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SEISMIC BEHAVIOUR OF "ALL-STEEL" CFS STRUCTURES:
Experimental Tests

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INTRODUCTION

The Cold-Formed Steel (CFS) structures are able to ensure a good structural response in seismic areas. Among them, the stick-built constructions raise a considerable interest in recent studies. In these structures, the lateral load bearing systems are CFS stud walls, that are generally realized with a frame in CFS profiles that can be braced by light gauge steel straps installed in an X configuration. In this case, the "all steel" design methodology can be adopted and the lateral resisting system is assured by the CFS strap-braced stud walls.
The unconventionality of CFS structures has motivated, in recent times, the experimental characterization carried out by many international research groups. The investigations included several aspects that affect the seismic behaviour of CFS strap-braced stud walls. In particular, the monotonic and cyclic response of these systems has been examined by Adham *et al.* [1], Serrette & Ogunfunmi [2], Gad *et al.* [3], Fulop & Dubina [4], Tian *et al.* [5], Al-Kharat & Rogers [6], Kim *et al.* [7], Casafont *et al.* [8], Moghimi & Ronagh [9] and Velchev *et al.* [10] which have observed a satisfactory experimental behaviour in terms of energy dissipation, stiffness, strength and deformation capacity. The contribution of the frame without bracing has been analyzed by many studies [2], [5] and [7]. Specifically, Tian *et al.* [5] estimated that a frame without any bracing system has a lateral strength 5% less than the braced one. The frame-to-strap connections have been investigated in [3], [4], [6], [8] and [10]. These studies have concluded that the connection behaviour highly influences the failure load and mechanism of the tested walls. In particular, Casafont *et al.* [8] demonstrated that the screws are the preferable connection type in the seismic design, because their small diameter involves a net-section area greater than other fastener types. Moreover, taking into account that gypsum sheathing panels are usually adopted as wall finishing, the effect of those panels at varying of their thickness on the in-plane shear response of CFS strap-braced stud walls has been evaluated in [1], [2], [3] and [9]. The results of these studies demonstrated a significant contribution to shear capacity provided by sheathing panels. In particular, Gad *et al.* [3] observed that the overall stiffness and strength of the system can be obtained as the sum of the individual contributions of plasterboard and strap braces. The effect of loading type (monotonic and cyclic) on the wall lateral behaviour has been investigated in [1], [3], [4], [5], [6], [8], [9] and [10]. These researches observed a highly non-linear behaviour and a cyclic response characterized by the phenomenon of "pinching" and, therefore, by a reduced energy dissipation capacity. Furthermore, the stiffness and strength degradation becomes larger as the number of cycles increases, as highlighted in [1] and [8].
As an attempt to provide a contribution for the knowledge improvement of this system, a theoretical and experimental study has been carried out at University of Naples Federico II within the research project RELUIS-DPC 2010-2013. The research program has been articulated in two main phases. The experimental phase has been devoted to the evaluation of the local and global behaviour of CFS strap-braced stud walls by an experimental study. In the theoretical phase, seismic design criteria have been deeply investigated [11]. This paper mainly presents the results of the experimental phase.

1 TEST PROGRAM

The first phase of research program has been devoted to the development of study cases representative of typical seismic applications, in order to define the prototypes to be tested. With the purpose to investigate a large number of possible applications, three buildings located in different seismic area have been designed and for each of them the main lateral resisting system has been defined. In particular, these last are composed of CFS strap-braced stud walls that have been designed according to elastic or dissipative design approaches. Therefore, three configurations have been defined: elastic light (WLE), dissipative light (WLD) and heavy (WHD) walls. The WLE typology represents the seismic force resisting system of a single-story building in low-medium seismic area and all wall components have been designed according to an elastic approach. Instead, the dissipative wall configurations (WLD and WHD) have been designed and detailed by adopting capacity design principles, in such a way to ensure a ductile performance by promoting the brace yielding. The WLD system corresponds to the same conditions of WLE, while the WHD wall represents the lateral resisting system of three-story building in high-medium seismic area. The lateral response of these systems has been investigated by testing each of three selected configuration by two monotonic and two cyclic tests for a total of twelve tests on full-scale wall specimens in size of 2.4 m x 2.7 m. Moreover, taking into account that materials and components affect the wall seismic global response in terms of lateral resistance, stiffness and ductility, the component response has been 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-bracing. The experimental campaign has been carried out at the Department of Structures for Engineering and Architecture of the University of Naples Federico II.

