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EVALUATION OF THE SEISMIC PERFORMANCE OF LIGHT GAUGE STEEL WALLS BRACED WITH FLAT STRAPS.

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ABSTRACT

The development of light weight steel structures in seismic area as Italy requires the upgrading of National Codes. To this end, in the last years a theoretical and experimental study was carried out at the University of Naples within the research project RELUIS-DPC 2010-2013. The study focused on "all steel design" solutions and investigated the seismic behaviour of strap braced stud shear walls. Three wall configurations were defined according to both elastic and dissipative design criteria for three different seismic scenarios. The lateral in-plane behavior of these systems were evaluated by 12 tests performed on full-scale CFS strap-braced stud wall specimens with dimensions 2.4 m x 2.7 m subjected to monotonic and reversed cyclic loading protocols. The experimental campaign was completed with 17 tests on materials, 8 shear tests on elementary steel connections and 28 shear tests on strap-framing connection systems. On the basis of the experimental results, and taking into account the AISI S213 provisions, behaviour factors were evaluated. This paper provides the main outcomes of the experimental tests on walls and behaviour factors evaluation.

Introduction

The Cold-Formed Steel (CFS) structures are able to ensure a good structural response in seismic areas. In these structures, the lateral load bearing systems are CFS stud walls, that are generally realized with frames in CFS profiles braced by sheathing panels or light gauge steel straps installed in a X configuration. The seismic behaviour of CFS structures laterally braced by panels ("sheathing-braced" approach) was the object of several studies carried out at the University of Naples "Federico II" in the last years [Landolfo et al.2006, Fiorino et al. 2007, Iuorio 2007, Fiorino et al. 2008, 2014]. When a X-braced configuration is adopted, the design is carried out according to a "all-steel" approach and steel straps are generally used to obtain the diagonal elements. In particular, because of the steel straps slenderness, only those in tension are considered active. Therefore, the lateral load applied on a wall is adsorbed only by the diagonal in tension, which transmits a significant axial compression force to the ends of the wall. For this reason, the design of members and connections located at wall corners is crucial, especially for the chord studs, strap connections, gusset plate and anchors. Guidelines for the seismic design of CFS structures are not provided by the European codes (EN 1998-1). Hence, as an attempt to provide a contribution to the code development, a theoretical and experimental study was carried out by the Authors within the Italian research project RELUIS-DPC 2010-2013. In the following, the experimental investigation and the evaluation of behavior factors are presented.

Test Program

In order to investigate a large range of possible CFS solutions for low-rise dwellings, three buildings to be located in different seismic area were designed. Each of them has a rectangular plan with dimensions 12.2 m x 18.1 m and storey height of 3.00 m. The lateral resisting system is made of CFS strap-braced stud walls that were designed according to elastic or dissipative design approaches. Therefore, three wall configurations were defined as follows: elastic light (WLE), dissipative light (WLD) and dissipative heavy (WHD) walls (Fig. 1). More details about the case study and the design of walls are presented in the papers Iuorio et al. and Macillo et al.

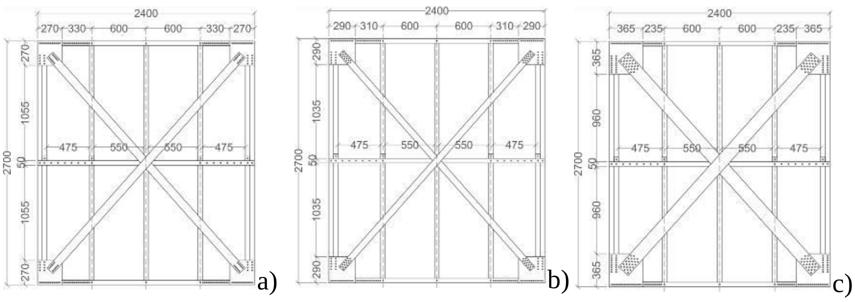


Figure 1. Schematic drawings of the three wall configurations: a) elastic light wall (WLE); b) dissipative light wall (WLD); c) dissipative heavy wall (WHD)

The lateral response of these systems was investigated by testing each of the three selected configurations by two monotonic and two cyclic tests for a total of 12 tests on full-scale wall specimens in size of 2400 x 2700 mm. Moreover, taking into account that materials and components influence the wall seismic global response in terms of lateral resistance, stiffness and ductility, the components response was investigated by means of 17 tests on materials, 8 shear tests on elementary connections between steel profiles and 28 shear tests on connections between gussets and strap-bracings. The experimental campaign is summarized in Table 1. All tests were carried out in the laboratory of the Department of Structures for Engineering and Architecture of the University of Naples Federico II. In the following the tests on walls and on connections between gussets and strap-bracings are presented in detail.

