RESPONSE OF CFS SHEATHED SHEAR WALLS

Matteo Accorti*; Nadia Baldassino*; Riccardo Zandonini*, Federica Scavazza**, Colin A. Rogers***

* Department of Civil, Environmental and Mechanical Engineering, University of Trento, Italy
matteo.accorti@libero.it, nadia.baldassino@unitn.it, riccardo.zandonini@unitn.it

** Cogi s.r.l., Calliano, Italy
fedescavazza@yahoo.it

*** Department of Civil Engineering and Applied Mechanics, McGill University, Canada
colin.rogers@mcgill.ca

Keywords: Cold Formed Steel; Trussed Wall; Diagonal Bracing; Sheathing Wall; Ductility; Energy Dissipation, Analytical design methods.

Abstract. The adoption of cold-formed steel (CFS) profiles for residential buildings started in USA. Positive experiences in this field took place in other countries such as Australia, Canada and Japan. In Italy, where traditionally steel in residential buildings has a rather limited application, this type of construction has not yet found a 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, a study of the response of CFS shear walls was performed. The experimental part of the study comprised tests on the bare steel skeleton and on framed walls sheathed with cement board panels. Reversed cyclic testing of representative walls subjected to in-plane lateral and vertical loads was carried out. The main features of the experimental study and the main results are presented and discussed. The performances of the different typologies of shear walls are compared in terms of resistance, stiffness, ductility parameters and energy dissipation capacity. The paper in its final part, focuses on the results of a study of the effectiveness of analytical methods available in literature and developed for wooden shear panels, for a reliable evaluation of the response of CFS shear walls.

1 INTRODUCTION

The use of cold-formed steel (CFS) profiles in housing systems has recently grown. The combination of the positive and consolidated experiences in wood framing systems with the advantages of cold-formed steel profiles such as lightweight, high structural efficiency, durability, rapidity and simplicity of installation of the building equipment gave rise to the development of new building systems which have shown to be competitive with respect to the more traditional constructional systems. These systems have been adopted since many years in countries such as USA, Australia, Canada and Japan. In Europe, applications of these structural systems were made in Scandinavian countries, United Kingdom and Romania. In Italy, where the use of traditional steel structural systems for residential purposes is quite limited, CFS structural systems are not widespread.

The economic crisis of recent years and the increased competitiveness of the market provided the fertile environment for a new interest by companies in the process of innovating and diversifying their production. An Italian company recently decided to expand its production with the development of a CFS industrialized building system. The structural system is composed with shear walls and flooring systems. The shear walls, which transfer to the foundations the vertical loads of the flooring systems and the horizontal forces due to wind and earthquake, are built-up with studs located at regular intervals and bottom and top chords. An additional chord is located at mid-height reducing the slenderness of the studs. The task to transfer the horizontal forces is entrusted to bracing systems. Two solutions are adopted: steel strap diagonal cross bracings and trussed bracings. Walls are hence completed with sheathings which can be realized with different materials.

In the framework of the activities associated with the development of the building system, the University of Trento was involved in both the experimental and the numerical studies aiming at the structural characterization of the single sections and of 2-D and 3-D subassemblies (e.g., shear walls and trussed floor systems).

This paper summarizes the outcomes of the experimental investigation focusing on the shear walls. Several configurations were tested, comprising walls with vertical studs and strap bracing, walls with vertical studs and vertical trusses at each end, walls with a trussed frame bracing in presence of a window opening and, finally, walls with vertical studs only. The possible influence of the sheathing to the wall’s response was also considered. This paper presents the key features of the experimental study and discusses the main results. The performances of the different configurations of shear walls are compared in terms of resistance, stiffness, ductility parameters and energy dissipation capacity. The last part of the paper focuses on the reliability of analytical methods developed for wooded shear walls, as simplified tools of analyses for the evaluation of the CFS walls response in terms of strength and lateral displacement.

2 LITERATURE REVIEW

As to the knowledge of the Authors, the number of studies of the response of braced CFS walls without sheathing is quite limited. Sheathed shear walls are generally investigated and cases without sheathing are considered for comparison with the related sheathed solutions. In all these studies the non-negligible contribution of the sheathing system to the wall's stiffness and resistance was demonstrated. Most of the studies on bare steel walls deal with the case of wall panels with diagonal straps. As a general outcome, the potential ductility of these systems, associated with high pinching, was clearly pointed out [1], [2]. The importance of a correct detailing of the straps’ joints as a critical design factor, and the need of applying the capacity design method was also stressed [2], [3], [4]. Other parameters studied were the aspect ratio of the panel [5], the type of strap’s connection, if screwed or welded [6], [7], [8], the one-sided or two-sided bracing [9], the use of coupled end studs [10], [11] and the pre-tensioning of the straps [1].

Shaking table analyses were performed by Barton [3] and Kim et al. [12], while numerical nonlinear dynamic analyses were carried out by Comeau et al. [13]. The latter study showed that the use of AISI Specification S213 [14] leads to an adequate level of safety with respect to collapse, at least for low and medium seismicity zones. The general adequacy of these specifications was further confirmed by Velchev et al. [5] and Macillo et al. [15]. Contributions to the determination of a behaviour factor were given by Comeau et al. [13], but no definitive result is still achieved.

Baran and Alica [16], Zeynalian and Ronagh [17] recently contributed to the understanding of the performance of trussed braced panels; the latter study considered a K bracing. Stud buckling and failure of the riveted joints appear to be the criticalities to be accounted for in design. Therefore, the use is suggested only in low seismicity areas.

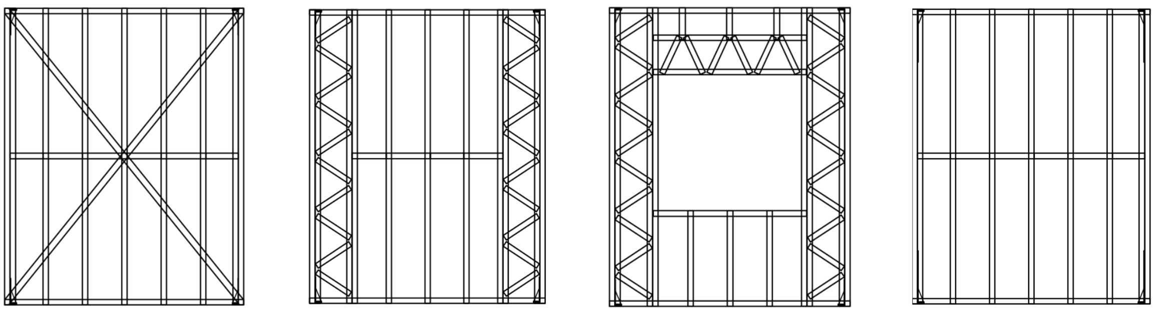
In case of sheathed solutions the need of an adequate design of the connection between the sheathing and the steel framing was also pointed out [18].

At the end of this overview, it should be underlined that different issues are not yet studied at the right depth. Among other facets, the inadequate knowledge of the trussed bracing solution is apparent.

3 SHEAR WALL TEST PROGRAM

The experimental study of the response of CFS shear walls under vertical and lateral loads comprises 21 shear walls specimens of a single storey in height with dimensions of 2400mm x 3018mm. The first part of the study focussed on the influence of several geometrical and structural parameters [19], [20] on the response of bare framing. The present paper reports of six shear tests. For these walls, the steel framing elements (e.g., chords and studs) were built up using the same C-like cold-formed section with a height of 100mm, a width of 50mm and a thickness of 1,2mm. The steel framing systems consisted of configurations with vertical studs with strap bracing (Fig. 1a) and vertical studs with a 400mm deep vertical truss at each end (Fig. 1b) including a wall with a trussed frame bracing to incorporate a window opening (Fig. 1c). The case of a wall with vertical studs without bracing system was also considered (Fig. 1d). The straps of the wall configurations of Figure 1a were made of a steel plate 85mm in width and 1,2mm in thickness.