2 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.

The material coupons of straps and frame members have been subjected to conventional tension tests according to EN ISO 6892-1: 2009 [12]. In particular, tests have been performed on three specimen types S235-2.0, S350-1.5, S350-3.0, characterized by steel grades S235 with nominal yield stress f_y= 235 MPa and ultimate stress f_t = 360 MPa, S350GD + Z with f_y = 350 MPa and f_t = 420 MPa and thicknesses 2.0 mm, 1.5 mm, 3.0 mm, respectively. In order to investigate the phenomenon of "strain-rate", tests at low rate (0.05 mm/s) and high rate (50 mm/s) have been carried out for each specimen type. The effects of “strain rate” have been assessed to observe how the behaviour can change by increasing the test rate. This last aspect has not been investigated in the quasi-static tests on wall specimens. *Table 1* shows the average values of measured yield ($f_{y,m}$) and ultimate stress ($f_{t,m}$) for each test rate, the ratio between nominal and average values and the ratio between average values at low (L) and high (H) rate. As regard the tests at low rate, the experimental values of the yield stress are larger than the nominal values (28%, 2%, 4% for S235-2.0, S350-1.5, S350-3.0, respectively), while the results in terms of ultimate stress record a moderate increase for S235-2.0 and S350-3.0 specimens (2% e 1%, respectively) and a reduction of 3% for S350-1.5 specimens. The "strain-rate" effect produces an increment of the strength. In particular, the yield and ultimate stresses increases between 5% and 7%, respectively, as the test rate increases.

Table 1. Tests results on material

Type	Steel grade	Thickness.	v = 0.05 mm/s					v = 50 mm/s				
		[mm]	n. tests	$f_{y,m}$ [MPa]	$f_{t,m}$ [MPa]	$f_{y,m}/f_y$	$f_{t,m}/f_t$	n. tests	$f_{y,m}$ [MPa]	$f_{t,m}$ [MPa]	$f_{y,m(H)}/f_{y,m(L)}$	$f_{t,m(H)}/f_{t,m(L)}$
S235-2.0	S235	2.0	3	302	366	1.28	1.02	2	323	389	1.07	1.06
S350-1.5	S350GD+Z	1.5	3	355	409	1.02	0.97	3	380	430	1.07	1.05
S350-3.0	S350GD+Z	3.0	3	364	425	1.04	1.01	3	387	454	1.06	1.07

The elementary connections between frame and strap-bracing have been tested. The shear tests have been carried out according to the procedure described in ECCS TC7 TWG 7:10 [13]. Three connection configurations, corresponding to each of the investigated wall typologies, have been considered: (SLE) connections between 1.5 mm thick S350GD + Z steel plates with 6.3 mm diameter self-drilling screws; (SLD) connections between 1.5 mm thick S350GD + Z and 2.0 mm thick S235 steel plates with 4.8 mm diameter self-drilling screws; (SHD) connections between 1.5 mm thick S350GD + Z and 2.0 mm thick S235 steel plates with 6.3 mm diameter self-drilling screws. The results in terms of average failure load ($F_{t,m}$) and stiffness ($k_{e,m}$) and the failure mechanisms are listed in *Table 2*. The results show that the average failure loads of SLE and SHD types are greater than the failure value of SLD specimen, respectively by 17% and 37%. In addition, the force-displacement curves (*Fig. 1a*) show a very limited deformation capacity of SLD specimens. The different behaviour is due to dissimilar failure mechanisms: tilting and pull-out of screws for SLE and SHD configurations and shear failure for SLD connections (*Fig. 1b*).