<table>
<tr><th colspan="10">WALLS</th></tr>
<tr><td>label</td><td>WLE</td><td colspan="4">WLD</td><td colspan="4">WHD</td></tr>
<tr><td>no. monotonic tests</td><td>2</td><td colspan="4">2</td><td colspan="4">2</td></tr>
<tr><td>no. cyclic tests</td><td>2</td><td colspan="4">2</td><td colspan="4">2</td></tr>
<tr><th colspan="10">MATERIALS</th></tr>
<tr><td>label
(steel grade - thickness in mm)</td><td>S350 - 1.5</td><td colspan="4">S235 – 2.0</td><td colspan="4">S350 – 3.0</td></tr>
<tr><td>no. tests</td><td>$3_a + 3_b$</td><td colspan="4">$2_a + 3_b$</td><td colspan="4">$3_a + 3_b$</td></tr>
<tr><th colspan="10">ELEMENTARY CONNECTIONS</th></tr>
<tr><td>label</td><td>SLE</td><td colspan="4">SLD</td><td colspan="4">SHD</td></tr>
<tr><td>no. tests</td><td>3_b</td><td colspan="4">3_b</td><td colspan="4">2_b</td></tr>
<tr><th colspan="10">JOINTS between GUSSETS and STRAP-BRACINGS</th></tr>
<tr><td>label</td><td>CLE</td><td colspan="4">CLD</td><td colspan="4">CHD</td></tr>
<tr><td>configuration</td><td>1</td><td>1</td><td>2</td><td>3</td><td>4</td><td>1</td><td>2</td><td>3</td><td>4</td></tr>
<tr><td>no. tests</td><td>$3_a + 3_b$</td><td>$3_a + 3_b$</td><td>2_b</td><td>2_b</td><td>2_b</td><td>$1_a + 3_b$</td><td>2_b</td><td>2_b</td><td>2_b</td></tr>
</table>

$_a$ stands for test speed equal to 50 mm/s;
$_b$ stands for test speed equal to 0.05 mm/s;
WLE is Elastic Light Wall; WLD is Dissipative Light Wall; WHD is Dissipative Heavy Wall;
SLE is Single connection for Elastic Light wall; SLD is Single connection for Dissipative Light wall; SHD is Single connection for Dissipative Heavy wall;
CLE is Connection joint for Elastic Light wall; CLD is Connection joint for Dissipative Light wall; CHD is Connection joint for Dissipative Heavy wall

Table 1. Experimental program

Tests on full-scale CFS strap-braced stud walls

The lateral in-plane behaviour of the selected wall configurations (WLE, WLD, WHD) was investigated by means of 12 physical tests, including six monotonic tests and six cyclic tests on full-scale 2400 mm long and 2700 mm high wall specimens. The wall framing (Fig.2) was made with stud members, having lipped channel sections (C-sections), spaced at 600 mm on the center and connected at the ends to track members, having unlipped channel sections (U-sections). Since chord studs are subjected to higher axial load, aiming to avoid any buckling and failure of those studs, they were composed by double C-sections screwed back-to-back. In order to reduce the unbraced length of the chord and interior studs, flat straps were placed at the mid-height of the wall specimens and were screwed to blocking members placed at the ends of walls. The local buckling phenomena of tracks were avoided by reinforcing the ends of members with C-section profiles assembled in a box sections. Hold-down devices, made with S700 steel grade, and connected to the studs by four M16 class 8.8 bolts and to the beams of the testing frame by one M24 class 8.8 bolt, were placed within the lower and upper track at the four corners of the walls. The upper and bottom tracks of the tested walls were connected respectively to the loading (top) and bottom beams of the testing frame by M8 class 8.8 bolts spaced at 300 mm on the center, which were used as shear connections. The wall specimens were completed with strap braces installed in an X configuration on both sides and connected to the wall framing by gusset plates. For each wall configuration an appropriate fastener was chosen: 6.3 x 40 mm (diameter x length) hexagonal flat washer head self-drilling screws (AB 04 63 040 type) for WLE and WHD specimens, and 4.8 x 16 mm modified truss head self-drilling screws (CI 01 48 016 type) for WLD prototypes, produced by Tecfi S.p.A. All the steel members were fabricated by S350GD+Z steel grade, except the diagonal straps of dissipative systems, which were made with S235 steel grade. Table 2 lists the nominal design dimensions and material properties of the tested wall components. Schematic drawings of the WHD configuration is provided in Figure 2.