All the steel framing elements, including the strap bracings, were made of steel with a nominal yielding stress of 280 MPa. Tensile tests performed according to the EN ISO 6892-1:2009 [21] allowed evaluating the actual steel properties. The average upper and lower yielding strength and the ultimate strength were of 290,2MPa, 286,7MPa, and 380,9MPa, respectively, while the steel elongation at fracture was of 31,9%.



a) b) c) d)

Figure 1: Steel framing systems investigated.

For framing systems in Figure 1a) and 1b) two different configurations were tested for comparison: the first with sheathing and the second without sheathing. Configuration 1c) was tested only in un-sheathed condition. Finally, configuration 1d) was tested only in a sheathed condition, due to the lack of any bracing in the steel framing. Four different types of cement board and one gypsum board were considered in the study (Table 1).

Table 2 summarises the wall configurations considered in the study.

ID	Product	Company	Material	Nominal thickness mm
B	RIGIDUR H	Gyproc Saint-Gobain	Fibreboard which combines gyproc & cellulose fibres	12,5
E	BLUCLAD	Edilit	Cement board reinforced with fibre	10,0
F	DURIPANEL S3 (B1)	Edilit	Wood-fibre cement sheet	12,5
G	POWERPANEL HD	Fermacell	Cement-bonded panels reinforced with a glass fibre mesh	12,5
H	GESSOFIBRA	Fermacell	Gypsum fibreboard	12,5

Table 1: Types of sheathings.

Specimen	Construction information	Sheathing Side 1	Sheathing Side 2
G6 100 400 XX	Trussed frame with double inner & outer chords and hold-downs on outer chords	-	-
G7 100 400 XX	Trussed frame with window opening, double inner & outer chords and hold-downs on outer chords	-	-
G9 100 400 XX	Diagonal bracing with double outer chords and hold-downs on outer chords	-	-
G5 100 400 BB	Trussed frame with double outer chords, and hold-downs on outer chords	B	B
G8 100 400 EF	No bracing frame with double outer chords, and hold-downs on outer chords	E	F
G9 100 400 GH	Trussed frame with double outer chords, and hold on outer chords	G	H

Table 2: Wall test specimen configurations.

In all the walls, the steel framing elements were connected using Avdel Monobolt ® 2771 6,4 mm diameter rivets, while self-drilling screws 6,3mm x 25mm with a spacing of 300mm were used to connect the double chords. Self-drilling screws 4,2 mm x 25 mm were adopted to connect the sheathing to the framing elements and the diagonal bracing to the studs.

In order to check the influence of screws spacing, in walls G5 100 400 BB and G8 100 400 EF the spacing of the self-drilling screws of the sheathing to the framing elements connection was of 150mm and 300mm, on the external and internal studs, respectively. In wall G9 100 400 GH the spacing was of 200mm and 400mm, as to the external and the internal studs.

In walls G9 (i.e, walls G9 100 400 XX and G9 100 400 GH) the strap bracings were connected to the frame by means of screws 6,3mm x 25mm, the pattern of which is illustrated in Figure 2.

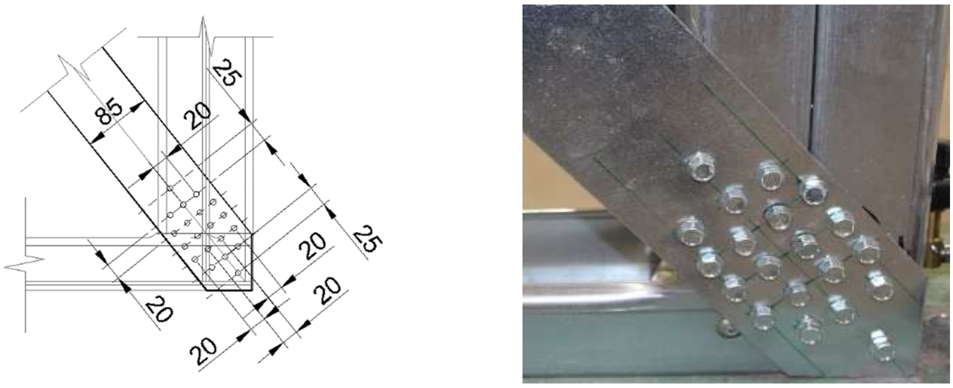


Figure 2: Detail of the strap bracings to framing elements connection (measure in mm).

As to the hold-downs, Rothoblaas® WHT with a height of 265mm hold-downs were used in most of the specimens tested in the study [19], [20]. The capacity design suggested that these hold-downs were not adequate in case of walls with greater shear capacity. The Rothoblaas® WHT hold-downs with height 540mm were hence used in these cases. Table 3 relates the hold-downs and the specimens for the series of tests here presented.

Specimen	Hold-down type
G6 100 400 XX	Rothoblaas® WHT h=265
G7 100 400 XX	Rothoblaas® WHT h=265
G9 100 400 XX	Rothoblaas® WHT h=540
G5 100 400 BB	Rothoblaas® WHT h=265
G8 100 400 EF	Rothoblaas® WHT h=540
G9 100 400 GH	Rothoblaas® WHT h=540

Table 3: The hold-downs.

In order to characterize the sheathing material and the connection between the sheathing and the steel skeleton complementary tests were also carried out (§7.1 and 7.2).

4 TEST SET-UP, INSTRUMENTATION AND LOADING PROTOCOLS

The performance of the shear walls to lateral loads was investigated by means of a testing set-up 'ad hoc' designed for light framed structures. The testing system allows applying both vertical and lateral loads and to perform tests both in monotonic and cyclic regime. The maximum horizontal displacement allowed by the system for the walls' geometry investigated is of 200mm and ±100mm for the monotonic and cyclic tests, respectively. In few tests, this limitation did not allow to reach the maximum deformation capacity of the specimen.

In order to investigate the walls' response in conditions close to the operational ones, the tests were performed by applying a vertical load of 17,07 kN/m, which represents the factored load on the lower wall of a two storey building. At this aim a lever system was adopted and a cantilevered frame installed above the test walls distributed the load along the length of the wall. At the base, the specimens were connected to a rigid counter-beam by means of M12 bolts at 320mm. During the tests the out of plane displacements of the specimen were prevented.

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. A load cell (linearity 0,15%, hysteresis 0,2%) in line with the actuator's head enabled measurement of the lateral force applied to the wall. The vertical and horizontal displacements of the wall were measured using linear transducers (LDT) and a wire transducer (WDS), as shown in Figure 3. A data acquisition system HBM Spider 8 allowed the data logging at 3 Hz sampling frequency.

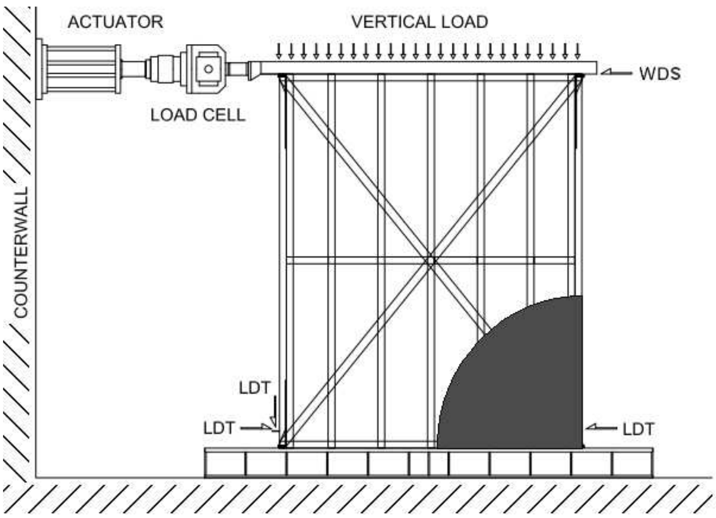


Figure 3: The testing set-up.

The load history of the cyclic tests was defined following the ECCS protocol [22], which consists of the following phases:

1- execution of monotonic test and evaluation of the force (F_{y_conv}) and of the displacement (e_{y_conv}) associated to the conventional yielding. At this aim, the load-top lateral displacement curve is considered. Let E_t be the slope of the tangent at the origin of the load-displacement curve, F_{y_conv} and e_{y_conv} are the coordinates of the intersection point between this tangent and the tangent to the load-displacement curve having a slope of $E_t/10$ (Fig. 4);

2- execution of cyclic test following a loading history which consists of fully reversal cycles with amplitudes defined as a function of e_{y_conv} (Fig. 5).

