Thin-Walled Structures 84 (2014) 275–288

Contents lists available at ScienceDirect

Thin-Walled Structures

journal homepage: www.elsevier.com/locate/tws

Testing CFS structures: The new school BFS in Naples

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ARTICLE INFO

Article history:
Received 11 November 2013
Received in revised form
11 June 2014
Accepted 11 June 2014
Available online 2 August 2014

Keywords:
Cold-formed steel structures
Connection tests
Hold-down device
Panel tests
Seismic behavior
Shear wall tests

ABSTRACT

The potentiality of cold-formed steel structures (CFS) in terms of lightness, rapid on-site erection and high structural performance is spreading the use of the technology in the most industrialized countries. In particular, the overall recognized capability to assure a good structural response in high seismic areas is allowing the adoption in some of the most conservative communities. On the other hand, despite the many ongoing research in different countries, the current seismic design codes are often inadequate if compared to the evolutionary process. In this context, the new British Force School (BFS) represents one of the first cold-formed steel structures realized in Italy and, therefore, it required a long and complex verification process. Indeed, the seismic performance at the global and local scale had to be verified during the construction phase. This paper focuses on the evaluation of lateral response of the sheathed cold-formed steel (SCFS) shear walls, which represent the seismic resisting system of this structural typology through the presentation and discussion of the experimental campaign carried out to support the design.

1. Introduction

Light weight steel structures are spreading around the most industrialized country thanks to their main futures such as lightness, short execution times and high structural performance. Moreover, the recognized capability to provide a good structural response in high seismic areas is increasing the adoption of these systems also in more conservative communities as Italy. Over the last two decades many theoretical and experimental studies have been addressed to capture the complex seismic behavior of cold formed steel (CFS) structures and to improve the current calculation models and design codes. As a result, at the moment, only the AISI S213-07/S1-09 [1] is specifically addressed to design the lateral force resisting systems of cold-formed steel framing to resist wind and seismic forces. This standard is based on research undertaken at Virginia Polytechnic Institute, State University, Santa Clara University and McGill University under the supervision of professor Serrette, Dolan and Rogers, respectively. Both the wind and seismic provisions in AISI S213 assume a shear wall with steel stud chords, infill steel studs and attached sheathing acting as a shear element. Design data are provided including factored shear resistances, the seismic ductility (R_d) and overstrength (R_o) force modification factors. The data are referred to wall sheathed by wood-based panels (CSP, DFP or OSB in various thicknesses) and 12.5 mm thick gypsum panels. A deflection equation is also provided for wood sheathed walls. Building height limitations are imposed for the seismic design and these limitations are a function of site specific seismic data. The AISI S213 provisions are based on capacity based design criteria. This approach provides a fuse element in the seismic force resisting system which must be able to carry seismic loads over extensive inelastic displacements without sudden failure. For sheathed shear walls, these last themselves can be thought of as the fuse elements, in which the sheathing-to-steel framing connections failure allow a global ductile behavior. The standard is intended for adoption and use in United States, Canada and Mexico and taking into account all the geometrical and material limitations, the provisions cannot be readily adopted in European countries. Therefore, the seismic design of CFS structures in Europe is still a concern [2]. Designers have to deal with a lack in codification and new constructions can pass through a long verification process. This has been the case of a school recently realized in Italy and analyzed in this paper. In particular, this work presents the experimental campaign carried out to characterize the seismic behavior of the British Force School (BFS), a foundation and primary school, that was designed and realized by the present authors for the Defense Estate in Lago Patria, Naples between 2009 and 2011. It covers 3000 m^2 and is organized in eight independent buildings, six of which are one-story stick-built constructions (Fig. 1). The description of the design and execution phases with main regards to the structural aspects are extensively treated in Fiorino et al. [3]. The structural design of the stick-built buildings was carried out following the "sheathing-braced" approach, that considers the contribution provided by the sheathing panels to the wall studs to resist to horizontal actions. Moreover, these lasts

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http://dx.doi.org/10.1016/j.tws.2014.06.006
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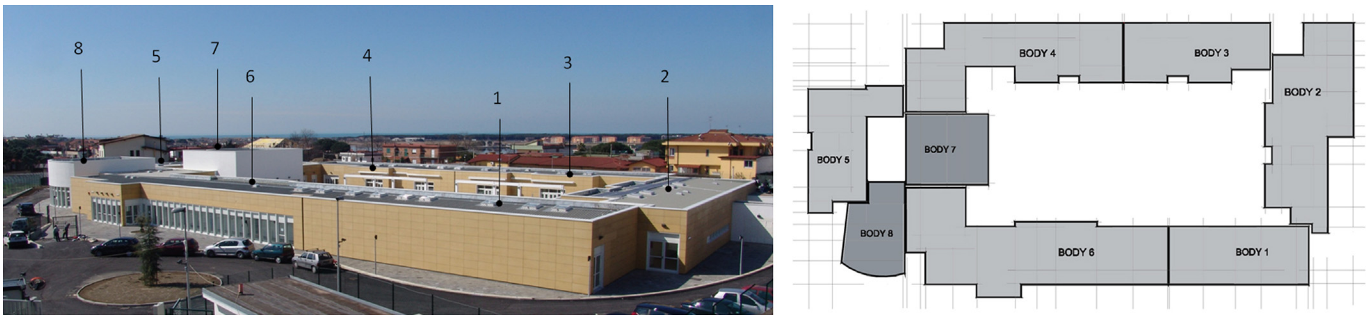


Fig. 1. Perspective view of the realized school and building layout.

that act on the lateral-resisting walls were evaluated according to the "Segment method" [4], for which only the wall segments delimited between two successive openings were considered effective. Those lateral resisting wall segments were under investigation [5].