	WLE		WLD		WHD	
	Section [mm]	Grade	Section [mm]	Grade	Section [mm]	Grade
Studs	C150x50x20x1.5[a]	S350	C150x50x20x1.5[a]	S350	C150x50x20x3.0[a]	S350
Tracks	U153x50x1.5[b]	S350	U153x50x1.5[b]	S350	U153x50x1.5[b]	S350
Diagonal straps	90x1.5[c]	S350	70x2.0[c]	S235	140x2.0[c]	S235
Gusset plates	270x270x1.5[d]	S350	290x290x1.5[d]	S350	365x365x1.5[d]	S350
Track reinforc.	C150x50x20x1.5[a]	S350	C150x50x20x1.5[a]	S350	C150x50x20x3.0[a]	S350
Blocking members	C150x50x20x1.5[a]	S350	C150x50x20x1.5[a]	S350	C150x50x20x3.0[a]	S350
Flat straps	50x1.5[c]	S350	50x1.5[c]	S350	50x1.5[c]	S350

[a] C-section: outside-to-outside web depth x outside-to-outside flange size x outside-to-outside lip size x thickness; [b] U-section: outside-to-outside web depth x outside-to-outside flange size x thickness; [c] width x thickness; [d] height x width x thickness

Table 2. Nominal design dimensions and material properties of the tested wall components

Tests on full-scale wall specimens were carried out by using a specifically designed testing frame for in-plane shear loading (Fig. 3). Horizontal loads were transmitted to the upper wall track by means of a steel beam made of a 200x120x10 mm (width x height x thickness) rectangular hollow section. The out-of-plane displacements of the wall were avoided by two lateral supports realized with HEB 140 columns and equipped with double roller wheels. The tests were performed by using a hydraulic actuator having ±250 mm stroke displacement and 500 kN load capacity. A sliding-hinge was placed between the actuator and the tested wall in order to avoid the transmission of external vertical load components. Eight LVDTs were used to measure the specimen displacements. In particular, three LVDTs (W1, W2 e W3) were installed to record the horizontal displacements and two LVDTs (W4, W5) for the vertical displacements. The local deformations of the diagonal straps were recorded by means of two strain-gauges for each diagonal (S1 and S4 placed at the end and S2 and S3 placed in the center of the straps). A load cell was used to measure the applied loads.

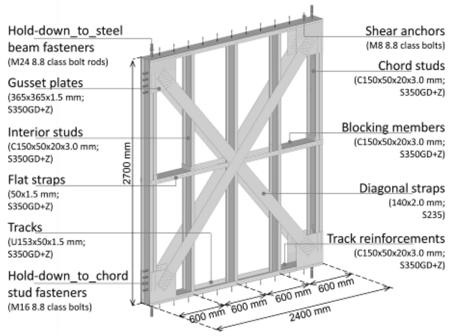


Figure 2. WHD wall configuration

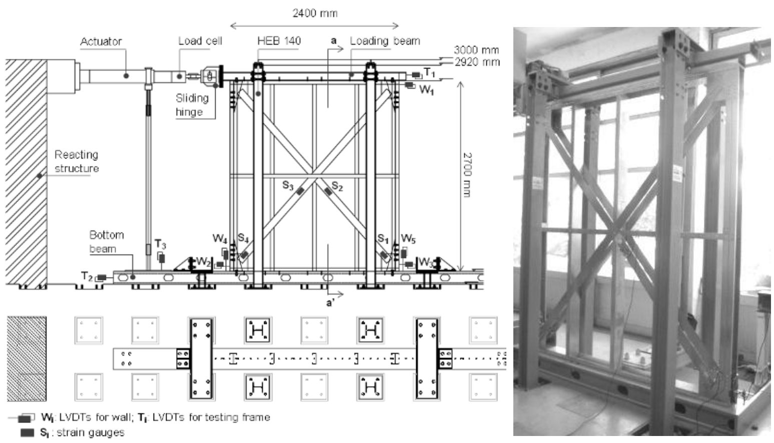


Figure 3. Test on full-scale walls

Monotonic tests

In the monotonic loading regime, the tests were performed by applying a loading protocol organized in two phases. In the first phase the wall specimens were pulled and in the second phase they were pushed. Both phases have been followed by the unloading of the wall prototypes in order to lead them back to the initial position. This testing protocol involved displacements at a rate of 0.10 mm/s up to a maximum of ± 240 mm defined by the stroke limit of the actuator or until the occurred collapse.