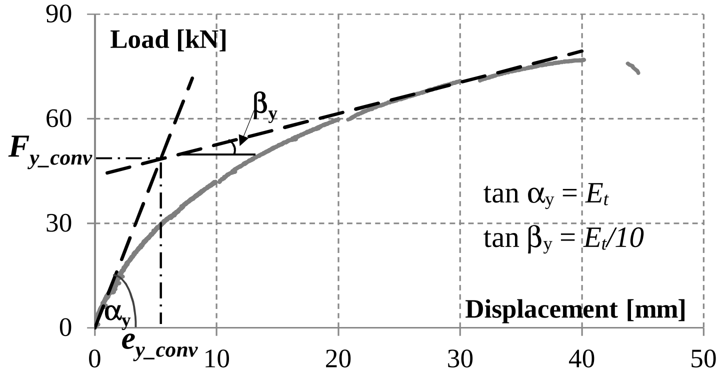
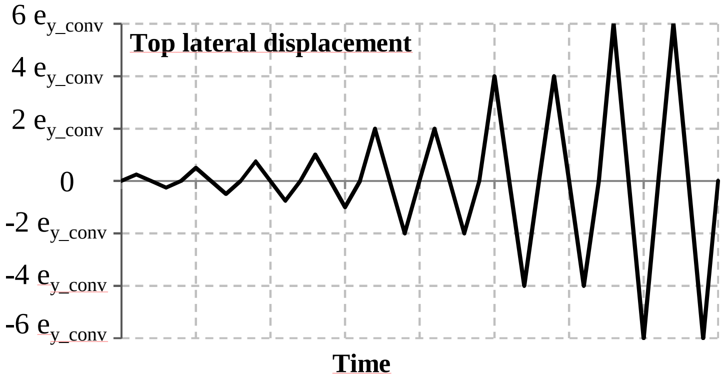


Figure 4: Definition of the conventional yielding force and displacement.



N. of cycles	Amplitude
1	$\pm 1/4\ e_{y\_conv}$
1	$\pm 2/4\ e_{y\_conv}$
1	$\pm 3/4\ e_{y\_conv}$
1	$\pm\ e_{y\_conv}$
2	$\pm\ e_{y\_conv}$
2	$\pm\ (2+2n)e_{y\_conv}$

Figure 5: Loading history for cyclic loading tests.

For each shear wall typology, except for wall G7 100 400 XX, two tests were hence performed: a test following a monotonic protocol and a subsequent reversed cyclic test. The wall G7 100

400 XX has been tested only in cyclic regime. The cyclic loading history has been based on the results of the wall G6 100 400 XX.

The values of e_{y_conv} adopted for the definition of the loading history of the cyclic tests are summarised in Table 4. In the table, the values of E_t are also reported.

Specimen	E_t kN/mm	e_{y_conv} mm
G6 100 400 XX	0,47	18,42
G9 100 400 XX	2,88	9,62
G5 100 400 BB	11,09	3,86
G8 100 400 EF	8,23	5,41
G9 100 400 GH	8,97	5,40

Table 4: E_t and e_{y_conv} values from the monotonic tests.

5 TEST RESULTS

The failure of the bare steel specimens was caused mainly by localised buckling failures that occurred in the stud and chord elements of the vertical truss members (Fig. 6a). In wall G7 100 400 XX-1, where the reinforcement of the chords was adopted, the local buckling/distortion of the end of the chords associated with rivet connection failure by pull-out (Fig. 6b) caused the collapse. In strap bracings walls the local deformation of the diagonal-to-chords connection region, instability phenomenon of the compressed stud and, in the monotonic test, the collapse of the strap connection (Fig. 6c), were observed.

a) b) c)

Figure 6: Examples of collapse modes for walls without sheathing.

The presence of the sheathing significantly modifies the wall's response and the mode of collapse. Failure was associated with the degradation in resistance of: the sheathing-to-stud screw connections, the rivets connecting the steel frame members (Fig. 7a), the screws between studs and hold-downs and, finally, the hold-down anchor rods (Fig. 7b). Local deformation of the studs and crack patterns of the sheathing panels (Fig. 7c) were also observed.

a) b) c)

Figure 7: Examples of collapse modes for walls with sheathing.

The results in terms of initial secant stiffness (at a load equal to 0,4F_{ult}), ultimate lateral resistance (F_{ult}) as well as the lateral drift at ultimate are provided in Tables 5 and 6 which are related to the un-sheathed and sheathed solutions, respectively.

Specimen	Loading protocol	Positive Load			Negative load		
		Secant Stiffness kN/m	Ultimate Resistance F_{ult} kN	Drift at Ultimate Resistance mrad	Secant Stiffness kN/m	Ultimate Resistance F_{ult} kN	Drift at Ultimate Resistance mrad
G6 100 400 XX-1	Monotonic	261	12,560	36,4	-	-	-
G6 100 400 XX-2	Cyclic	280	14,920	36,5	317	-14,960	-36,6
G7 100 400 XX-1	Cyclic	429	14,240	28,4	606	-14,880	-24,3
G9 100 400 XX-1	Monotonic	2361	35,920	40,9	-	-	-
G9 100 400 XX-2	Cyclic	2356	35,840	31,5	2388	-39,520	-25,6

Table 5: Measured response of wall test specimens without sheathing.

In the tables, the results for monotonic tests (e.g., the preliminary tests) and cyclic tests are reported. As to the results of the wall G6 100 400 XX-1, it should be mentioned that the maximum deformation capacity allowed by the test set-up was reached before occurrence of collapse. However, local buckling failure of the studs had already commenced when the test was stopped. The test of wall G9 100 400 XX-2 also was stopped at the maximum displacement allowed by the test-rig. No signs of incipient collapse was observed. For these walls, the values listed in Table 5 are referred to the maximum resistance actually measured.

Specimen	Loading protocol	Positive Load			Negative load		
		Secant Stiffness	Ultimate Resistance F_{ult}	Drift at Ultimate Resistance	Secant Stiffness	Ultimate Resistance F_{ult}	Drift at Ultimate Resistance
		kN/m	kN	mrad	kN/m	kN	mrad
G5 100 400 BB-1	Monotonic	6760	64,200	9,7	-	-	-
G5 100 400 BB-2	Cyclic	5639	62,720	10,3	5535	-60,600	-10,1
G8 100 400 EF-1	Monotonic	6044	70,040	17,3	-	-	-
G8 100 400 EF-2	Cyclic	5463	66,800	10,8	5254	-68,880	-10,6
G9 100 400 GH-1	Monotonic	5320	76,920	13,3	-	-	-
G9 100 400 GH-2	Cyclic	3824	70,760	18,0	2769	-67,120	-14,1

Table 6: Measured response of sheathed wall test specimens.

In Figures 8-13 the results of the monotonic and cyclic tests are compared in terms of load-lateral displacement curves. In the figures, the dashed curves refer to the monotonic tests while the continuous curves are associated to the cyclic tests.

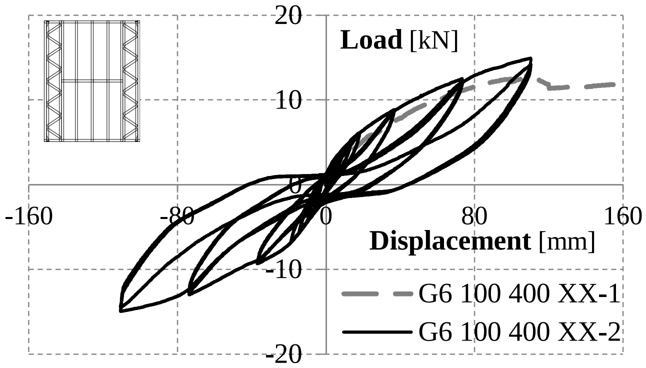


Figure 8: Monotonic and cyclic responses of walls G6 100 400 XX.