Table 2. Test results on elementary connection

Type	Plate type		Screw		N. tests	$F_{t,m}$	$k_{e,m}$	Failure mode
	steel	thickness [mm]	Type	diameter [mm]		[kN]	[kN/mm]	
SLE	S350GD+Z	1.5	AB 04 63 040	6.3	3	7.6	4.0	T + PO
SLD	S350GD+Z	1.5	CI 01 48 016	4.8	3	6.5	3.4	S
	S235	2.0						
SHD	S350GD+Z	1.5	AB 04 63 040	6.3	2	8.9	4.6	T + PO
	S235	2.0						

T: tilting; PO: pull-out; S: shear

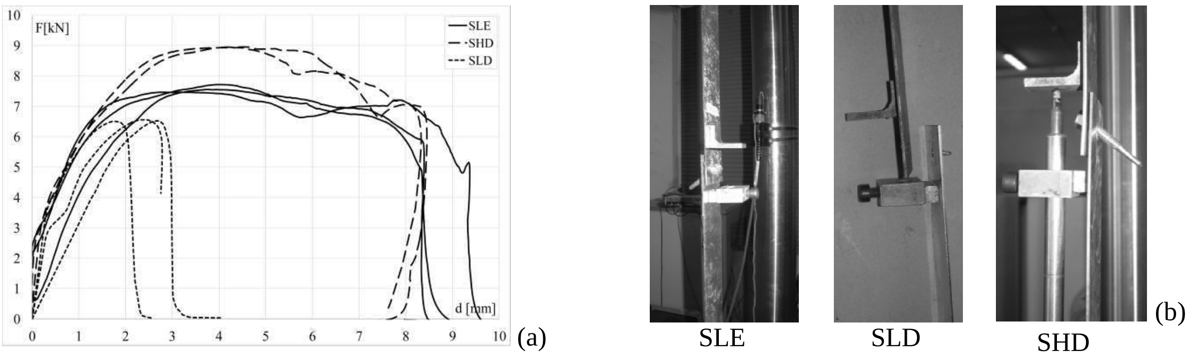

Fig. 1. Test on elementary connection: a) *F-d* curves; b) Failure modes for SLE, SLD and SHD specimens.

The CFS strap-braced stud walls behaviour is particularly influenced by the design of frame-to-strap connections, which usually takes place through steel gussets. For this reason, the local response evaluation of the investigated X-braced CFS systems has been completed with shear tests on connection prototypes reproducing the joints between gusset and strap-bracing. The behaviour of the connections adopted for the three selected wall configurations (indicated with subscript 1) has been investigated. Furthermore, three additional connection types for WLD and WHD systems, corresponding to different screw layouts in strap-bracing cross-section, have been 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. 2a*): (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 3*. The force-displacement curves obtained for the type 1 configurations (*Fig. 2b*) 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. 2c*), 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. 2d*).

Table 3. Test results on gusset-strap connection

Type	Plate type steel	Plate type thickness. [mm]	Screw diameter [mm]	N. screws	Configuration	Test rate [mm/s]	N. tests	$F_{t,m}$ [kN]	$k_{e,m}$ [kN/mm]	Failure mode
CLE	S350GD+Z	1.5	6.3	10	CLE-1	0.05	3	50.4	38.9	T+NSF
						50	3	54.9	-	T+NSF
CLD	S350GD+Z	1.5	4.8	15	CLD-1	0.05	3	43.8	59.7	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	51.8	T+NSF
					CLD-4	0.05	2	43.8	66.4	T+NSF
CHD	S350GD+Z	1.5	6.3	25	CHD-1	0.05	3	90.3	153.4	T+NSF
						50	1	95.1	-	T+NSF
					CHD-2	0.05	2	84.4	134.0	T+NSF
	S235	2.0			CHD-3	0.05	2	84.9	125.4	T+NSF
					CHD-4	0.05	2	84.4	187.6	T+NSF