Hereafter, the experimental campaign carried out to support the design and the inspection is presented. In particular, the results of two on-site full-scale wall tests as well as an experimental characterization of the most important structural wall components such as screws, sheathing panels, sheathing-to-steel frame screw connections and hold-down devices are discussed.

2. Testing program

The attention of the testing program was mainly focused on the in-plane lateral response of walls, which represent the seismic force resisting systems of stick-built constructions. The testing program included two tests on full scale walls and a large number of components tests. In particular, in order to investigate the global response, full scale tests on two identical $4.80 \times 3.95\ m^2$ walls have been carried out, and at the component scale the following tests were performed: 40 shear tests on screws, 30 shear tests on OSB panels, 44 tests on sheathing-to-frame connections and 10 tests on hold-down devices. Table 1 summarizes the features of all the wall components while the test matrix is presented in Table 2.

The two tested walls were made with studs having $150 \times 50 \times 20 \times 1.50$ mm (outside-to-outside web depth × outside-to-outside flange size × outside-to-outside lip size × thickness) lipped channel sections spaced at 600 mm on the center. The studs were connected at the ends to $152 \times 40 \times 1.50$ mm (outside-to-outside web depth × outside-to-outside flange size × thickness) unlipped channel section wall tracks. All steel members were fabricated by S350GD+Z steel grade with basic yield strength equal to 350 MPa (f_y) and nominal tensile strength 420 MPa (f_u). Steel frames were sheathed with vertically oriented 9.0 mm thick type 3 oriented strand board (OSB) panels on both faces, characterized by nominal shear strength higher than 3.00 MPa and shear modulus higher than 750 MPa. Steel frames were connected to the sheathings by 4.2 mm diameter bugle-head self-drilling screws spaced at 100 mm at the perimeter and 300 mm in the field. In order to avoid buckling phenomena and any wall overturning, back-to-back coupled studs and adequately designed hold-down devices were placed at the ends of each shear wall segment. In particular, the hold-down devices were very stiff in the longitudinal direction to limit uplift and were made with S700 steel grade, connected to the studs by four M16 bolts and to the foundation by one HIT RE 500 with 5.8 class M24 HAS adhesive-bonded anchors by HILTI [6]. Shear anchors made by M8 HST mechanical anchors were placed along the bottom track at regular spacing of 200 mm. The steel-to-steel connections were realized by 4.8 mm diameter lath head screws and 6.3 mm hex washer head screws. In order to reproduce perfectly the boundary condition within the realized building an acoustic insulation pad was placed between the bottom track and the concrete foundation.

3. Wall tests

3.1. Literature review of wall tests

The seismic behavior of CFS structures sheathed with panels is influenced by the response of shear walls, which are characterized by a highly non-linear structural response [8]. In order to assess the seismic performance of sheathed cold-formed steel (SCFS) structures, several experimental and/or numerical research programs have been carried out on different wall configurations. In the last 30 years, more than 600 tests have been carried out on walls with height ranging between 2.4 m and 3.60 m and width between 1.2 and 7.3 m, and sheathed by cement plaster (CP), chipboard (CHI), fiberbond wallboard (FBW), gypsum sheathing board (GSB), gypsum wallboard (GWB), oriented strand board (OSB), plywood (PLY), steel corrugated sheet (SCS), steel sheet sheathing (SSS), steel flat strap X-bracing (X-B) or calcium silicate board (CSB). In few cases, also walls on two stories have been tested. In particular 348 monotonic tests and 296 cyclic tests have been carried out as summarized in Table 3. Some of the previous tests have been at the basis of the development of current design codes as the *Uniform Building Code – Edition 1997* (International Conference of Building Officials. Whittier, CA. USA, [9]), *International Building Code - Edition 2012* (International Code Council, Inc. Falls Church, VA, USA, [10]), *National Building Code of Canada. Edition 2010* (National Research Council of Canada, Ottawa, Ont. [11]) and AISI S213 [1]. Information from the previous research was used to select the wall configuration, the test method and as comparison for the test results.

3.2. Test setup and loading protocols

Two full scale specimens were tested under design vertical and horizontal loads in order to characterize experimentally the lateral wall strength and stiffness. In particular, two identical 3.95 m high and 4.80 m long walls, able to reproduce in each detail a specific wall within the building (Fig. 2), were under investigation. The vertical and horizontal design actions for the seismic load conditions as well as the strength and stiffness calculated in the design phase are summarized in Tables 4 and 5, respectively.

The gravity and racking loads were distributed to the wall by two coupled steel beams (a RHS $300 \times 150 \times 6$ and a IPE 500 beams) that were set on the wall top (Fig. 3). To ensure the effective horizontal load transfer, the rectangular hollow beam was

Table 1
Wall components.