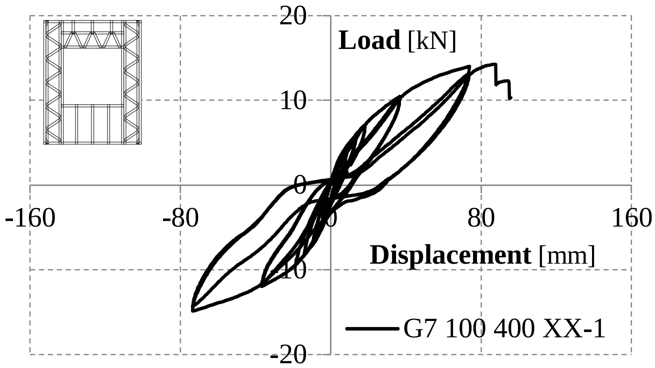


Figure 9: Cyclic response of wall G7 100 400 XX-1.

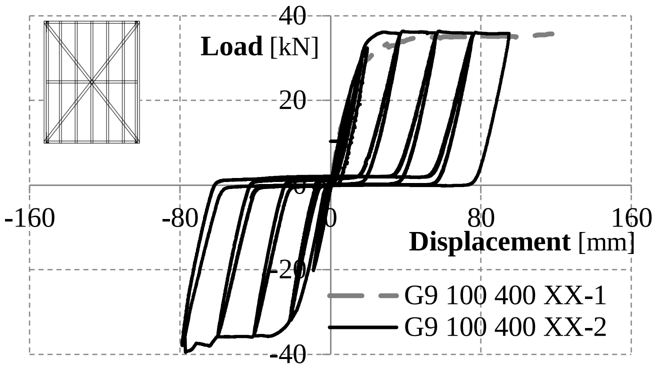


Figure 10: Monotonic and cyclic responses of walls G9 100 400 XX.

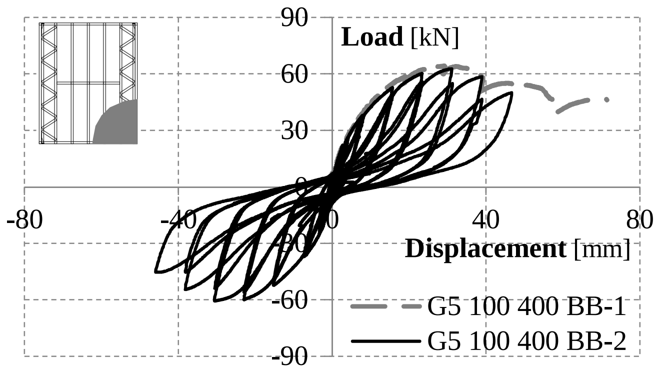


Figure 11: Monotonic and cyclic responses of walls G5 100 400 BB.

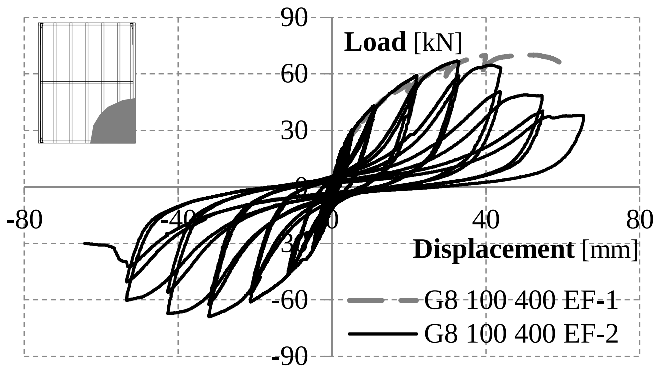


Figure 12: Monotonic and cyclic responses of walls G8 100 400 EF.

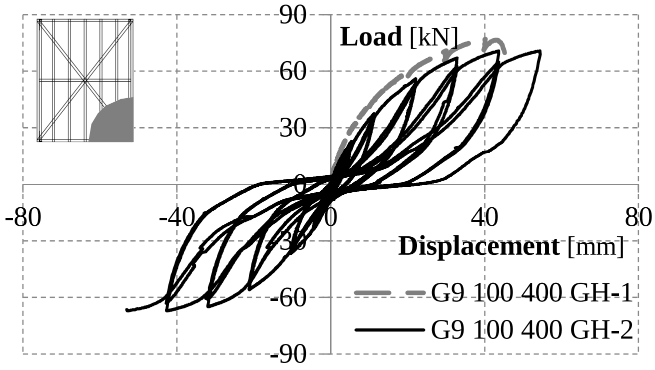


Figure 13: Monotonic and cyclic responses of walls G9 100 400 GH.

The results of the tests on the walls without sheathing (Table 5 and Fig.s 8-10) show that:

- specimen G7 100 400 XX-1 exhibited a stiffer performance than that of specimens G6 100 400 XX, although the wall incorporates a central opening. The horizontal trussed framing, connecting the lateral trussed bracing modules, results in a stiffer mechanism of force transmission which determinate an increase of the stiffness of approximately 50% respect to that of the specimens G6 100 400 XX. No remarkable increase of the resistance was observed: all of these walls were in fact associated with the same collapse mode (e.g., local instability phenomena);
- the best performance, in terms of both stiffness and resistance, was achieved with the adoption of an X-type bracing system. If the case of the specimens G6 100 400 XX is assumed as reference case, an increase of 805% and 186% in terms of stiffness and resistance, respectively, was achieved in the monotonic tests. In the cyclic test, an increase of 695% and 152%, for the stiffness and resistance, respectively, was observed.

As to the tests on the walls with sheathing (Table 6 and Fig.s 11-13), it can be observed 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 provided by the cement board sheathing.

In particular:

- the adoption of an X-type bracing system, i.e. the solution with the better performance in wall tests without sheathing, 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 (Fig. 7b);

– the complete absence of a steel bracing system for a sheathed wall seemed to have a negligible effect on the wall's performance: specimen G8 behaved in close agreement with the other tested walls.

An appraisal of the sheathing’s contribution to the walls’ performance is achieved by comparing the results of the un-sheathed and sheathed solutions. At this aim, for the sheathed walls G5 and G8 the results of the un-sheathed walls G6 are assumed as the reference case. The results, which are compared in Figures 14-16 for cyclic tests, clearly show that, independently from the steel bracing system adopted, a remarkable increase of both the stiffness and the resistance was achieved, which is more pronounced for the walls G5 and G8 than for wall G9.

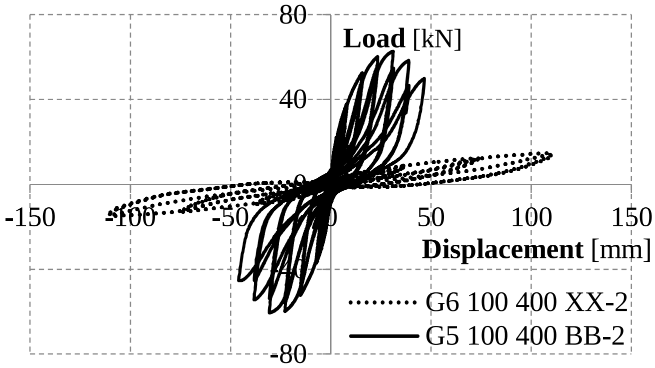


Figure 14: Test's results comparison between wall G6 100 400 XX-2 and G5 100 400 BB-2.

The presence of the sheathing leads to a different and more ‘efficient’ mechanism of forces transmission between the steel framing elements if compared to one of the un-sheathed solutions. The sheathing and the connections sheathing-to-steel-framing elements redistribute forces between the steel elements, and prevent or delay the instability phenomena of studs and chords. On the other hand the screws between studs and hold-downs and of the hold-down anchor rods are more severely stressed. The combination of these factor leads to improved resistance and stiffness but at the ‘price’ of reducing the ultimate deformation capacity.

