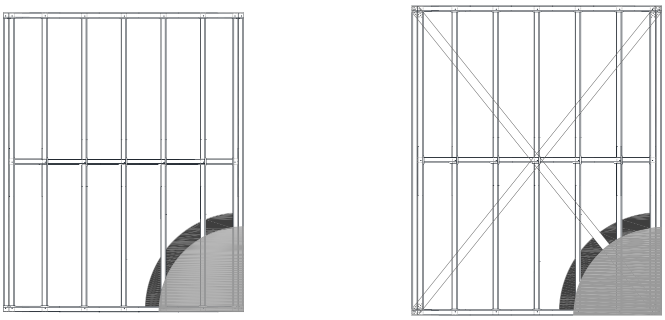
Test Walls, Set-up and Loading Protocols

The wall testing was limited to eight specimens of a single storey in height. The steel framing consisted of configurations with vertical studs, vertical studs with strap bracing and vertical studs with a 400 mm deep vertical truss at each end (including a wall with a window opening) (Figs. 1-2). The screw (4.2 mm x 25 mm self-drilling) connected sheathing included five different types of cement board and one gypsum board (Table 1).



G5 **G7**

Figure 1: Steel framing of the walls with truss members.



G8 **G9**

Figure 2: Steel framing of the walls with and without strap braces.

Table 1: Types of sheathing.

ID	Product	Company	Material	Nominal thickness mm
A	Aquaroc	Gyproc-Saint Gobain	cement board	12.5
B	Rigidur	Gyproc-Saint Gobain	fibreboard which combines gyproc & cellulose fibres	12.5
E	Bluclad	Edilit	cement board reinforced with fibre	10.0
F	Duripanel	Edilit	wood-fibre cement sheet	12.5
G	Powerpanel	Fermacell	cement-bonded panels reinforced with a glass fibre mesh	12.5
H	Fermacell	Fermacell	gypsum fibreboard	12.5

Table 2 reports a summary of the walls' configuration. All shear walls were 2400 mm x 3018 mm in size with 100 mm deep framing members (f_y = 280 MPa (CEN, 2004) & t = 1.2 mm) spaced at 400 mm o/c that were connected using Avedl Monobolt ® 2771 4.8 mm diameter rivets.

Table 2: Braced wall test specimen configurations.

Specimen	Construction information	Sheathing		Loading Protocol
		Side 1	Side 2	
G5 100 400 BB-1	Trussed frame with double outer chords, and hold-downs on external chords	B	B	Monotonic
G5 100 400 BB-2	Trussed frame with double outer chords, and hold-downs on external chords	B	B	Cyclic
G7 100 400 AB-1	Trussed frame with double outer and inner chords, with window opening and hold-downs on external chords	A	B	Monotonic
G8 100 400 EF-1	Trussed frame with double outer chords, and hold-downs on external chords	E	F	Monotonic
G8 100 400 EF-2	Trussed frame with double outer chords, and hold-downs on external chords	E	F	Cyclic
G8 100 400 BB-1	Trussed frame with double outer chords, and hold-downs on external chords	B	B	Monotonic
G9 100 400 GH-1	Trussed frame with double outer chords, and hold	G	H	Monotonic
G9 100 400 GH-2	Trussed frame with double outer chords, and hold	G	H	Cyclic

A specially constructed test set-up for light framed shear wall structures was used to apply a constant gravity load of 16.96 kN/m along the length of the wall, as well as an in-plane lateral displacement at the top of the wall (Fig. 3). The lateral displacements either followed a monotonic protocol (min speed rate 0.5 mm/min - max speed rate 16 mm/min) or a reversed cyclic protocol (min speed rate 0.6 mm/min - max speed rate14.7 mm/min) as per the ECCS testing procedure for structural steel elements under cyclic loads (ECCS, 1986).

Figure 3: Test set-up.

An MTS ± 250 mm actuator with a maximum capacity of 1MN in compression and 0.6 MN in tension was used to apply the lateral displacements while a cantilevered frame was installed above the test walls to apply a gravity load. The lateral force applied to the walls along with the vertical and horizontal displacements of the walls were measured using a HBM Spider 8 data acquisition system.