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

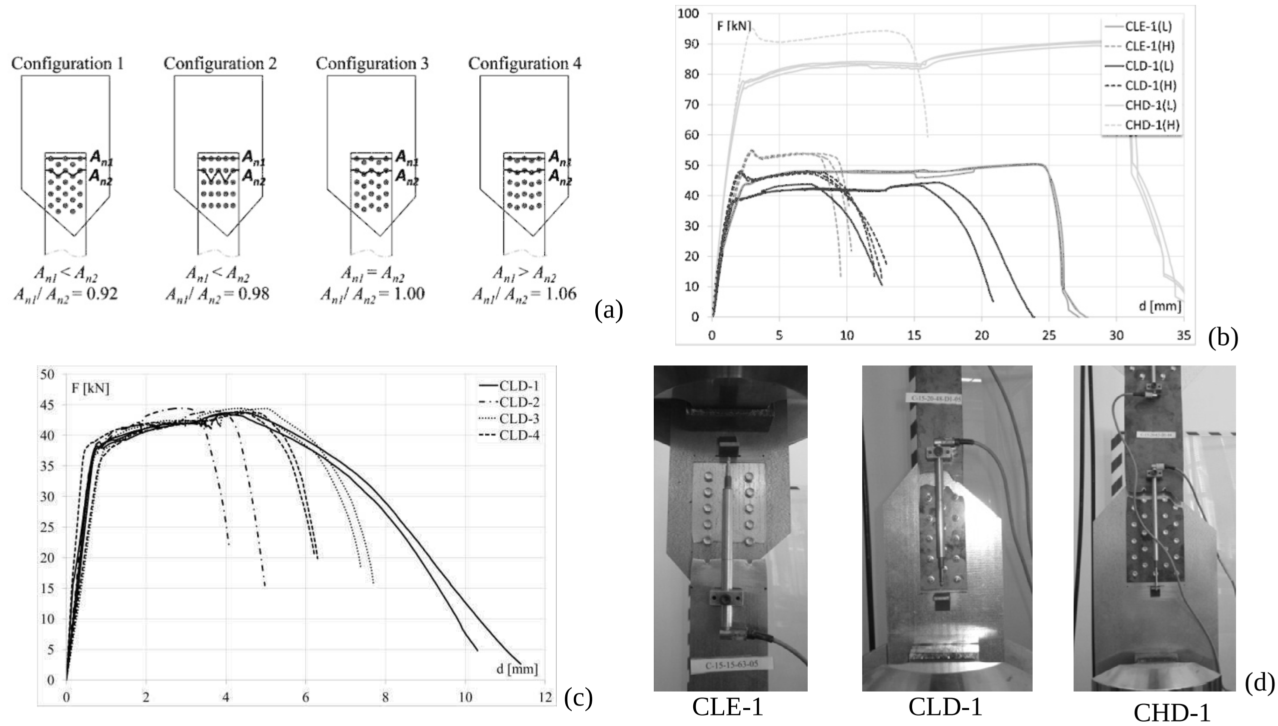

Fig. 2. Test on gusset-to-strap connections: a) CHD specimens; b) *F-d* curves for type 1 configurations; c) *F-d* curves for CLD specimens; d) failure modes for CLE-1, CLD-1 and CHD-1 specimens.

3 TESTS ON FULL-SCALE CFS STRAP-BRACED STUD WALLS

The lateral in-plane behaviour of the selected wall configurations (WLE, WLD, WHD) has been investigated by means of 12 physical tests, including 6 monotonic tests and 6 cyclic tests on full-scale 2400 mm long and 2700 mm high wall specimens. The wall framing is 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, they are composed by double C-sections screwed back-to-back. In order to reduce the unbraced length of the chord and interior studs, flat straps are placed at the mid-height of the wall specimens and are connected to blocking members at the ends of walls. The local buckling phenomena of tracks are avoided by reinforcing the ends of members with C-section profiles and then by creating box sections. Uplift forces are transferred from the chord studs to the testing frame by hold-down devices made with S700 steel grade, each of which is connected to the studs by four M16 8.8 grade bolts and to the bottom beam of the testing frame by one M24 8.8 grade bolt. The upper and bottom tracks of the tested walls are connected respectively to the loading (top) and bottom beams of the testing frame by M8 8.8 grade bolts, which are used as shear connections and are spaced at 300 mm on the center. The wall specimens are completed with strap braces installed in an X configuration on both sides and connected to the wall framing by gusset plates. All the connections are made with self-drilling screws. All the steel members are fabricated by S350GD+Z steel grade, except the diagonal straps of dissipative systems, which are made with S235 steel grade. *Table 4* lists the nominal design dimensions and material properties of the tested wall components. Schematic drawings of the WHD wall configuration is provided in *Fig. 3*. Tests on full-scale wall specimens were carried out by using a specifically designed testing frame for in-plane shear loading. Racking loads were transmitted to the upper wall track by means of a steel beam with rectangular hollow section. The wall prototype was constrained to the laboratory strong floor by the bottom beam of testing frame. 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 have been performed by using a hydraulic actuator having 500 mm stroke displacement and 500 kN load capacity. A sliding-hinge has been placed between the actuator and the tested wall in order to avoid additional vertical load components. Eight LVDTs have been used to measure the specimens displacements, as shown in *Fig. 4*. In particular, three LVDTs (W_1, W_2 e W_3) have been installed to record the horizontal displacements and two LVDTs (W_4, W_5) for the vertical displacements. The local deformations of the diagonal straps have been recorded by means of two strain-gauges (S_i). A load cell has been used to measure the racking loads.