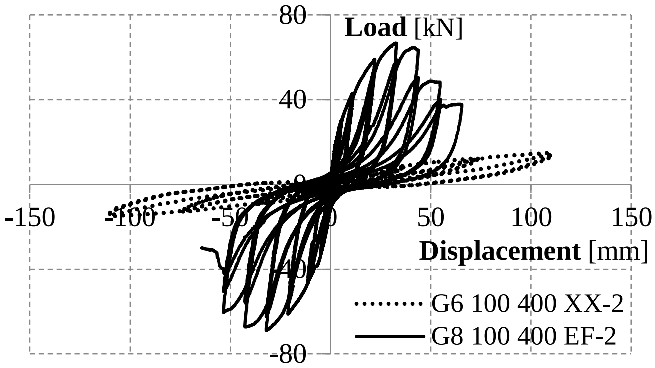


Figure 15: Test's results comparison between wall G6 100 400 XX-2 and G8 100 400 EF-2.

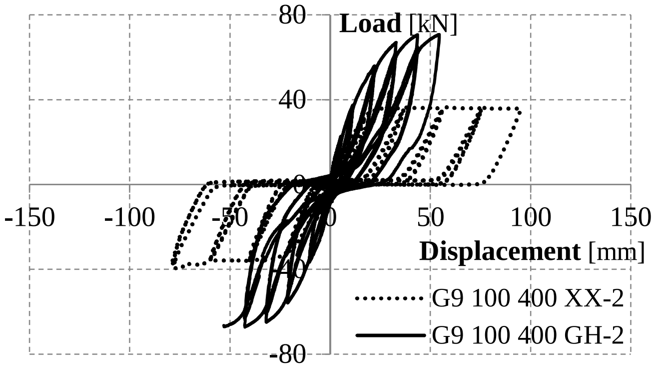


Figure 16: Test's results comparison between wall G9 100 400 XX-2 and G9 100 400 GH-2.

6 DUCTILITY PARAMETERS AND ENERGY DISSIPATION CAPACITY

The results presented in Figures 14-16 clearly point out the significant pinching effect which characterizes the cyclic response of the walls. This effect is more evident in the bare steel X-type bracing specimen G9 100 400 XX-2 (Fig. 16) whose behaviour is remarkably affected by the yielding of the diagonal strap in tension. The peculiar cyclic response of this wall can be observed in Figure 17 which compares its response to the one of specimen G6 100 400 XX-2. In the figure the load-lateral displacement curves associated to two cycles performed at maximum displacement values greater than e_{y_conv} (e.g., the displacement associated to the conventional yielding) are reported. The second cycle is highlighted in grey. The remarkable reduction of the dissipative capacity associated to the second cycle of specimen G9 100 400 XX-2 is apparent. The ratio between the energy dissipated in the first and second cycle is of 1,7 and 4,6 for wall G6 and G9, respectively. Nevertheless, the high deformation capacity and strength of wall G9 allows it to reach the highest ductility performance and energy dissipation as demonstrated in the following.

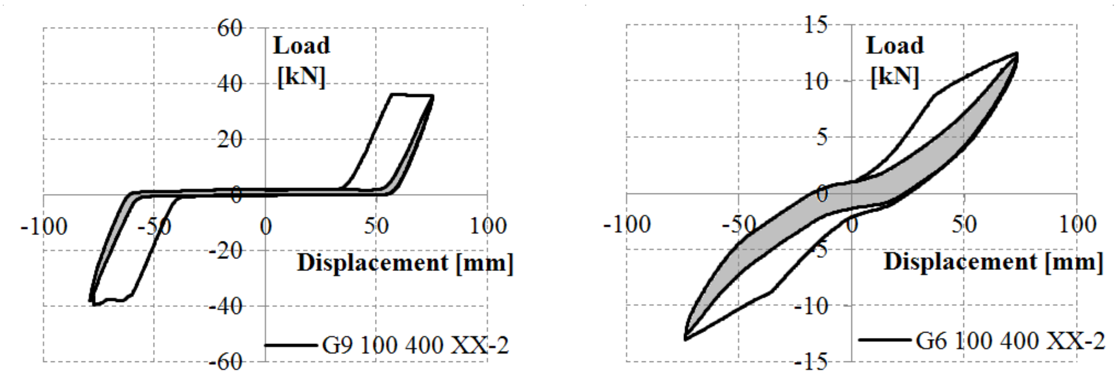


Figure 17: Comparison between G9 100 400 XX-2 and G6 100 400 XX-2 cyclic responses.

An improvement of the cyclic response for the X-type bracing wall is observed in the sheathed solution as shown in Figure 18, which compares the response of two subsequent cycles at the same lateral displacement of walls G5 and G9.

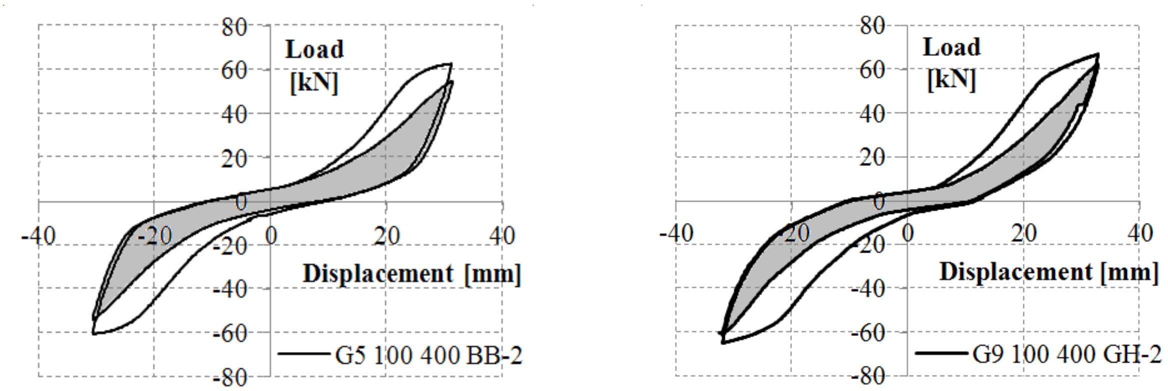


Figure 18: Comparison between G5 100 400 BB-2 and G9 100 400 GH-2 cyclic responses.

The pinching in wall G9 is substantially reduced and the ratio between the energy dissipated in the first and second cycle is comparable to that of wall G5 (ratios of 1,5 and 1,7 for wall G5 and G9, respectively). This result is associated with the more ‘efficient’ mechanism of shear resistance due to the combined mechanism involving the steel framing and the sheathing, which results in an enhanced cyclic response.

For a quantitative appraisal of the cyclic behaviour of the walls parameters such as the kinematic ductility, the cyclic ductility and the energy dissipated can be used. The kinematic and cyclic ductility parameters, μ_{kin} and μ_{cyc}, were hence determined according to the ECCS recommendations [22] as:

$$\mu_{kin} = e_i \;/\; e_y \tag{1}$$

$$\mu_{cyc} = \Delta e_i \;/\; e_y \tag{2}$$

where:

e_i absolute value of the maximum displacement of the ith cycle (Fig. 19);

e_{y_conv} absolute value of the displacement associated to the conventional yielding;

Δe_i absolute value of the maximum displacement in the positive or negative force range in the ith cycle (Fig. 19).

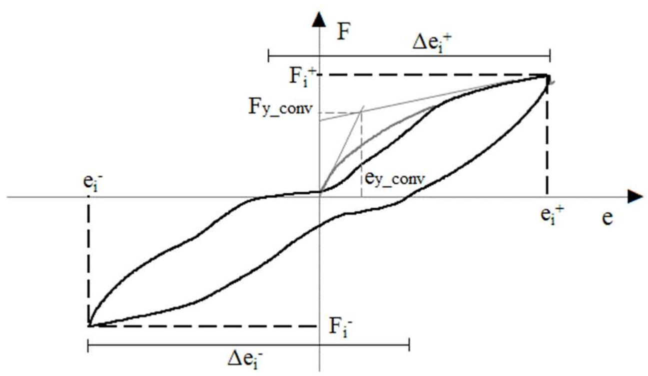


Figure 19: Displacement values to be accounted for the cyclic and kinematic ductility parameters.

Tables 7 and 8 summarise the main results of the calculations for the un-sheathed and sheathed walls, respectively. For each wall tested under cyclic conditions in the tables the maximum values of both the kinematic and cyclic ductility together with the total energy dissipated are reported. Aiming at an appraisal of the evolution of the energy dissipated during the tests, Figure 20 shows relationships between the energy dissipated and the lateral displacement.