Response of Walls to Gravity and Lateral Loading

The results in terms of initial secant stiffness (up to $0.4S_{ult}$), ultimate lateral resistance (S_{ult}) and lateral drift at ultimate, as well as the $0.8S_{ult}$ (post-peak) drift are provided in Table 3.

Table 3: Measured response of wall test specimens.

Specimen	Secant Stiffness (kN/m)	Ultimate Resistance (kN)	Drift at Ultimate Resistance (mrad)	Drift at $0.8S_{ult}$ (mrad)
G5 100 400 BB-1	6760	64.20	9.7	18.2
G5 100 400 BB-2	5639	62.72	10.3	-
G7 100 400 AB-1	2864	40.40	19.8	32.2
G8 100 400 EF-1	6044	70.04	17.3	-
G8 100 400 EF-2	5463	66.80	10.8	-
G8 100 400 BB-1	6170	66.48	11.2	19.3
G9 100 400 GH-1	5320	76.92	13.3	-
G9 100 400 GH-2	3824	70.76	18.0	-

Lateral force *vs.* displacement graphs for the monotonic tests are reported in Figs. 4 and 5.

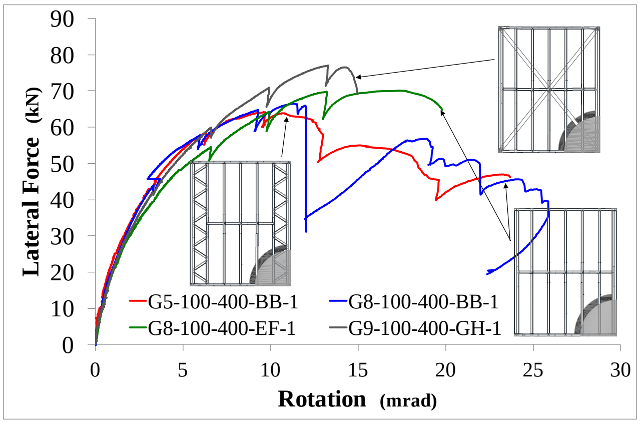


Figure 4: Test results of walls G5, G8 & G9.

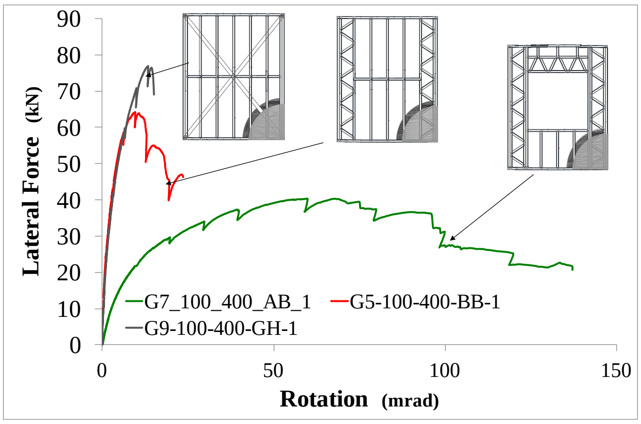


Figure 5: Test results of walls G5, G7 & G9.

The curves in Figure 4 show that the steel bracing system type did not influence in a substantial way the stiffness or the ultimate load capacity of the walls, which were mainly attributed to the cement board sheathing. The adoption of an X -type bracing system, i.e. the solution with the better performance in wall tests without sheathing (Baldassino et al., 2013), along with the installation of cement board sheathing leads to a quite limited increase of the maximum load capacity but to a premature loss in load carrying ability, which was associated with the tension failure of the hold-down anchor rod. Also the complete absence of a steel bracing system for a sheathed wall (Specimen G8) seemed to have minimal effect on the wall's performance, which behaved in agreement with the other tested walls. The weakening of the sheathing due to the introduction of a window opening

(Fig. 5) induced a remarkable reduction of both the stiffness and ultimate shear capacity, but at the same time resulted in a substantial increase of the displacement at collapse. The different response is largely related to the high aspect ratio (height/width) of the remaining full-height sheathing sections on the wall adjacent to the window opening, which resulted in flexural behaviour dominating the wall's response to lateral loading instead of shear.
The failure of the specimens was caused mainly by the degradation in resistance of the sheathing-to-stud screw connections, the rivets connecting the steel frame members, the screws between studs and hold-downs and finally of the hold-down anchor rods (Fig. 6). Local deformation of the studs and crack patterns of the sheathing panels were also observed (Fig. 7).