Table 4. Nominal design dimensions and material properties of the tested wall components

	WLE		WLD		WHD	
	Section [mm]	Grade	Section [mm]	Grade	Section [mm]	Grade
Studs	C150x50x20x1.5[1]	S350	C150x50x20x1.5[1]	S350	C150x50x20x3.0[1]	S350
Tracks	U153x50x1.5[2]	S350	U153x50x1.5[2]	S350	U153x50x1.5[2]	S350
Diagonal straps	90x1.5[3]	S350	70x2.0[3]	S235	140x2.0[3]	S235
Gusset plates	270x270x1.5[3]	S350	290x290x1.5[3]	S350	365x365x1.5[3]	S350
Track reinforcements	C150x50x20x1.5[1]	S350	C150x50x20x1.5[1]	S350	C150x50x20x3.0[1]	S350
Blocking	C150x50x20x1.5[1]	S350	C150x50x20x1.5[1]	S350	C150x50x20x3.0[1]	S350
Flat straps	50x1.5[3]	S350	50x1.5[3]	S350	50x1.5[3]	S350

[1]*C-section: outside-to-outside web depth x outside-to-outside flange size x outside-to-outside lip size x thickness;*
[2]*U-section: outside-to-outside web depth x outside-to-outside flange size x thickness;* [3]*Plate: width x thickness*