Specimen	Kinematic ductility - μ_{kin}	Cyclic ductility - μ_{cyn}	Total energy dissipated kJ
G6 100 400 XX-2	6,09	8,47	3,55
G7 100 400 XX-1	4,85	5,80	2,68
G9 100 400 XX-2	9,73	16,32	7,60

Table 7: Ductility parameters and energy dissipated for specimen without sheathing.

Specimen	Kinematic ductility - μ_{kin}	Cyclic ductility - μ_{cyc}	Total energy dissipated kJ
G5 100 400 BB-2	13,55	16,99	9,25
G8 100 400 EF-2	11,17	15,13	13,23
G9 100 400 GH-2	10,00	13,77	10,37

Table 8: Ductility parameters and energy dissipated for sheathed specimens.

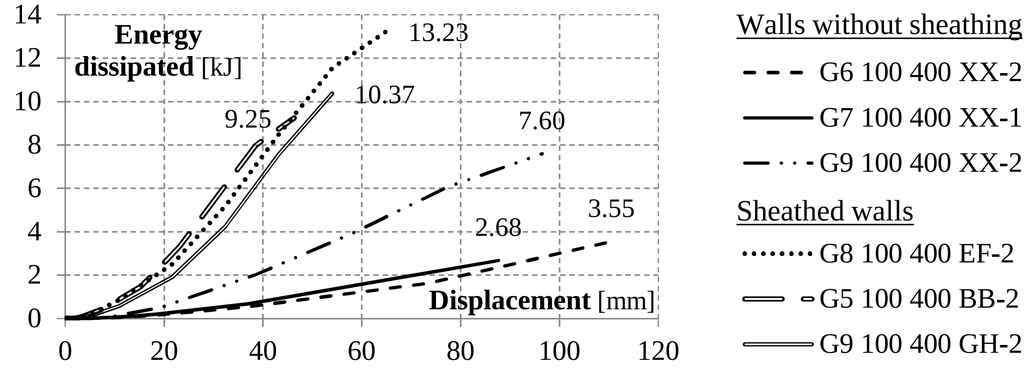


Figure 20: Energy dissipated vs. lateral displacement.

The values of the kinematic and cyclic ductility as well as of the total energy dissipated are rather high and confirm the remarkable deformation capacity of this type of walls. It should be noted that the sheathed solutions (Table 8) possess the greater values of both the ductility parameters and the total dissipated energy. The key role of the sheathing in contributing not only to stiffness and strength (see Tables 5 and 6) but also to the walls' dissipative capacity is apparent. Among the un-sheathed solutions (Table 7) the X-type bracing walls show the best performance also in the cyclic regime, while the poorest behaviour is that of wall G7 100 400 XX-1, despite the high stiffness and strength (Table 5). Among the sheathed solutions, the wall G8 100 400 EF-2, the wall with no steel bracing system, shows values of stiffness and strength comparable with the braced solutions (e.g., walls G5 100 400 BB-2 and G9 100 400 GH-2) (see Table 6), and even the best cyclic performance (Table 8 and Fig. 20) in terms of dissipated energy. This result stresses once more the substantial contribution of the sheathing to both vertical and lateral loads resistance also under cyclic loading.

7 ANALITICAL EVALUATION OF THE WALLS’ RESPONSE

As mentioned in the above, the central role of shear walls in the mechanism of transmission of both gravity and lateral loads led to extensive investigations aiming to an in-depth understanding of their behavior and to the identification of the key parameters affecting their response. Wooden shear walls were investigated first allowing researchers to identify the central role of sheathing and of sheathing to framing connections to the walls’ lateral response. The wide experimental experience also allowed establishing simple analytical models for ‘hand’ calculations providing simplified computational tools for evaluating elastic stiffness and strength of the walls. 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 the framed support is disregarded. Experimental results of the walls tested in Trento clearly showed the substantial influence of the sheathing and of the sheathing to framing connection on the overall wall response. This outcome indicates that the same assumptions can be made. A study of the effectiveness of few selected models developed for wooded shear walls in evaluating the response of the CFS walls studied in Trento was hence performed. The methods of Tuomi and

McCutcheon [23, 24], Easley et al. [25], Kallsner and Girhammar [26] are considered. These methods are based on static equilibrium or energetic approaches and apply to the wall's response in the elastic range. The following two hypothesis are also assumed by the methods:

- sheathing material homogeneous and isotropic;
- sheathing to framing elements fasteners characterised by a linear load-displacement relationship.

The practical application of the methods require the mechanical characterization under shear of the sheathing and of the sheathing to framing connections. At this aim, ancillary tests focusing on the response of both sheathing and connections adopted on the walls considered in this study were performed as described in the following sections. All tests were performed with a universal loading machine Galdabini (model PM10, maximum capacity of 100kN, class 0,5 as for the EN ISO 7500-1:2004 [27]).

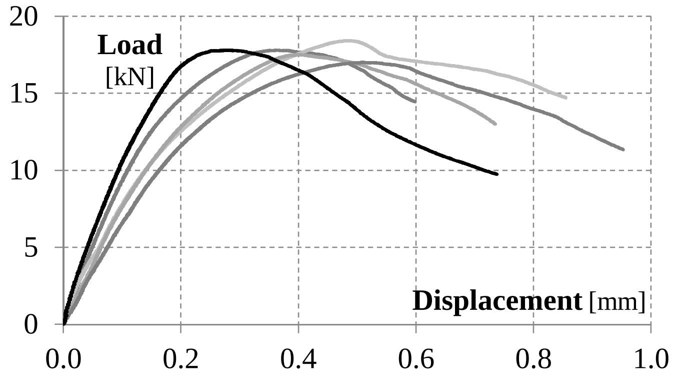
7.1 Edgewise shear tests

The tests were performed according to the provisions of ASTM D1037-12 [28]. Specimens with nominal dimensions of 90x250mm were taken from the sheathing panels considered in the study and loaded in edgewise shear (Fig. 21).

Figure 21: Test set-up for edgewise shear test.

Tests were performed under displacement control with a speed of 0,05 mm/s. During the tests, the load and the shear displacement were recorded.

For each sheathing material at least four tests were performed. Additional tests were carried out in case of scatter of results greater than 10% (Table 9). Figure 22 illustrates the tests results for sheathing type F and the related failure modes.



a)

b)

Figure 22: Edgewise shear test: a) typical tests results, b) collapse mode.

Test results were analysed so as to evaluate the shear modulus G and the shear stress τ according the statistical procedure proposed in [29].

The tests results in terms of mean values of the shear module and the maximum shear stress are reported in Table 9.

Sheathing type	n. tests	G N/mm^2	τ_{max} N/mm^2
B	4	1346	3,97
E	4	1584	7,69
F	5	1395	5,99
G	6	792	2,93
H	5	1235	3,87

Table 9: Edgewise shear tests' results.

7.2 Shear tests on the stud-sheathing connections

The tests were performed following the procedures proposed by the ASTM D 1761-88 [30] for the evaluation of the mechanical properties of fasteners in wood, as modified by Serrette et al. [18]. 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. 23). The studs were connected to two 600x600mm sheathing panel by means of the screws adopted in the tested 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 (Fig. 23).