Figure 6: Typical connection failures.

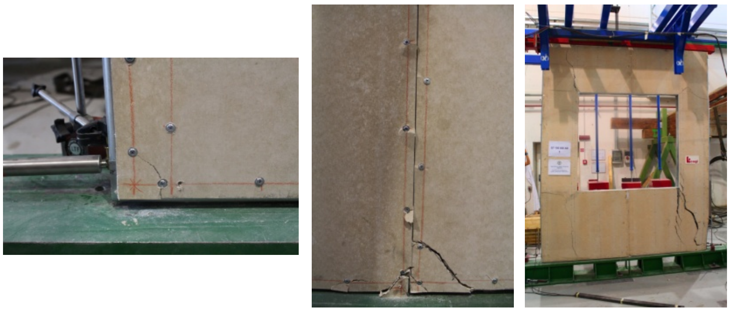


Figure 7: Sheathing cracking patterns.

Very similar responses were also noted in the cyclic and monotonic tests, if the same shear wall type is considered. The same failure modes and a similar lateral force- displacement curve, using the outer envelope of the reversed cyclic curve, were in fact observed. As an example, in Figure 8 the lateral load-displacement curves for the monotonic test (dotted line) and the reversed cyclic test (continuous line) for the wall type G8 are compared. In both tests at ultimate conditions, the uplift of the bottom chord which indicates the failure of the hold-down anchor rods, the collapse of the screws between studs and hold-downs and of the connections between sheathing-to-stud screw connections were observed (Fig. 9).

As a preliminary comment regarding the reversed cyclic test results it can be observed that the ECCS tests procedure adopted for these tests seemed not to affect the wall response. Further analyses of these data are planned including an evaluation of the ductility properties of the walls.

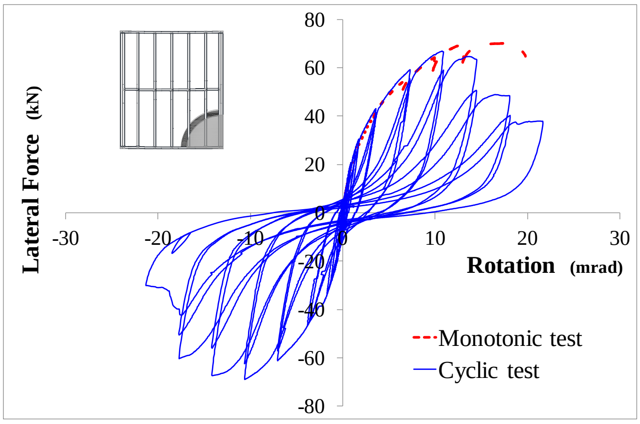


Figure 8: Test results of walls G8 (monotonic and cyclic tests).

Figure 9: Failure mode for wall G8.

Figure 10 allows for the identification of the influence of the cement board sheathing on the wall's performance through a comparison of the response of a wall with X-type bracing system tested with and without sheathing. The wall with sheathing demonstrated an increase in terms of the maximum shear resistance and stiffness of 125% and 114%, respectively, and a reduction of the ultimate deformation of 67%.

The tests on the walls described in this section clearly illustrate the key role played by both the sheathing and the sheathing-to-frame connections on the overall walls' response. Failure of the sheathing and of the connections between the studs and the sheathing was in fact observed in all the tests. For a reliable evaluation of the walls' response a better understanding of the shear response of the sheathing products and their connections was needed. Data usually provided by the producers of the sheathing used for this test programme are limited to the modulus of elasticity, the bending resistance and the fire resistance. In an attempt to eliminate this 'gap' in knowledge, shear tests on the sheathing and on the connections between the sheathing and steel profiles were hence completed.