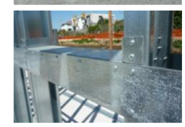
Cold-formed steel profiles			
Steel grade	S350GD $f_y = 350$ MPa; $f_t = 420$ MPa		
Steel profiles	Studs	150 × 50 × 30 × 1.50 mm (height × width × lip length × thickness) C section	
	Wall track	153 × 50 × 1.50 mm (height × width × thickness) U section	
	Wall blocking	150 × 50 × 30 × 1.50 mm C section	
	Flat strap	100 × 1.50 mm (width × thickness)	
Sheathing			
Wall sheathing	Internal and external	OSB/3 type KRONOSPAN 1200 × 4000 × 9.0 mm	
Steel-to-foundation connections			
Hold-down connector		Specific steel components Base material: S700; $f_y = 700$ MPa; $f_t = 750$ MPa	 
Tension anchors		HIT-RE 500 injection adhesive with 5.8 class M24 HAS rod	
Shear anchors		Mechanical anchors HILTI HST 8.8 class M8 @ 200 mm spacing	
Steel-to-steel connections			
Profile-to-profile		Self drilling, self tapping screws type TECFI CI 01 48 [7]	
Hold-down-to-studs		8.8 Class M18 Bolts	
Steel framing-to-sheathing connections			
		Self drilling, self tapping screws type TECFI CH 01 42 025 @ 100 mm spacing on the perimeter and @ 300 mm in the field, edge distance no less than 15 mm	

Table 2
Test matrix.

Number of tests	Test typology
2	Shear walls
40	Screws
30	OSB panels
44	Sheathing-to-frame connections
10	Hold-down connectors

Table 3
State of art of tests on sheathed braced walls.

Author	No and type of tests	Bracing system
McCreless and Tarpy^{NA} [12]	16^{M}	GWB
Tarpy and Hauenstein^{NA} [13]	18^{M}	GWB
Tarpy^{NA} [14]	4^{M}, 8^{C}	GWB, CP
Tarpy and Girard^{NA} [15]	14^{M}	GWB, PLY
Tissell^{NA} [16]	8^{M}	OSB, PLY
Serrette^{NA} [17]	12^{M}	GWB, GSB, OSB, PLY, X-B
Serrette and Ogunfunmi^{NA} [18]	13^{M}	GSB, GWB, X-B
Serrette et al.^{NA} [19]	24^{M}, 16^{C}	GWB, OSB, PLY
Serrette et al.^{NA} [20]	33^{M}	GWB, FBW, OSB, PLY
Serrette et al.^{NA} [21]	16^{M}, 28^{C}	OSB, PLY, SSS, X-B
NAHB Research Center^{NA} [22]	4^{M}	OSB, GWB
Gad et al.^{A} [23]	(1) 1^{C}	X-B
Selenikovich et al.^{NA} [24]	6^{M}, 10^{C}	OSB, GWB
Vagh et al. [25]	17^{C}	OSB, GWB
COLA-UCI^{NA} [26]	18^{C}	OSB, PLY
Dubina and Fulop^{E} [27]	7^{M}, 9^{C}	SCS, GWB, OSB, X-B
Filippson^{E} [28]	(2) 1^{C}	GWB
Serrette et al.^{NA} [29]	10^{M}, 10^{C}	OSB, PLY, SSS
Landolfo et al.^{NA} [30]	(1) 1^{M}, 1^{C}	OSB, GWB
Branston et al.^{NA} [31]	6^{M}, 6^{C}	OSB, PLY
Branston^{NA} [32]	21^{M}, 22^{C}	OSB, PLY
Cremer^{E} [33]	4^{M}	GWB
Serrette et al.^{NA}[34]	8^{C}	OSB, SSS
Tian et al.^{E} [35]	10^{M}	CP, OSB, X-B
Boudreault^{NA} [36]	10^{M}, 10^{C}	PLY
Chen^{NA} [37]	24^{M}, 22^{C}	OSB, PLY
Blais^{NA} [38]	9^{M}, 9^{C}	OSB
Rokas^{NA} [39]	15^{M}, 10^{C}	PLY
Kesti et al.^{E} [40]	19^{C}	X-B, GWB, PLY, SSS
Lange and Naujoks^{E} [41]	27^{M}	CHI, GWB, CP, SCS
Al-Kharat and Rogers^{NA} [42]	9^{M}, 7^{C}	X-B
Moghimi and Ronagh^{A} [43]	20^{C}	GWB, X-B
Serrette and Nolan^{NA} [44]	14^{C}	OSB, PLY
Velchev et al. [45]	44	X-B
Yu^{NA} [46]	30^{M}, 30^{C}	SSS
Pan and Shan^{C} [47]	1^{M}	GWB, OSB, CSB
Shamin et al. [48]	(1) 5^{ST}, (2) 5^{ST}	SSS

(1): one story 3D structure; (2) two stories wall.
^{NA}: North American tests; ^{A}: Australian tests; ^{E}: European tests; ^{C}: Chinese tests.
^{M}: monotonic tests; ^{C}: cyclic tests; ST: shaking table tests.
CP: cement plaster; CHI: Chipboard; FBW: FiberBond wallboard; GSB: gypsum sheathing board; GWB: gypsum wallboard; OSB: oriented strand board; PLY: plywood; SCS: steel corrugated sheet; SSS: steel sheet sheathing; X-B: steel flat strap X-bracing; CSB: calcium silicate board.

attached to the specimen with M12 bolts along 2 lines, spaced every 200 mm. Two different gravity loads were applied in the first and the second tests, respectively: a load of 5.93 kN/m, corresponding to the design load per unit wall length defined for the X51 wall (Table 4) and a second load equal to 10.20 kN/m reproducing the maximum vertical design load per unit wall length acting in the building. Horizontal loads were applied by means of a double effect jack (COD25N260 by Europress) with the range of displacement of 260 mm. The out of plane displacements of the wall were avoided by four bracing systems.