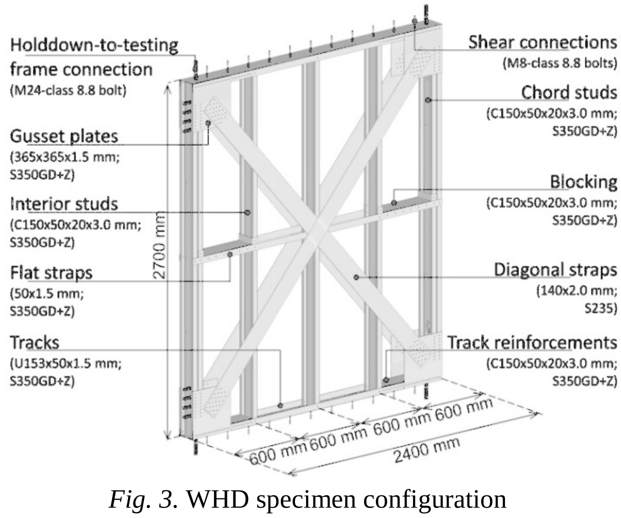

Fig. 3. WHD specimen configuration

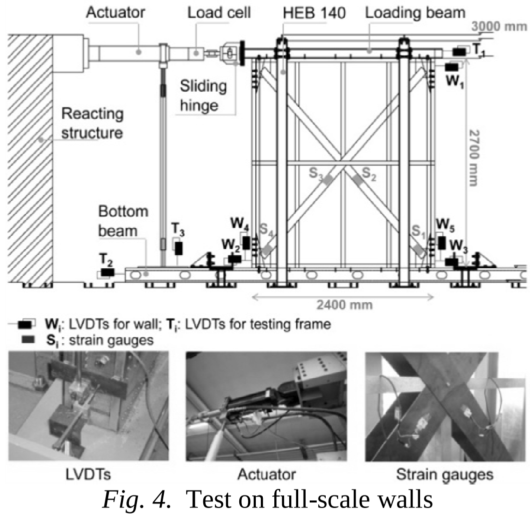

Fig. 4. Test on full-scale walls

3.1 Monotonic tests

In the monotonic loading regime, the tests have been performed by applying a loading protocol organized in two phases. In the first phase the wall specimens have been pulled and in the second phase they have been 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. 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 5*. Moreover, the theoretical predicted values of the strength ($H_{y,p}$ and $H_{max,p}$) and stiffness ($k_{e,p}$), which are evaluated using the experimental mechanical properties, are provided. *Figure 5* shows the acting loads (H) vs top wall displacements (d) curve for the WHD-M2 prototype and the measured and predicted parameters, which are used to evaluate the structural response. Test results reveal variations of maximum strength contained within 14% between the pulling and pushing phases, while the conventional elastic stiffness records significant decreases up to 42% in the pushing phase, due to the occurrence of local damages of some wall components in the previous pulling phase. For the WLE configurations the collapse was governed by the net section failure of diagonal straps (*Fig. 6a*), while WLD and WHD specimens showed the brace yielding without reaching the rupture, in accordance with the maximum stroke of the actuator (*Fig. 6b*). The results highlight variations up to 10% between the experimental and theoretical strengths.

Table 5. Test results of monotonic tests on full-scale walls

Type		H_y ($H_{y,p}$)	H_{max} ($H_{max,p}$)	d_y	d_{max}	k_e ($k_{e,p}$)	Failure mode
		[kN]	[kN]	[mm]	[mm]	[kN/mm]	
WLE-M1	pull/push	64.9/65.6	**66.3/66.6**	18.7/24.2	36.7/35.3	3.5/2.7	NSF/NSF
WLE-M2	pull/push	65.9/63.7	**67.6/64.3**	15.1/15.6	30.2/27.1	4.4/4.1	NSF/NSF
	theoretical	62.0	**61.4**	-	-	4.4	NSF
	Exper./theor.	-	*1.05 ÷ 1.10*	-	-	*0.61 ÷ 1.00*	
WLD-M1	pull/push	**56.7/58.8**	61.7/62.3	14.1/18.6	214.5/244.2	4.0/3.2	BY/BY
WLD-M2	pull/push	**56.0/54.4**	64.2/56.5	13.1/17.0	237.9/139.0	4.3/3.2	BY/BY
	theoretical	**55.0**	57.6	-	-	4.9	BY
	Exper./theor	*0.99 ÷ 1.07*	-	-	-	*0.65 ÷ 0.88*	
WHD-M1	pull/push	**110.3/107.8**	116.9/119.3	17.7/29.7	157.6/159.7	6.2/3.6	BY/BY
WHD-M2	pull/push	**109.5/114.2**	118.4/119.3	18.6/40.1	203.5/217.6	5.9/3.4	BY/BY
	theoretical	**110.0**	115.5	-	-	6.6	BY
	Exper./theor	*0.98 ÷ 1.04*	-	-	-	*0.52 ÷ 0.94*	

NSF: net section failure of strap-bracing ; BY: brace yielding

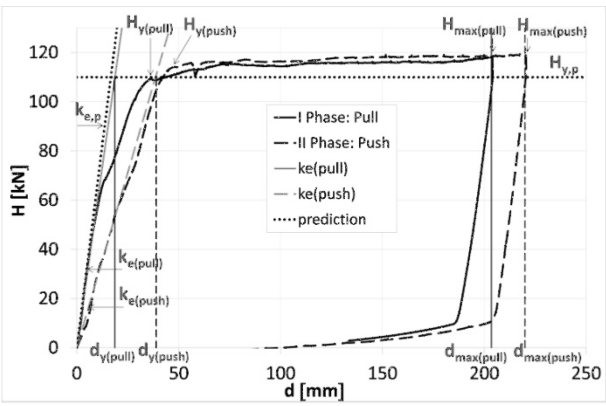

Fig. 5. Monotonic test on WHD-M2 specimen: load vs. displacement curve

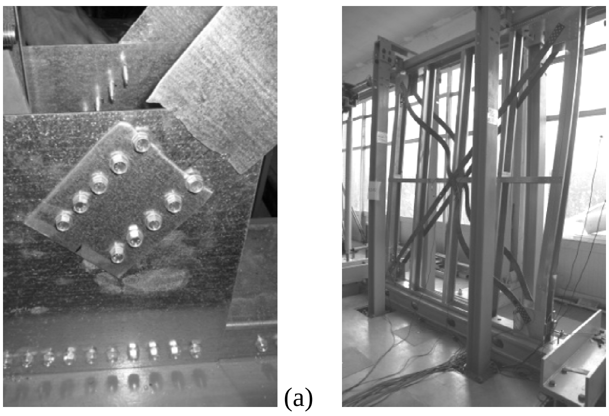
(a) (b)