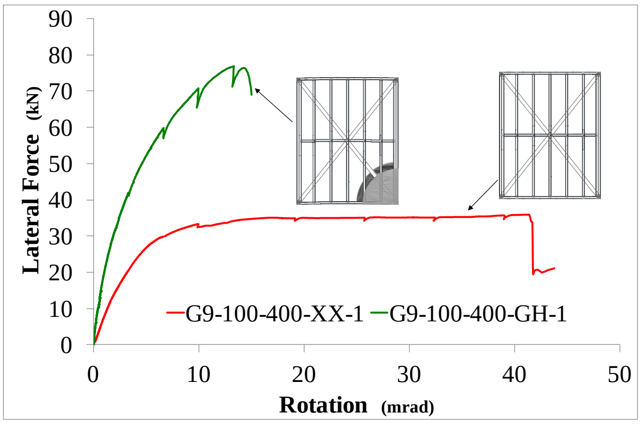


Figure 10: Test results of X strap braced wall with and without cement board sheathing.

Edgewise shear tests on sheathings: Set-up, Loading Protocols and test results

Tests were performed in agreement with the provisions of ASTM D1037-06 (2006) for the case of 'Edgewise shear test'. Specimens with nominal dimensions of 90x250mm were taken from the sheathing panels and loaded in edgewise shear (Fig. 11). The load was applied with a universal loading machine Galdabini (model PM10, maximum capacity of 100kN, class 0,5 in accordance with the UNI EN ISO 7500). Tests were performed under displacement control with a speed of 0.05 mm/s. During the tests, load and the shear displacement were recorded.

For each sheathing type at least four tests were performed. Additional tests were carried out when a scatter of results greater than 10% was observed. The total number of tests was of 27. A typical failure mode is presented in Figure 11.

The test data were analyzed so as to determine the shear modulus G and the shear stress at ultimate τ. The following equation was adopted to determine G:

$$G = \frac{\tau_{40\%}}{\gamma_{40\%}} \qquad [1]$$

where

$\tau_{40\%}$ shear stress associated with a load of 40% of the maximum load;

$\gamma_{40\%}$ shear deformation associated with $\tau_{40\%}$.

The shear stress (τ) was evaluated as:

$$\tau = \frac{P}{t * L} \qquad [2]$$

where
P applied load ($P=P_{max}$ or $P=P_{40\%}$ depending on the τ value required)
t thickness of sheathing;
L length of the specimen.

Figure 11: Test set-up for edgewise shear test and failure mode.

The test results for all the sheathing types are reported in Table 4 in terms of average values of both G and τ_{max}.

Table 4: Edgewise shear test on sheathing.

Sheathing type	n. tests	G N/mm^2	τ_{max} N/mm^2
A	4	2932	2.96
B	4	1554	3.97
E	4	1827	7.71
F	5	1645	6.00
G	6	1591	2.96
H	4	1319	3.87

Shear tests on the stud-sheathing connections: Set-up, Loading Protocols and Tests Results

These connection tests were performed following the procedures found in ASTM D 1761-88 (1988), which applies to the test methods used for the evaluation of the mechanical properties of fasteners in wood. The procedures were hence adapted for use for this specific case study. Each specimen was composed of three stud profiles, two of them were coupled and located at the base of the specimen, while the third was at the top (Fig.

12). The studs were connected to the sheathing by means of the screws adopted in the test walls. In order to induce the failure of the screw connections at the top of the specimen an increased number of fasteners was installed at the bottom.

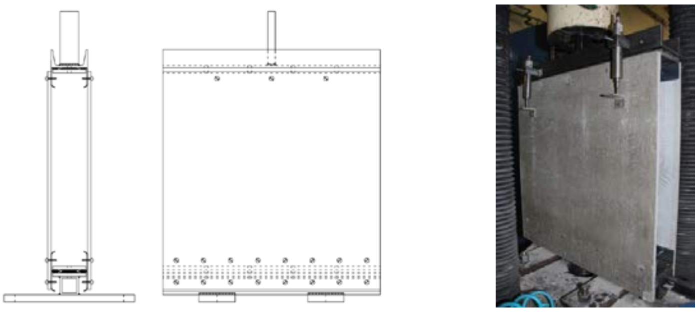


Figure 12: Specimen and test set-up for shear tests on the connections.