Eleven LVDTs were placed to measure the displacements, as shown in Fig. 4. In particular, at each bottom side of chord stud, vertically and horizontally oriented position transducers were placed (LVDTs 3,4,7,8) to capture the wall uplift and slip. Lateral wall displacements were recorded by transducers placed at the top and mid-height of each chord stud (LVDTs 1, 2, 5, 6). One transducer (LVDT 9) recorded the top lateral displacement of the reactive structure. Two transducers (LVDTs 10, 11) measured diagonal displacements. Two load cells were used to measure the horizontal loads.

The specimens were tested under two loading protocols, each of them consisting of two phases. In the first phase, in order to evaluate the residual displacement under design loads, the walls were subjected to loading and unloading cycles up to displacements equal to 9 and 13 mm, for the first (Fig. 5a) and the second (Fig. 5b) tests respectively. Then in both tests, the walls were unloaded and in the second phase, a monotonic load was applied up to the wall collapse.

3.3. *Test results*

The experimental response curves for both tests in terms of resistance (H) versus displacement (d) measured at wall top (LVDT1) are shown in Fig. 6. In both cases the collapse was governed by tilting and pull-through of sheathing-to-frame screws and the global deformation was coherent with this type of collapse mechanism: the frame deformed as a parallelogram and the panels were subjected to rigid rotation (Fig. 7). The maximum reached wall strengths (H_p) for the first and the second tests were equal to 147.5 kN and 127.6 kN, respectively, while the conventional elastic stiffnesses (k_e, secant stiffness evaluated at the 40% of the maximum strength) were equal to 8.24 kN/mm and 5.33 kN/mm for the first and second tests, respectively. The test results in terms of peak strength (H_p), elastic strength (H_e), elastic stiffness (k_e) and displacements at the top of the wall ($\Delta_{top,e}$) are summarized in Table 6.

In terms of strength a comparison between design calculation and experimental results can be developed taking into account the experimental peak strength per unit length ($H_p^* = H_p/L$). Therefore, assuming a theoretical length equal to $L = 4700$ mm (spacing between the centroids of chord studs), the experimental unit strengths for the first and the second tests are $H_{p,1}^* = 31.4$ kN/m and $H_{p,2}^* = 27.1$ kN/m, respectively, with an average strength of $H_{p,a}^* = 29.3$ kN/m. To obtain the design strength starting from experimental results, the following relationship can be considered:

$$H_{p,d}^* = H_{p,k}^*/\gamma_m$$

where $H_{p,k}^*$ is the experimental characteristic value of the peak strength that for the sheathed CFS walls can be defined as $0.9H_{p,a}^*$ according to Branston et al. [32] and γ_m can be assumed equal to 1.25 according to Veljkovic and Johansson [49]. Hence

$$H_{p,k}^* = (0.90 \times 29.3) = 26.4\ \text{kN}/m \text{ and}$$

$$H_{p,d}^* = 26.4/1.25 = 21.1\ \text{kN}/m$$

Since the design strength obtained from calculation (predicted) for the wall X51 is $H_d = 103.2$ kN and the design strength for unit length is equal to $H_d^* = 22.0$ kN/m, it can be noted that the predicted value is slightly higher than the experimental one ($22.0/21.1 = 1.04$).

In terms of stiffness, the tested walls showed an elastic stiffness of $k_{e1} = 8.25$ kN/mm and $k_{e2} = 5.33$ kN/mm, for the first and the second tests respectively (Table 6). Taking into account that in the