Test results in terms of yield strength (H_y), maximum strength (H_{max}), displacement at the conventional elastic limit (d_y), maximum displacement (d_{max}), conventional elastic stiffness (k_e), defined as the secant stiffness at 40% of the maximum strength, and observed failure mechanisms are shown in Table 3. In addition, Table 3 provides the theoretical predicted values of the strength and stiffness, which were evaluated using the experimental mechanical properties, and the ratios between the average experimental and theoretical values. Figure 4 shows the acting load (H) vs top wall displacement (d) curves for the WHD-M2 prototypes with the experimental values measured in the pulling and pushing phases and the predicted parameters, which are used to evaluate the structural response. Test results reveal a decrement of maximum strength contained within 12% in the pushing phase with respect to the pulling phase, while the conventional elastic stiffness records significant decrement up to 42% in the pushing phase, due to the occurrence of local damages of some wall components in the previous pulling phase. Moreover, the strength prediction is very close to the experimental results. In agreement with the predicted failure mechanisms, the WLE configurations collapse was reached with the net section failure of diagonal straps (Fig. 5a), while the performance of WLD and WHD specimens was governed by the brace yielding (Fig. 5b) up to the maximum stroke of the actuator without reaching the wall failure.

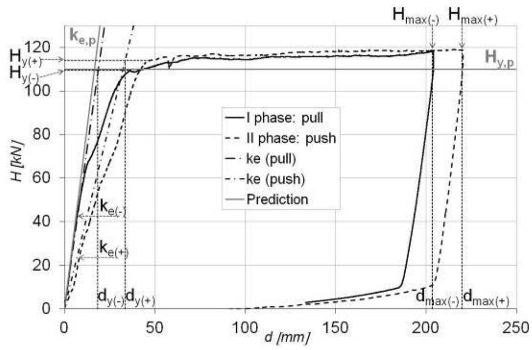


Figure 4. Monotonic test on WHD-M2 specimen: load vs. displacement curve

type		H_y [kN]	H_{max} [kN]	d_y [mm]	d_{max} [mm]	k_e [kN/mm]	failure mode
WLE-M1	pull/push	64.9/65.6	66.3/66.6	18.5/24.3	36.7/35.3	3.5/2.7	NSF/NSF
WLE-M2	pull/push	65.9/63.7	67.6/64.3	15.0/15.5	30.2/27.1	4.4/4.1	NSF/NSF
	exp,AV	65.4/64.7	67.0/65.5	16.8/19.9	33.5/31.2	4.0/3.4	-
	th	-	*61.4/61.4*	-	-	*4.4/4.4*	*NSF*
	exp,AV/th	-	1.09/1.07	-	-	0.90/0.77	-
WLD-M1	pull/push	56.7/58.8	61.7/62.3	14.2/18.4	214.5/244.2	4.0/3.2	BY/BY
WLD-M2	pull/push	56.0/54.4	64.2/56.5	13.0/17.0	237.9/139.0	4.3/3.2	BY/BY
	exp,AV	56.4/56.6	63.0/59.4	13.6/17.7	226.2/191.6	4.2/3.2	-
	th	*55.0/55.0*	-	-	-	*4.9/4.9*	*BY*
	exp,AV/th	*1.02/1.03*	-	-	-	*0.85/ 0.65*	-
WHD-M1	pull/push	110.3/107.8	116.9/119.3	17.8/29.9	157.6/159.7	6.2/3.6	BY/BY
WHD-M2	pull/push	109.5/114.2	118.4/119.3	18.6/33.6	203.5/220.0	5.9/3.4	BY/BY
	exp,AV	109.9/111.0	117.7/119.3	18.2/31.8	180.6/189.9	6.1/3.5	-
	th	*110.0/110.0*	-	-	-	*6.6/6.6*	*BY*
	exp,AV/th	*1.00/1.01*	-	-	-	*0.92/ 0.53*	-

exp,AV: average experimental values; th: theoretical values;
NSF: net section failure of strap-bracing ; BY: brace yielding