The specimens were tested under pure tension, which allowed for failure of the upper stud-to-sheathing connection (Fig. 12). Tests were performed under displacement control (speed of 5mm/min) with an universal loading machine Galdabini (model PM10, maximum capacity of 100kN, class 0.5 in accordance with the UNI EN ISO 7500). The displacements of the top of the specimen were measured in two positions and on both the sides of the specimens (Fig. 12). Both the load and the displacements were recorded during the tests.
A minimum of 4 tests were performed for each type of sheathing; but the wide scatter of results required additional tests for all the sheathing types except one (sheathing type B). An evaluation of both the stiffness and resistance of the connections was then carried out. Table 5 summarizes the results of the tests in terms of mean value of the secant stiffness evaluated at 40% of the maximum load ($k_{40\%}$) and of the maximum load (F_u). For an appraisal of the scatter of results also the maximum and minimum values of both the secant stiffness and maximum load are reported. In Table 5, F_u refers to the maximum load associated with the upper connection which is composed of six screws.

Table 5: Shear test results on the sheathing-to-frame connections.

Sheathing type	n. tests	$k_{40\%}$ N/mm	$k_{40\%,min}$ N/mm	$k_{40\%,max}$ N/mm	F_u N	$F_{u,min}$ N	$F_{u,max}$ N
A	8	2575	732	4112	5266	4280	6520
B	4	2848	2603	3052	7530	7350	7710
E	6	2370	1055	4239	8760	7130	10400
F	7	1410	818	1963	7935	6080	9275
G	6	1928	1152	2728	4671	3400	5900
H	6	2054	1432	2564	6503	4820	7140

Resistance and lateral displacement by hand calculations

Since the 1970s in the United States, and in the following years in other countries, extensive experimental campaigns were performed aimed at investigating the response of framed shear wall systems with and without sheathing under vertical and lateral loads. Tests allowed for an in-depth understanding of the complex mechanisms which govern the wall's response and to identify the key parameters affecting their behaviour.
The wide-ranging experimental studies of wood shear walls allowed researchers to indentify simplified relationships between the sheathing connections and the overall shear wall resistance to lateral loads and lateral stiffness. In the literature various formulations are available which mainly focus on the wall's response in the elastic range. In these formulations parameters such as dimensions and properties of the sheathing, number and position of the sheathing-wall frame connections, stiffness and resistance of the bare connection are considered while the contribution of a framed support is completely disregarded.
An attempt to apply these existing formulations proposed for wood shear walls to the cases considered in this study was done. Experimental results of the walls presented in this paper clearly show the preeminent influence of the sheathing on the overall wall response. The contribution of the bare steel skeleton was hence assumed negligible and the wall performance attributed to the sheathing and its connections.
The following formulations, used to determine the lateral resistance of shear walls, were adopted; Easley et al. (1982), Tuomi & McCutcheon (1977) and Kallsner & Lam (1995). For the lateral displacements the formulations of Easley et al., Kallsner & Lam (elastic formulation) and McCutcheon (1985) were considered.
In the calculations the results of the sheathing shear tests and the sheathing-to-frame connections presented in this paper were used. The results of the calculations are presented in Tables 6-9 for walls G5, G8 (in the two tested configurations) and G9, respectively. In the tables R_{mod} and e_{mod} identify the wall lateral resistance and the related displacement evaluated by hand calculations. For an appraisal of the reliability of the considered methods, in R_{mod} and e_{mod} are compared to F_y and $e_{y,}$ which identify conventional elastic limits for the lateral resistance and the related displacement evaluated following the ECCS procedure.
The results presented in the tables showed quite good agreement between the experimental and the hand method results if the lateral resistance were considered. In particular the method proposed by Easley et al. (1982) leads to a general underestimation of the lateral resistance which can reach a maximum of 36% for the wall type G9, i.e. the wall with X-type bracing systems. In contrast, the Tuomi & McCutcheon (1977) and Kallsner & Lam (1995) methods provide a general overestimation of the lateral resistance for all the cases with the exception of wall G9. As to the displacements it can be

noted that all the considered methods, provide a substantial overestimation which ranges from 94% to357%.

Table 6: Comparison between hand method calculations and experimental results for wall G5 100 400 BB.