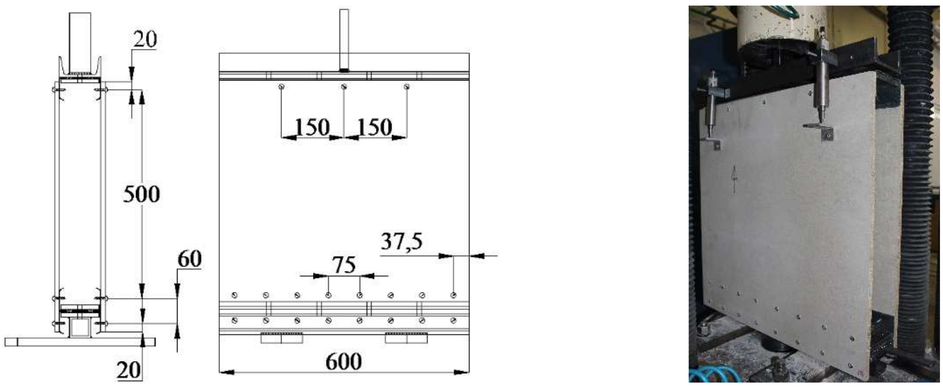
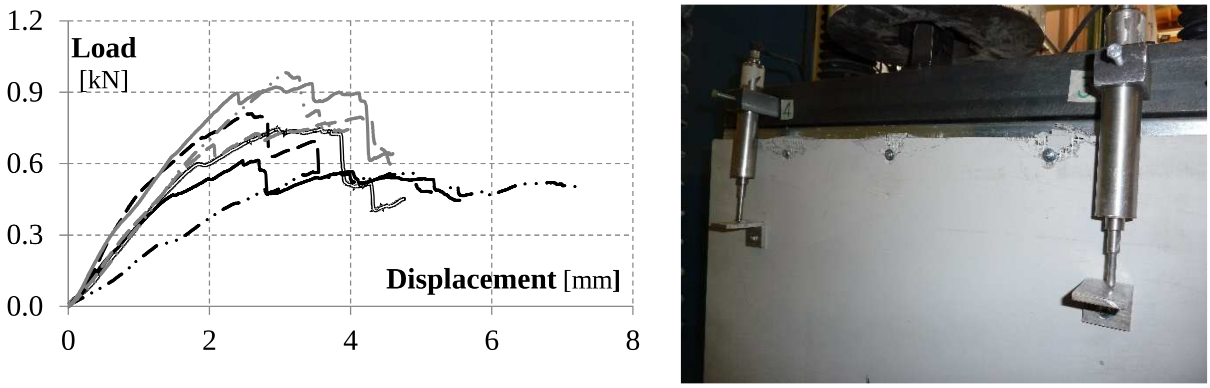


Figure 23: Specimen and test set-up for shear tests on the connections (measures in mm).

The specimens were tested under pure tension and displacement control. In tests, the displacements of the top of the specimen were measured in two positions and on both the sides of the specimens (Fig. 23). Both load applied and displacements were recorded during the tests. For each type of sheathing a minimum of four tests were performed. In case of wide scatter of the tests' results additional tests were executed. As an example of the tests results, Figure 24 illustrates the load-displacement curves for the sheathing type G and the observed collapse mode. Figures 25 and 26 illustrate the typical collapse modes for the other sheathings types considered.



a) b)

Figure 24: Test of the sheathing to framing connection: a) typical tests results, b) collapse mode.

Sheathing type B Sheathing type E

Figure 25: Test of the sheathing to framing connection: typical collapse modes.

Sheathing type F Sheathing type H

Figure 26: Test of the sheathing to framing connection: typical collapse modes.

Tests' results were analysed to evaluate the stiffness and the ultimate resistance of the connections. Let n be the number of tests performed for the same sheathing to frame connection, the following procedure enabled determination of the connection stiffness k_{mean} [29]:

1. the 5% characteristic value of connection's resistance (F_{uk}) is first obtained as:

$$F_{uk} = F_{u,mean} - k \cdot s \tag{3}$$

where

$F_{u,mean}$ mean value of the ultimate resistance obtained for the n tests;

k statistical coefficient depending on n;

s standard deviation.

2. the secant stiffness $k_{i,sec}$ for the each test is then determined by equating the dashed areas A_1 and A_2 in Figure 27;
3. the stiffness k_{mean} is finally obtained as the mean value of $k_{i,sec}$.

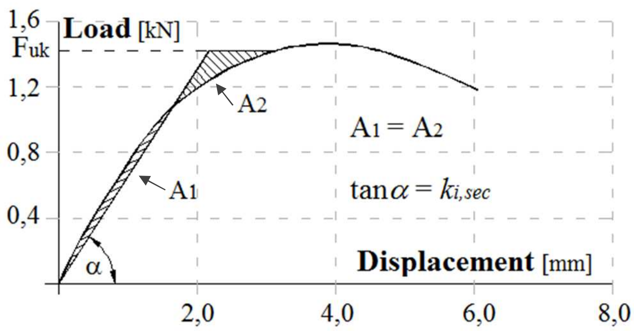


Figure 27: Secant stiffness for the sheathing to framing connection.

Table 10 summarises the results of the tests in terms of mean values of the stiffness (k_{mean}) and of the maximum load ($F_{u,mean}$) related to a single screw.

Sheathing type	n. tests	k_{mean} N/mm	$F_{u.mean}$ N
B	4	582	1260
E	7	398	1460
F	7	337	1322
G	7	354	780
H	6	424	1083

Table 10: Shear tests' results of the stud-sheathing connections.

7.3 Wall's response evaluation

The application of the methods of Tuomi and McCutcheon [23, 24], Easley et al. [25], Kallsner and Girhammar [26] to the walls considered in this paper was performed under the following assumptions:

- the overall resistance of the CFS wall is evaluated as sum of the resistance associated to the single sheathing panel;
- the contribution offered by continuous tracks, which characterized the CFS walls, is neglected;
- the performance of the sheathing to frame connection does not depend on the direction of the acting shear force.

Moreover, the methods of Tuomi and McCutcheon [23, 24], Easley et al. [25], in the form proposed by the authors, would require a regular and symmetric pattern of the fasteners. A generalization of these methods to the case of irregular pattern of the fasteners, which characterized the cases investigated on Trento, was hence necessary.

Tables 11-13 compare the lateral resistance (F_{hand}) and the lateral displacement (e_{hand}) evaluated by means of the analytical methods with the experimental results. For the comparison, the cases of the three sheathed walls tested under monotonic loading were considered. For them, the force and displacement associated with the experimental conventional elastic limit, F_{y_conv} and e_{y_conv} respectively, evaluated according to [29] were assumed.

The main goal of the comparison is to check the applicability of the methods. It should be stressed indeed that the simplified assumptions at the base of all the methods make them not able to catch the complex mechanisms of force transmission between structural components.

In particular, these methods do not incorporate the contributions of the steel framing and of the anchorages. This is apparent from the results related to the wall G9 100 400 GH-1, i.e., the wall with diagonal bracing system, for which the contribution of the steel skeleton is significant.

Hand method	F_{hand} kN	F_{hand}/F_{y_conv}	e_{hand} mm	e_{hand}/e_{y_conv}
Tuomi & McCutcheon	54,67	1,00	13,16	1,26
Easley & al.	33,22	0,61	7,97	0,76
Kallsner & Girhammar	44,37	0,81	11,14	1,07

Table 11: Experimental vs hand methods results for wall G5 100 400 BB-1.

Hand method	F_{hand} kN	F_{hand}/F_{y_conv}	e_{hand} mm	e_{hand}/e_{y_conv}
Tuomi & McCutcheon	47,09	0,79	16,82	1,06
Easley & al.	28,61	0,48	10,19	0,64
Kallsner & Girhammar	38,22	0,64	14,27	0,90

Table 12: Experimental vs hand methods results for wall G8 100 400 EF-1

Hand method	F_{hand} kN	F_{hand}/F_{y_conv}	e_{hand} mm	e_{hand}/e_{y_conv}
Tuomi & McCutcheon	28,26	0,43	8,48	0,51
Easley & al.	17,17	0,26	5,13	0,31
Kallsner & Girhammar	22,93	0,35	7,17	0,43

Table 13: Experimental vs hand methods results for wall G9 100 400 GH-1.

As to the two specimens G5 100 400 BB-1 and G8 100 400 EF-1, for which the sheathing provides the key contribution to the wall’s response, results of Tables 11-12 show that the Tuomi and McCutcheon [23, 24] approach enables a reasonably good assessment of the elastic shear capacity. On the contrary, the associated lateral displacement tends to be overestimated. For the same walls, the other analytical methods leads to a less accurate appraisal of both strength and lateral displacement.

8 CONCLUSIONS

The paper presents a limited series of tests part of a study focused on the response under monotonic and cyclic loading of walls made up of light gauge members. These walls are typical of light steel residential buildings. The goal of the study was to understand the response and compare the effectiveness of different bracing systems. As to the bare steel specimens, the use of diagonal bracing with flat straps, of trussed bracing and of a trussed frame are investigated.