Table 3. Test results of monotonic tests on full-scale walls

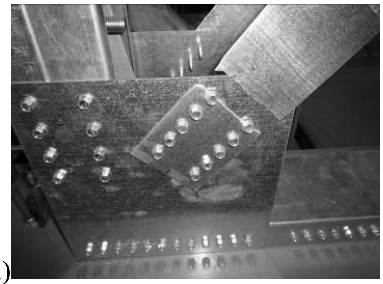
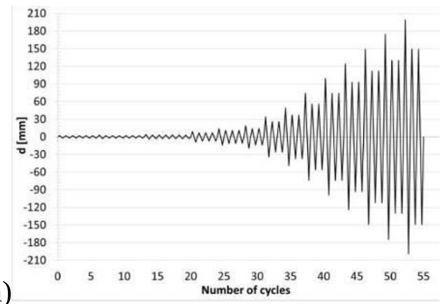
a)  b) 

Figure 5. Failure modes: a) net section failure, b) brace yielding

Cyclic tests

The cyclic tests were carried out by adopting a loading protocol known as "CUREE ordinary ground motions reversed cyclic load protocol" developed for wood walls by Krawinkler et al. and modified for CFS strap-braced stud walls by Velchev et al.. The cyclic loading test protocol consists of a series of stepwise increasing deformation cycles. The displacement amplitudes were defined starting from a reference deformation $\Delta = 2.667\Delta_y$, where Δ_y was defined as the displacement at the conventional elastic limit evaluated in the nominally identical monotonic wall tests. The cyclic protocol involved displacements at a rate of 0.5 mm/s, for displacements up to 9.97 mm, 7.36 mm e 7.27 mm for WLE, WLD and WHD walls respectively, and of 2.0 mm/s for displacement greater than those above mentioned. The adopted test protocol for WLE specimens is shown in Fig. 6a and the load (H) versus the measured displacement (d) curve together with the analyzed parameters for the WLE-C2 specimen is shown in Figure 6b. The results of the cyclic tests are shown in Table 4.

a) 

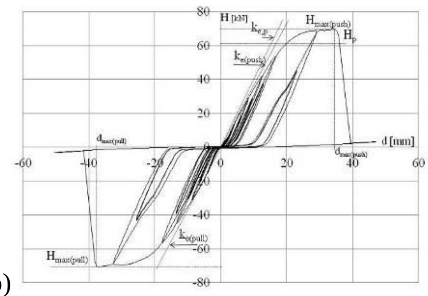
b) 

Figure 6. WLE specimens: a) cyclic protocol; b) load vs. displacement curve

Type		H_y [kN]	H_{max} [kN]	d_{max} [mm]	k_e [kN/mm]	failure mode
WLE-C1	pull/push	69.6/68.9	70.6/69.4	38.1/35.7	3.7/3.4	NSF/NSF
WLE-C2	pull/push	68.0/69.9	68.3/70.5	26.5/31.3	4.0/4.7	NSF/NSF
	exp,$_{AV}$	68.8/69.4	69.5/70.0	32.3/33.5	3.9/4.1	-
	th	-	*61.4/61.4*	-	*4.4/4.4*	*NSF*
	exp,$_{AV}$/th	-	1.13/1.14	-	0.88/0.92	-
WLD-C1	pull/push	58.7/59.8	63.1/64.4	176.2/165.5	3.8/4.0	NSF/NSF
WLD-C2	pull/push	58.7/60.0	66.6/64.9	141.2/144.8	4.6/4.5	NSF/NSF
	exp,$_{AV}$	58.7/59.9	64.9/64.7	158.7/155.2	4.2/4.3	-
	th	*55.0/55.0*	-	-	*4.9/4.9*	*BY*
	exp,$_{AV}$/th	1.07/1.09	-	-	0.86/0.87	-
WHD-C1	pull/push	116.7/116.0	124.0/124.2	197.0/221.0	5.7/7.7	NSF/BY
WHD-C2	pull/push	112.9/111.6	118.9/124.2	67.5/221.8	7.5/6.7	NSF/BY
	exp,$_{AV}$	114.8/113.8	121.5/124.2	132.3/221.4	6.6/7.2	-
	th	*110.0/110.0*	-	-	*6.6/6.6*	*BY*
	exp,$_{AV}$/th	1.04/1.03	-	-	1.00/1.09	-

exp,$_{AV}$: average experimental values; th: theoretical values;
NSF: net section failure of strap-bracing ; BY: brace yielding

Table 4: Test results of cyclic tests on full-scale walls.