Hand method	R_{mod} kN	R_{mod}/F_y	e_{mod} mm	e_{mod}/e_y
Easley et al.	35.94	0.83	10.74	2.78
Tuomi & McCutcheon	46.39	1.08	-	-
Kallsner & Lam (elastic method)	47.30	1.10	13.93	3.61
McCutcheon	-	-	17.63	4.57

Table 7: Comparison between hand method calculations and experimental results for wall G8 100 400 BB.

Hand method	R_{mod} kN	R_{mod}/F_y	e_{mod} mm	e_{mod}/e_y
Easley et al.	35.94	0.76	10.74	1.94
Tuomi & McCutcheon	46.39	0.98	-	-
Kallsner & Lam (elastic method)	47.30	1.00	13.93	2.51
McCutcheon	-	-	17.63	3.18

Table 8: Comparison between hand method calculations and experimental results for wall G8 100 400 EF.

Hand method	R_{mod} kN	R_{mod}/F_y	e_{mod} mm	e_{mod}/e_y
Easley et al.	37.97	0.85	13.72	2.54
Tuomi & McCutcheon	52.76	1.19	-	-
Kallsner & Lam (elastic method)	52.33	1.18	19.12	3.53
McCutcheon	-	-	24.29	4.49

Table 9: Comparison between hand method calculations and experimental results for wall G9 100 400 GH.

Hand method	R_{mod} kN	R_{mod}/F_y	e_{mod} mm	e_{mod}/e_y
Easley et al.	30.96	0.64	11.71	2.17
Tuomi & McCutcheon	35.27	0.73	-	-
Kallsner & Lam (elastic method)	34.98	0.73	11.85	2.19
McCutcheon	-	-	15.04	2.78

Conclusions

Eight CFS trussed wall test specimens with sheathing were tested under combined gravity and lateral loading. The specimens were characterised by a different steel skeleton, i.e. different bracing system, and by the sheathing. Six types of commercial sheathing were considered in the study. Monotonic and reversed cyclic tests were performed following the ECCS procedure. Tests results showed that the in-plane lateral load response of the walls is influenced by the sheathing and by the sheathing-to-frame connection to a greater extent than by the steel bracing systems. Besides, the bracing system type, when cement board sheathing was also installed, did not affect substantially the measured stiffness or the ultimate load capacity of the wall. The key role of sheathing and its connections was also evident by the failure modes observed in the tests, which typically involved failures of the sheathing-to-frame connections associated with a remarkable cracks pattern of the sheathing. This observation highlighted the need of an adequate and reliable characterisation under shear of both the sheathing and its connections. Shear tests on the sheathing and the sheathing-to-frame connection were hence performed. The results of the tests were summarized. The final part of the paper is devoted to the results of the application of existing hand calculation methods to determine shear wall resistance to lateral loads and lateral stiffness. Various methods originally developed for wood shear walls were adapted and applied to the cases considered in this study. The results in terms of lateral resistance and related displacement have been compared with the experimental results. It was observed that hand methods led to a substantial overestimation of the lateral displacement. As to the lateral resistance, the methods are in a reasonably good agreement with test results. However, the limited number of cases prevents the drawing of any general conclusion.

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Appendix - Notation

S_{ult}	ultimate resistance;
G s	shear modulus;
τ	shear stress;
τ_{max}	maximum shear stress;
$\tau_{40\%}$	shear stress associated with a load of 40% of the maximum load;
$\gamma_{40\%}$	shear deformation associated with $\tau_{40\%}$.
P	applied load;
t	thickness of sheathing;
L	length of the specimen (edgewise shear test on sheathings);
$k_{40\%}$	secant stiffness evaluated at 40% of the maximum;
$k_{40\%,min}$	minimum value of the secant stiffness $k_{40\%}$;
$k_{40\%,max}$	maximum value of the secant stiffness $k_{40\%}$;
F_u	maximum load;
$F_{u,min}$	minimum value of F_u;
$F_{u, max}$	maximum value of F_u;
R_{mod}	value of the lateral force at the end of the elastic range (hand method calculations);
e_{mod}	value of the lateral displacement at the end of the elastic range. (hand method calculations);
F_y	conventional value of the lateral force at the end of the elastic range (ECCS procedure);
e_y	conventional value of the lateral displacement at the end of the elastic range (ECCS procedure).