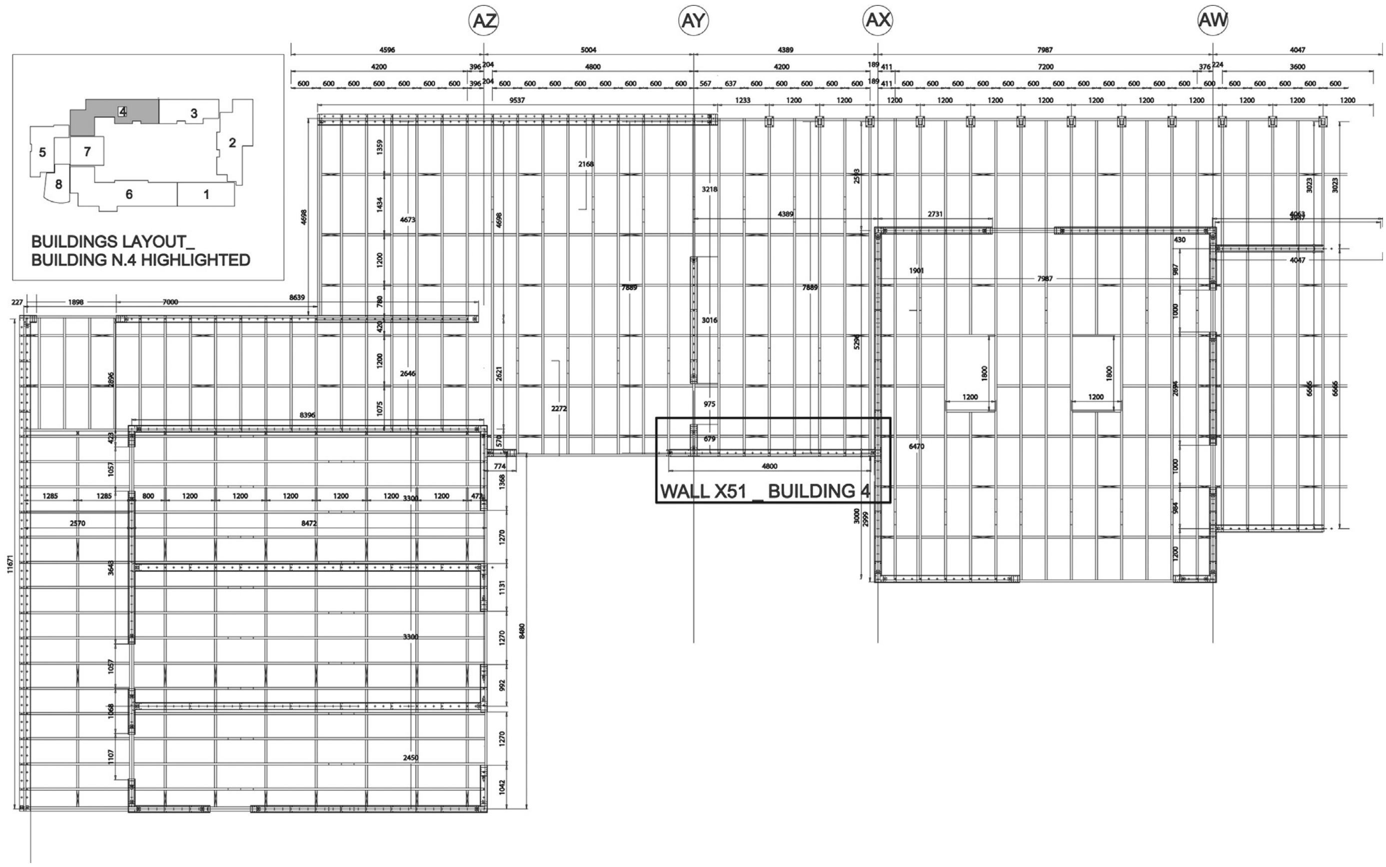


Fig. 2. Extract from building plan with indication of wall to be tested.

Table 4
Building 4_ Wall X51: Design Loads.

Vertical load per unit wall length	*Horizontal load*		
q_{sd} *[kN/m]*	$H_{sd,LS}$ *[kN]*	$H_{sd,IO}$ *[kN]*	$H_{sd,O}$ *[kN]*
5.93	*31*	*12*	*9.4*

LS: life safety.
IO: immediate occupancy.
O: operational.

Table 5
Building 4_ Wall X51: Strength and stiffness predicted in the design phase.

$H_{Rd,ULS}$ *[kN]*	k_e *[kN/m]*
103.2	7.0

$H_{d,ULS}$ design strength at the Ultimate Limit State.
k_e design stiffness.

design phase a stiffness equal to 7.0 kN/m was calculated, the average of the experimental results ((8.25 + 5.33)/2 = 6.79) and numerical provision are very close with a scatter of 3% (7.00/6.79 = 1.03).

Among all the literature tests summarized in Table 3, the closest to the presented wall specimens for sheathing panels (9.0 mm OSB), screws typology (diameter 4.2 mm) and screw spacing (100 mm along the panel edges and 305 mm in the field) were the ones carried out by Blais [38]. His work belonged to a wider research involving 43 light gauge steel frame/wood panel shear walls tested under lateral in-plane loading in the Jamieson Structures Laboratory of the Department of Civil Engineering and Applied Mechanics at McGill University, and was focused on 18 tests on OSB sheathed walls in three configurations. In particular, the tests labeled 43 A-B-C and 44 A-B-C were similar to the presented study with as main difference the fact that the OSB was present only on one side of the wall. The main results in terms of strength are summarized in Table 7.

Taking into account that these literature wall tests were carried out on walls sheathed on one side and designed in order to collapse for effect of sheathing-to-frame failure, a comparison in terms of strength between these results with those presented in this paper can be carried out by doubling the experimental strength values, so as defined by the AISI S213 in Chapter C2.1 [1]. Therefore, unit strengths equal to $H^*_{p,43} = 36.8$ kN/m and $H^*_{p,44} = 34.4$ kN/m can be considered for tests 43 and 44, respectively, with an average of $H^*_{p,B,a} = 35.6$ kN/m. These last strengths are comparable with those obtained in the experimental campaign presented in this paper ($H^*_{p,a} = 29.3$ kN/m) with a scatter of 22%.