The results show that the strength and stiffness recorded in the pushing phase with respect to the pulling phases have maximum differences of 4% and 18%, respectively, except a variation of 35% for the stiffness of WHD-C1 specimen. The ratios between the average experimental and theoretical values highlight that the experimental strengths are higher than the theoretical predictions with maximum difference of 14%, while the measured stiffness values are lower than the predicted parameters with a variation up to 14%. For all prototypes the observed collapse mode was the net section failure of diagonal straps, except for WHD wall specimens, which showed the brace yielding in the pushing phase.

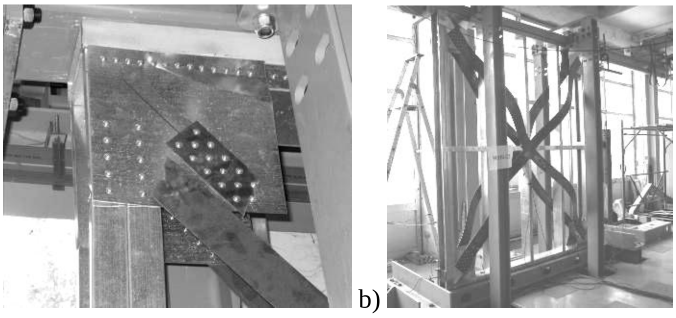

a) b)

Figure 7. Failure modes: a) net section failure for WLD-C1, brace yielding for WHD-C2.

Tests on material and components

The global lateral response of CFS strap-braced stud walls and the local behaviour of their components are strongly interrelated, therefore tests on materials, elementary connections and gussets - to - strap connections have been performed. In particular, since the CFS strap-braced stud walls behaviour is influenced by the design of frame-to-strap connections, which usually takes place through steel gussets, shear tests on connection prototypes reproducing the joints between gusset and strap-bracing were performed. The behaviour of the connections adopted for the three selected wall configurations (indicated with subscript 1) were investigated. Furthermore, three additional connection types for WLD and WHD systems, corresponding to different screw layouts in strap-bracing cross-section, were tested. Therefore, by naming A_{n1} and A_{n2} the minimum net areas defined by considering perpendicular cross-sections to strap-bracing axis and cross-sections obtained by a broken line, respectively, the following joint types for dissipative walls have been considered (Fig. 8): (1) connection configuration adopted in the selected walls, in which $A_{n1} < A_{n2}$; (2) connection with aligned screws arrangement, in which $A_{n1} < A_{n2}$; (3) connection with staggered screws, in which $A_{n1} = A_{n2}$; (4) connection with staggered screws, in which $A_{n1} > A_{n2}$. The phenomenon of "strain-rate" has been investigated only for the type 1 configurations. The examined configurations, the number of tests , the average failure loads ($F_{t,m}$) and stiffness ($k_{e,m}$) and the observed failure mechanisms are summarized in Table 5. The force-displacement curves obtained for the type 1 configurations (Fig. 9a) demonstrate that the CHD-1 specimens show the best response in terms of strength and stiffness, with average failure load values approximately twice the values obtained for the CLE-1 and CLD-1 specimens. Furthermore, the strength increases between 5% and 9% and the deformation capacity decreases between 50% and 65% as the test rate increases. As regards the connection response evaluation for different screw geometrical arrangements (Fig. 9b), the configurations do not play significant influence in terms of strength and stiffness, but the type 1 connections have larger deformation capability. For all tests the failure mechanism was screw tilting with subsequent net section failure of straps (Fig. 10).