Fig. 6. Monotonic tests on walls: a) net section failure for WLE-M1; b) brace yielding for WLD-M1

3.2 Cyclic tests

The cyclic tests have been 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.* [14] and modified for CFS strap-braced stud walls by Velchev *et al*. [10]. The cyclic loading test protocol consists of a series of stepwise increasing deformation cycles. The displacement amplitudes have been defined starting from a reference deformation, that is $\Delta = 2.667\Delta_y$, where Δ_y is the displacement at the conventional elastic limit evaluated in the monotonic tests on wall specimens. 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 mentioned above. The adopted test protocol for WLE specimens is shown in *Fig. 7*. *Figure 8* provides the acting loads (*H*) versus the measured displacements (*d*) curve and the analyzed parameters for the WLD-C2 specimen. The results of the cyclic tests are shown in *Table 6*. The results show that the strength and stiffness recorded for the two loading directions have maximum differences of 4% and 15%, respectively, except a variation of 26% for the stiffness of WHD-C1 specimen. For all prototypes the observed collapse mode has been the net section failure of diagonal straps, except for WHD wall specimens, which have showed the brace yielding in the pushing phase. The results highlight variations up to 16% and 23% between the experimental and theoretical values for strengths and stiffness, respectively. The comparison between the monotonic and cyclic responses in terms of strength and stiffness is quantified with variations contained within 12% and 17%, respectively.

Table 6. Test results of cyclic tests on full-scale walls

Tipologia		H_y ($H_{y,p}$) [kN]	H_{max} ($H_{max,p}$) [kN]	d_{max} [mm]	k_e ($k_{e,p}$) [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
	theoretical	62.0	**61.4**	-	4.4	NSF
	Exper./theor.	-	*1.11 ÷ 1.15*	-	*0.77 ÷ 1.07*	
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
	theoretical	55.0	**57.6**	-	4.9	BY
	Exper./theor.	-	*1.10 ÷ 1.16*	-	*0.78 ÷ 0.94*	
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
	theoretical	110.0	**115.5**	-	6.6	BY
	Exper./theor.	-	*1.03 ÷ 1.08*	-	*0.86 ÷ 1.17*	

NSF: net section failure of strap-bracing ; BY: brace yielding

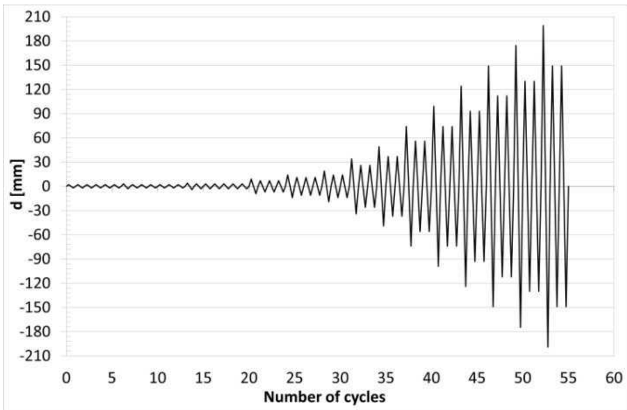

Fig. 7. Cyclic protocol for WLE specimens

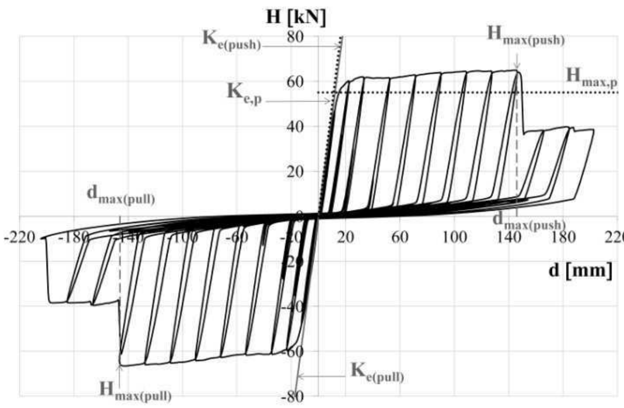

Fig. 8. Cyclic test on WLD-C2 specimen: load vs. displacement curve

4 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%). These results can be considered as a reference for theoretical studies aimed at defining seismic design criteria for the investigated systems.

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