4. Tests on connecting systems and materials

The shear response of sheathed CFS walls is influenced by the behavior of all the structural components: frame, sheathing-to-frame connections, sheathing panels and frame-to-foundation anchors. Therefore, in order to have a complete characterization of all the components, shear tests of screws, OSB panels and sheathing-to-frame connections and tension tests of hold-down

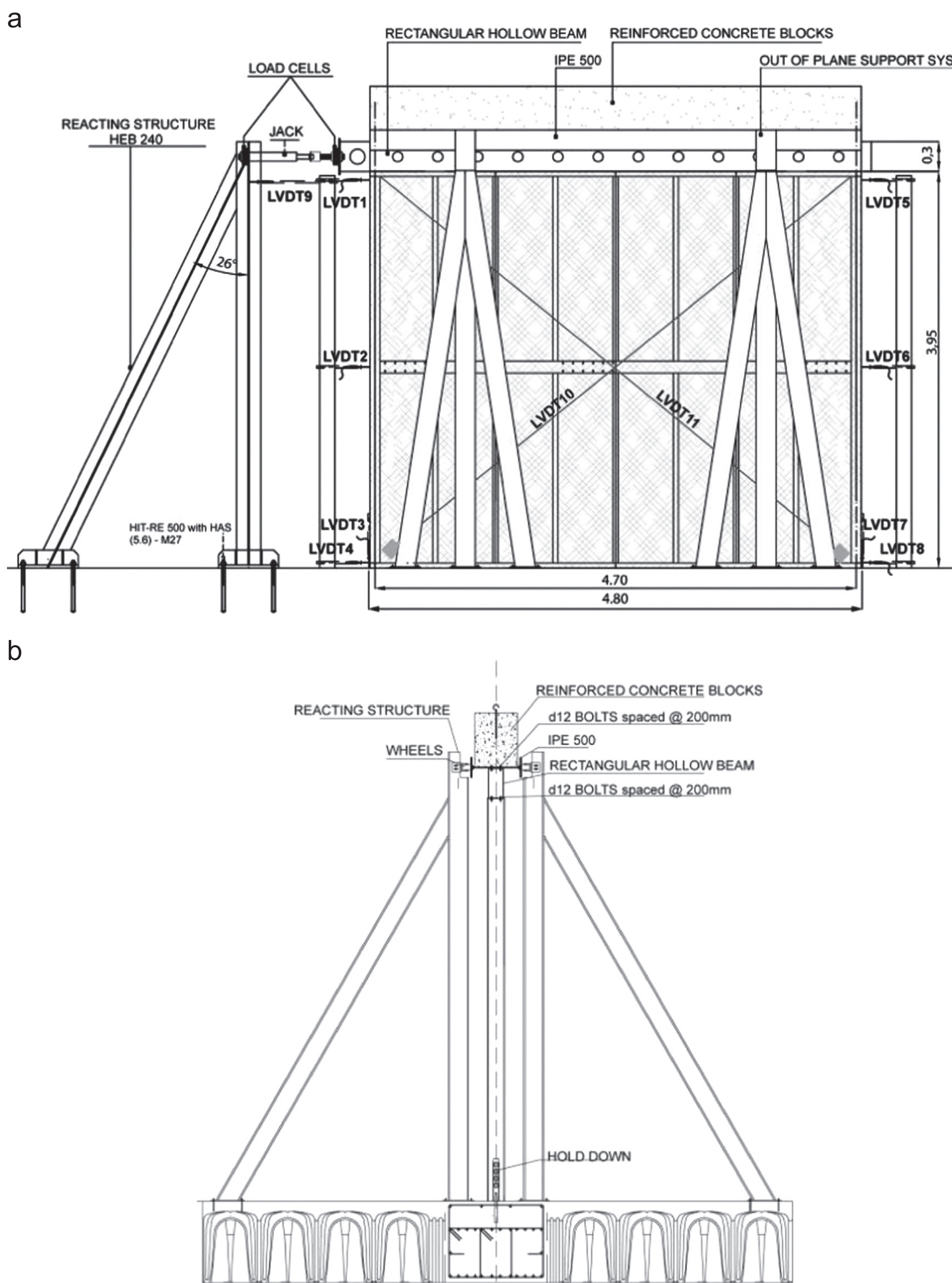


Fig. 3. Wall test setup: (a) front view and (b) section.

connectors were performed. In the following the tests on connecting systems and materials are analyzed in detail.

4.1. Screws

The self-drilling screws have been usually adopted as connections in non-load bearing elements and only in the last decades they have been used for floors and load bearing walls realized by CFS members. In one operation, these externally threaded fasteners can drill their own hole and form their own internal threads without deforming their own thread and without breaking during assembly. In order to satisfy a wide range of requirements, the screws are available in several typologies and dimensions, and they can be coated by zinc, cadmium or co-polymer coatings in order to be safely adopted in external environments. The design of fastener connections can be developed according to EN 1993-1-3