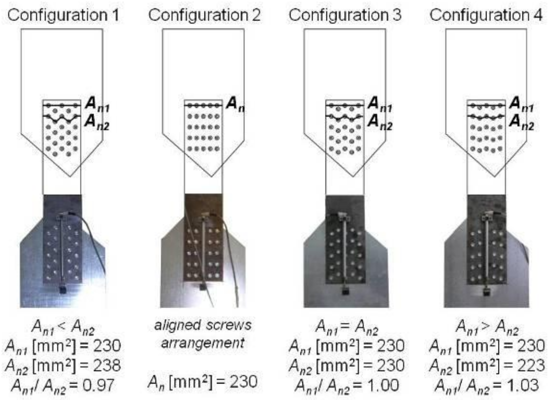
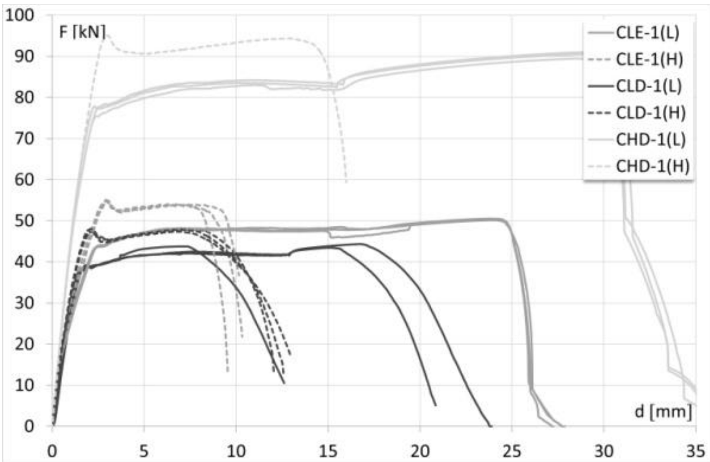
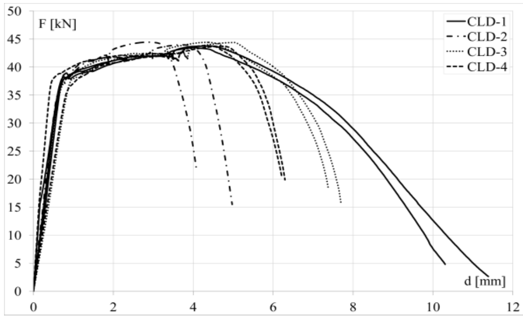


Figure 8. Test on gusset-to-strap connections



a)



b)

Figure 9. F-d curves for: a) type 1 configurations; b) CLD specimens

CLE-1 CLD-1 CHD-1

Figure 10. Failure modes for CLE-1, CLD-1 and CHD-1 specimens

type	plate type: steel	plate type: thick. [mm]	screw diameter [mm]	no. screws	conf.	test rate [mm/s]	no. tests	$F_{t,m}$ [kN]	$k_{e,m}$ [kN/mm]	failure mode
CLE	S350 GD+Z	1.5	6.3	10	CLE-1	0.05	3	50.4	38.1	T+NSF
						50	3	54.9	-	T+NSF
CLD	S350 GD+Z	1.5	4.8	15	CLD-1	0.05	3	43.8	58.7	T+NSF
						50	3	47.9	-	T+NSF
					CLD-2	0.05	2	44.2	59.1	T+NSF
	S235	2.0			CLD-3	0.05	2	44.4	56.5	T+NSF
					CLD-4	0.05	2	43.8	63.6	T+NSF
CHD	S350 GD+Z	1.5	6.3	25	CHD-1	0.05	3	90.3	166.4	T+NSF
						50	1	95.1	-	T+NSF
					CHD-2	0.05	2	84.5	119.1	T+NSF
	S235	2.0			CHD-3	0.05	2	84.9	113.7	T+NSF
					CHD-4	0.05	2	84.4	190.8	T+NSF

T: tilting of screw; NSF: net section failure of strap-bracing

Table 5: Test results on gusset-strap connection.

Evaluation of behaviour factors on the base of the experimental data

On the base of the results of both monotonic and cyclic wall tests, the behaviour factors for each investigated wall have been evaluated and then compared with the one provided by the AISI S213.

The behaviour factor has been defined as the product of the R_d (ductility) and R_o (overstrength) factors, as given in Uang (1991). In particular, The ductility-related force modification factor R_d can be evaluated as follows:

$$R_d = \sqrt{2\mu - 1} \text{ with } \mu = \frac{d_{\max}}{d_y}$$

where μ is the ductility; d_{max} and d_y are the maximum and the conventional elastic limit of the top wall displacement, respectively.

The displacement d_{max} has been defined as the displacement corresponding to the following limits of interstorey-drift (d/h, with h=2700 mm is the wall height): 1.5%, 2% and 7%. For the cases in which the wall collapse occurred for displacement lower than the given limits, d_{max} has been assumed as the displacement at the peak load. The limits of 1.5% and 2% are those provided by FEMA 356 (FEMA, 2000) for traditional concentrically braced structures at the Life Safety and Collapse Prevention limit states, respectively. On the other hand, the limit of 7% is the maximum displacement capacity obtained by shaking table tests (Isoda et al., 2007) on wooden shear walls, which represent a system similar to the investigated one.