The last solution enables incorporation of openings into the wall. The sheathing can provide also an important bracing action, which also was investigated in the work.

As expected, the performance of the walls with diagonal bracing is the best under all aspects. However, solutions using trussed members appear to be adequate for moderate wind and/or seismic loads [31].

The sheathing substantially contributes by itself to the lateral response, as clearly shown by specimens G8, whose steel framing is characterized by the absence of any bracing.

This importance points out the need of an appraisal of the 'skin' behavior, including its connections to the steel skeleton. The reliability of existing analytical methods developed for wooden shear was hence investigated. The limited comparison between experimental and analytical results does not allow to drawn general conclusions. However, the results obtained show that the simplified hypotheses at the base of the methods prevent a satisfactory evaluation of both strength and stiffness, in particular when the contributions of the steel framing and of the anchorages are significant.

A wider set of wall's configurations, and of sheathing material is going to be part of the next phases of the study. However, it seems already possible to state that this type of walls is more than adequate for use in low rise residential buildings in low to moderate seismic zones. Furthermore, it can be concluded that at this stage, the experimental approach seems to be the most reliable for the evaluation of the response of CFS shear walls.

ACKNOWLEDGMENTS

This project was funded by the Autonomous Province of Trento and Cogi s.r.l.. Partial support came also from the Programme ReLUIS III-2014/2015. The Authors also gratefully acknowledge the work by the laboratory technicians at the University of Trento: Stefano Girardi, Marco Graziadei and Alessandro Banterla.

REFERENCES

[1] Adham S.A., Avanessian V., Hart G.C., Anderson R.W., Elmlinger J. and Gregory J., "Shear wall resistance of lightgage steel stud wall systems", *Earthquake Spectra*, **6**(1), 1-14, 1990.

[2] Fülöp L.A. and Dubina D., "Performance of wall-stud cold-formed shear panels under monotonic and cyclic loading. Part I: Experimental research", *Thin-Walled Structures*, **42**, 321-338, 2004.

[3] Barton A.D., *Performance of steel framed domestic structures subject to earthquake loads*, PhD Thesis, Department of Civil and Environmental Engineering, University of Melbourne, Melbourne, Australia, 1997.

[4] Serrette R. and Ogunfunmi K., "Shear resistance of gypsum-sheathed light-gauge steel stud walls", *Journal of Structural Engineering*, **122**(4), 383-389, 1996.

[5] Velchev K., Comeau G., Balh N. and Rogers C. A., "Evaluation of the AISI S213 seismic design procedures through testing of strap braced cold-formed steel walls", *Thin-Walled Structures*, **48**(10-11), 846-856, 2010.

[6] Gad E.F., Duffield C.F., Hutchinson G.L, Mansell D.S. and Stark G., "Lateral performance of cold-formed steel-framed domestic structures", *Journal of Engineering Structures*, **21**, 83-95, 1999.

[7] Al-Kharat M. and Rogers C. A., "Inelastic performance of cold-formed steel strap braced walls", *Journal of Constructional Steel Research*, **63**(4), 460-474, 2007.

[8] Al-Kharat M. and Rogers C. A., "Inelastic performance of screw connected cold-formed steel strap braced walls", *Canadian Journal of Civil Engineering*, **35**(1), 11-26, 2008.

[9] Tian Y.S., Wang, J. and Lu, T.J., "Racking strength and stiffness of cold-formed steel wall frames", *Journal of Constructional Steel Research*, **60**, 1069-1093, 2004.

[10] Moghimi H. and Ronagh H.R., “Better connection details for strap-braced CFS stud walls in seismic regions”, *Thin-Walled Structures*, **47,** 122-135, 2009.

[11] Moghimi H. and Ronagh H.R., “Performance of light-gauge cold-formed steel strap-braced stud walls subjected to cyclic loading”, *Engineering Structures*, **31**, 69-83, 2009.

[12] Kim T.W., Wilcoski J., Foutch D.A. and Lee M.S., “Shaketable tests of a cold-formed steel shear panel”, *Engineering Structures,* **28**, 1462-1470, 2006.

[13] Comeau G., Velchev K. and Rogers C.A., “Development of seismic force modification factors for cold-formed steel strap braced walls”, *Canadian Journal of Civil Engineering*, **37**(2), 236-249, 2010.

[14] AISI S213-07/S1-09, *North American Standard for Cold-Formed Steel Framing – Lateral Design 2007 Edition with Supplement No. 1, Reaffirmed 2012*, American Iron and Steel Institute, Washington, DC, USA, 2009.

[15] Macillo V., Iuorio O., Terracciano M. T., Fiorino L. and Landolfo R., “Seismic response of Cfs strap-braced stud walls: Theoretical study”, *Thin-Walled Structures*, **85**, 301–312, 2014.

[16] Baran E. and Alica C., “Behavior of cold-formed steel wall panels under monotonic horizontal loading”, *Journal of Constructional Steel Research*, **79**, 1-8, 2012.

[17] Zeynalian M. and Ronagh H.R., “A numerical study on seismic performance of strap-braced cold-formed steel shear walls”, *Thin-Walled Structures*, **60**, 229–238, 2012.

[18] Serrette R., Encalada J., Juadines M. and Nguyen H., “Static racking behavior of plywood, OSB, gypsum, fiberbond walls with metal framing“, *Journal of Structural Engineering*, **123**, 1079-1086, 1997.

[19] Baldassino N., Accorti M., Zandonini R., Scavazza F. and Rogers C.A., “*The use of trussed frame cold-formed steel shear walls: an experimental study*”, University of

Trento, Department of Civil, Environmental and Mechanical engineering, Laboratory report, 2014.

[20] Baldassino N., Accorti M., Zandonini R., Scavazza F. and Rogers C.A., *"The use of cement board sheathing on cold-formed steel shear walls: an experimental study"*, University of Trento, Department of Civil, Environmental and Mechanical engineering, Laboratory report, 2014.

[21] EN ISO 6892-1:2009, *Metallic materials - Tensile testing - Part 1: Method of test at room temperature*, European Committee for standardization, Brussels, 2009.

[22] Technical Committee 1- Structural Safety and Loadings - Technical Working Group 1.3 - *Seismic Design, Recommended Testing Procedure for Assessing the Behaviour of Structural Steel Elements under Cyclic Loads*, European Convention for Constructional Steelwork, 45, 1986.

[23] Tuomi R.L. and McCutcheon W.J., "Racking strength of light-frame nailed walls", *Journal of Structural Division*, ASCE **104** (**ST7**), 1131-1140, 1978.

[24] McCutcheon W.J., "Racking deformation in Wood Shear Walls", *Journal of Structural Engineering*, **111**, 257-269, 1985.

[25] Easley J.T., Foomani M. and Dodds R.H., "Formulas for wood shear walls*", Journal of Structural Division*, ASCE **108** (**ST11**), 2460-2478, 1982.

[26] Kallsner B. and Girhammar U.A., "Analysis of fully anchored light-frame timber shear walls-elastic model", *Materials and Structures*, *42*, 301–320, 2009.

[27] EN ISO 7500-1:2004, *Metallic materials – Verification of static uniaxuial testing machine – Part 1: tension/compression testing machines – Verification and calibration of the force-measuring system (ISO 7500-1:2400)*, European Committee for Standardization, Brussels, 2004.

[28] ASTM D1037-12, *Standard Test Methods for Evaluating Properties of Wood-Base Fiber and Particle Panel Material,* American Society for Testing and Materials International, West Conshohocken, PA, 19428-2959 USA, 2012.

[29] Accorti M., Baldassino N. and Zandonini R., "*Guidelines for the design by testing of CFS shear panels for residential buildings*", Civil Protection Department, 2015 (in Italian).

[30] ASTM D 1761-88, *Standard Test Methods Mechanical Fasteners in Wood,* American Society for Testing and Materials International, West Conshohocken, PA, 19428-2959 USA, 1988.

[31] "*SteelMax: Experimental and numerical analysis of the performance of CFS profiles and subassemblies under elementary and complex states of stress*", Research report, Department of Civil, Environmental and Mechanical Engineering, University of Trento, 2014 (in Italian).