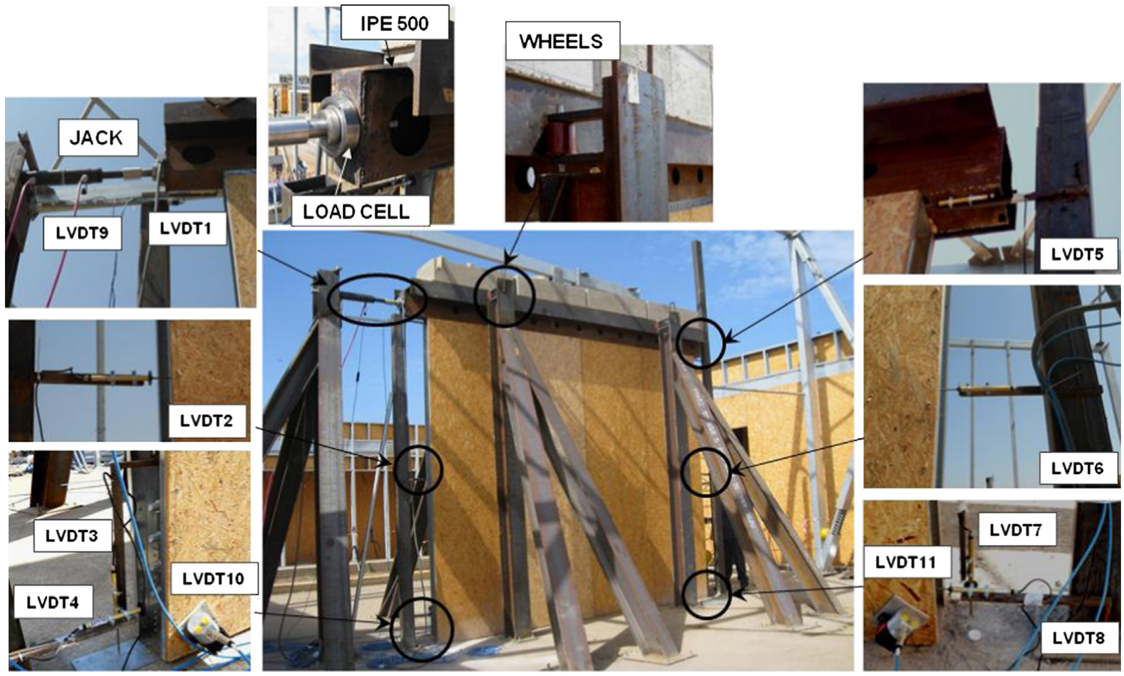


Fig. 4. Wall specimen.

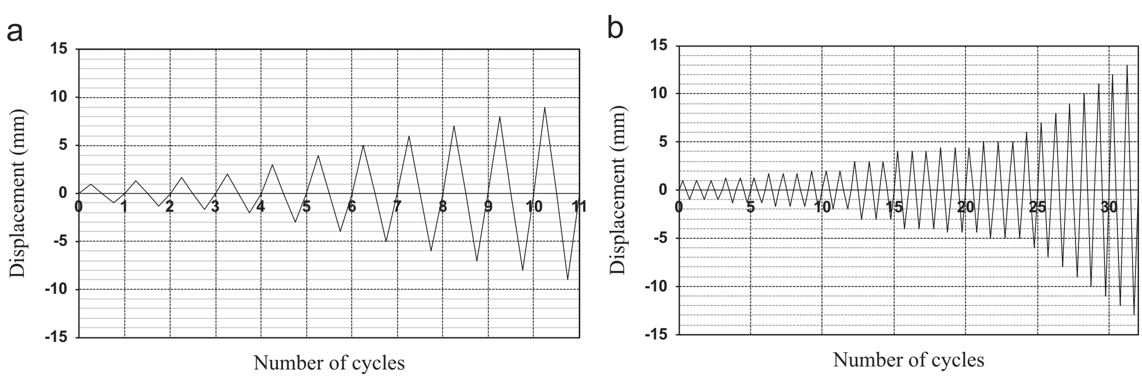


Fig. 5. Cyclic protocol. (a) First test and (b) second test.

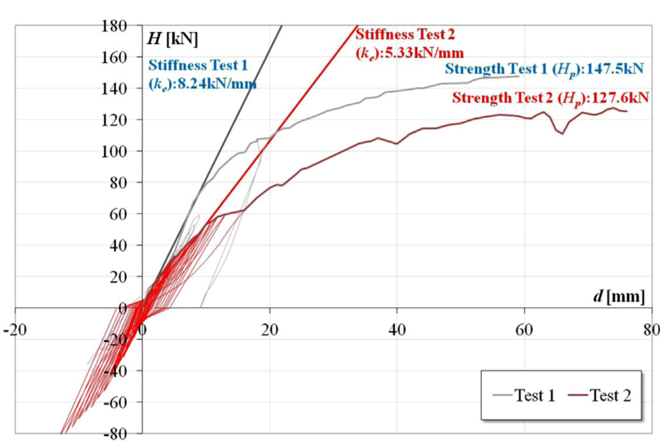


Fig. 6. Experimental response of the walls.

[50] or AISI S100 [51] and, in both cases, the connection strength is evaluated on the basis of the failure mechanisms. In particular, the EN 1993-1-3 allows the evaluation of shear strength on the basis of the bearing resistance ($F_{b,Rd}$), net-section resistance ($F_{n,Rd}$) and the screw shear resistance ($F_{v,Rd}$), where, this last depends on the characteristic shear screw resistance ($F_{v,Rk}$) evaluated by experimental tests. Therefore, $F_{v,Rk}$ has been investigated for each screw typology adopted in the BFS school (4.2 mm diameter flat-head, 4.8 mm diameter lath head, 5.5 mm diameter flat-head and 6.3 mm diameter hex washer head screws) by 10 shear tests were carried out, per a total of 40 tests. The lack of codified procedures for screw tests execution required the definition of a reliable setup [52]. Hence, the samples and test setup have been defined in order to reach the required failure mechanism of the fastener without the rising of other mechanisms that could infect the results. In particular, the screw shear response was evaluated by using a three fastener lap shear test (Fig. 8), where the screw passes through three drilled plates and, finally, it drills and taps an additional thin steel plate, in order to avoid the screw pull out during the test. For the setup, heat treated steel plates were adopted with hardness values ranging between 50 and 55 HRC, that assured high strength in order to avoid plate bearing. Moreover, the plate surfaces were adequately smoothed in order to reduce the friction and to avoid any loading dissipation. Plate dimensions and thickness are shown in Fig. 8a.