The overstrength-related force modification factor R_o can be evaluated through the formulation provided by Mitchell et al. (2003):

$$R_o = R_{sd} \cdot R_{\Phi} \cdot R_{yield} \cdot R_{sh}$$

where R_{sd}= H_c/H_d, with H_c and H_d design wall resistance and seismic demand, respectively; R_{Φ}= H_{yn}/H_c, with H_{yn} nominal yielding resistance; R_{yield}= H_y/H_{yn}, with H_y experimental yielding resistance (average); R_{sh}= $H_{\%}/H_y$, with $H_{\%}$ wall resistance at relevant inter-story drift.

Tables 6 and 7 show the values of the behaviour factor obtained by the experimental results. In particular, for WLE walls d_{max}/h result always less than 1.5%, so the evaluation of q is limited to the case d=d_{max}. In the case of WLE walls (Table 6), it can be noted that the behaviour factor values proposed by AISI S213 for Conventional construction category (q=1.6) is always smaller than those experimentally obtained (q=2.0÷2.2). As far as WLD and WHD walls are concerned, the value provided by AISI S213 in case of Limited ductility braced walls (q=2.5) represents a lower limit of the obtained behaviour factors (q=2.5÷3.0 for 1.5%, q=3.0÷4.3 for 2%, q=6.4÷8.2 for 7%).

Test	R_d	R_o	q
WLE-M1	1.74	1.15	2.00
WLE-M2	1.74	1.17	2.04
WLE-C1	1.80	1.21	2.19
WLE-C2	1.73	1.20	2.08

Table 6: Behaviour factor for WLE

	1.5% interstorey drift			2% interstorey drift			7% interstorey drift		
Test	R_d	R_o	q	R_d	R_o	Q	R_d	R_o	q
WLD-M1	2.2	1.4	3.1	2.6	1.4	3.7	5.1	1.5	7.8
WLD-M2	2.3	1.4	3.2	2.7	1.4	3.9	5.3	1.6	8.2
WLD-C1	2.2	1.5	3.3	2.6	1.5	3.9	5.1	1.5	7.8
WLD-C2	2.4	1.5	3.7	2.9	1.5	4.3	4.8	1.6	7.8
WHD-M1	1.9	1.4	2.6	2.3	1.4	3.1		(*)	
WHD-M2	1.9	1.4	2.5	2.2	1.4	3.1	4.4	1.5	6.4
WHD-C1	2.0	1.5	2.9	2.3	1.5	3.4	4.6	1.5	7.0
WHD-C2 (Pull)	2.1	1.4	3.0	2.5	1.4	3.6		(**)	
WHD-C2 (Push)	2.0	1.4	2.8	2.4	1.4	3.4	4.7	1.4	6.6

(*) The test was interrupted because of the occurrence of local buckling of the tracks;
(**) The diagonal net area collapse before reaching the limit of 7%.

Table 7: Behaviour factor for WLD and WHD

Conclusions

An experimental investigation for the evaluation of the seismic behaviour of CFS strap-braced stud walls has been presented and discussed in the current paper. The obtained results from the wall and connections tests show a satisfactory response in terms of strength, deformation capacity and stiffness. In particular, a good correspondence between wall experimental and theoretical predicted values is highlighted in terms of strength (maximum gap of 16%). As a further development, an extended numerical study including non-linear dynamic analysis should be performed for a more accurate estimation of the behaviour factor.

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Appendix. – Notation

A_n net area;
BY is the brace yielding;
d is the displacement;
d_y is the displacement at the conventional elastic limit;
d_{max} maximum displacement;
Δ_y displacement at the conventional elastic limit;
H is the acting load;
H_c is the design wall resistance;
H_d is the seismic demand;
H_y is the yielding strength;
H_{yn} is the nominal yielding strength;
H_{max} is the maximum strength;
$H_{\%}$ wall resistance at relevant inter-story drift;
k_e conventional elastic stiffness;
$F_{t,m}$ average failure load;
$k_{e,m}$ average stiffness;
μ is the ductility;
NSF is the net section failure of strap-bracing;
q behaviour factor;
R_d is the ductility factor;
R_o is the overstrength-related force modification factor;
T is the tilting of screw
WHD stands for Dissipative Heavy Wall
WLE stands for Elastic Light Wall
WLD stands for Dissipative Light Wall