Fig. 7. Deformed wall.

Table 6
Test results.

	H_p [kN]	H_e [kN]	k_e [kN/mm]
First test	147.50	59.00	8.24
Second test	127.50	51.04	5.33

H_p peak strength.
H_e elastic strength.
k_e conventional elastic stiffness.

Table 7
Strength and stiffness values recorded by Blais [38].

Blais	H_p [kN/m]
43 A	17.7
43 B	18.0
43 C	19.6
Average	**18.4**
44 A	18.2
44 B	16.2
44 C	17.3
Average	**17.2**

The test results, as summarized in Fig. 9 and Table 8, show that the experimental data present very low scattering with a maximum C.O.V. of 6%. Moreover, the experimental characteristic strength values are very similar to those provided by the producer and always higher than those adopted in the design (1–4% higher).

4.2. Sheathing panels

Among the several panel typologies usually adopted for the CFS systems, 9.00 mm thick OSB type3 with vertical orientation were adopted as sheathing boards in the analyzed case study, because of high mechanical properties that make these panels particularly suitable for load-bearing applications. In order to evaluate the shear strength and shear modulus of the OSB panels 15 shear tests were carried out on OSB parallel (OSB$\parallel$) and perpendicular (OSB $\perp$), respectively, for a total of 30 shear tests.

The adopted test procedure is the edgewise shear test, described in ASTM D 1037 [53]. The specimen consists of a 254×90 mm^2 panel clamped between two pairs of steel loading rails, as shown in Fig. 10. The specimen was tightly bolted between the rails to prevent slipping and then loaded in compression using an universal testing machine. The relative displacements between the two couple of rails were recorded by an LVDT.

Table 9 summarizes test results in terms of shear strength and shear modulus and Fig. 11 show the recorded shear value for each test together with the average value and the dispersion.

It can be noted that the results are not affected significantly by the panel orientation. In fact, the average shear strength and modulus for parallel direction is slightly higher (8% and 21% for shear strength and modulus, respectively) than those obtained for the perpendicular one. In addition, higher dispersion obtained in the case of OSB$\parallel$ leads to similar value of characteristic strength for both directions. For both orientations the characteristic shear strength values and the average shear modulus are higher than those adopted in the design phase.

4.3. Sheathing-to-frame connections

The sheathing-to-frame connections adopted for both walls and roof floor were tested. Therefore, three different configurations were considered: 9 mm thick OSB panels connected to 1.5 mm thick CFS profiles with 4.2 mm diameter flat head screws; 18 mm thick OSB panels connected to 1.5 mm thick CFS profiles with 5.5 mm diameter flat head screws; 18 mm thick OSB panels connected to 3.0 mm thick CFS profiles with 5.5 mm diameter flat head screws. The test setup procedure defined by the authors in previous experimental campaign [54–57] was adopted. In particular, the generic sheathing-to-profile connection specimen consisted of two single 200×600 mm sheathings attached to the opposite flanges of CFS profiles. Steel profiles were made of $100 \times 50 \times 20 \times t$ (with $t = 15$ mm or $t = 3.0$ mm) lipped channel sections (Fig. 12). In particular, taking into account that the OSB

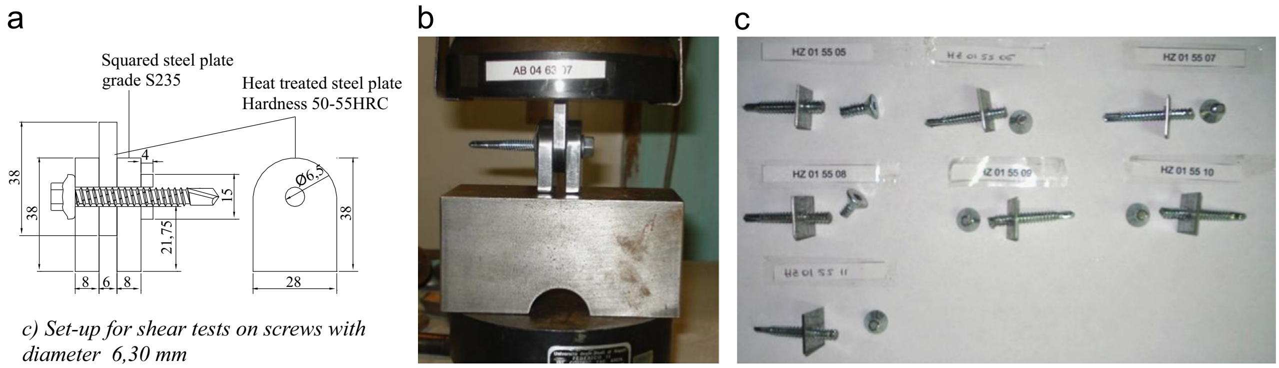


Fig. 8. Shear screw test. (a) Test set up, (b) specimen and (c) collapse.

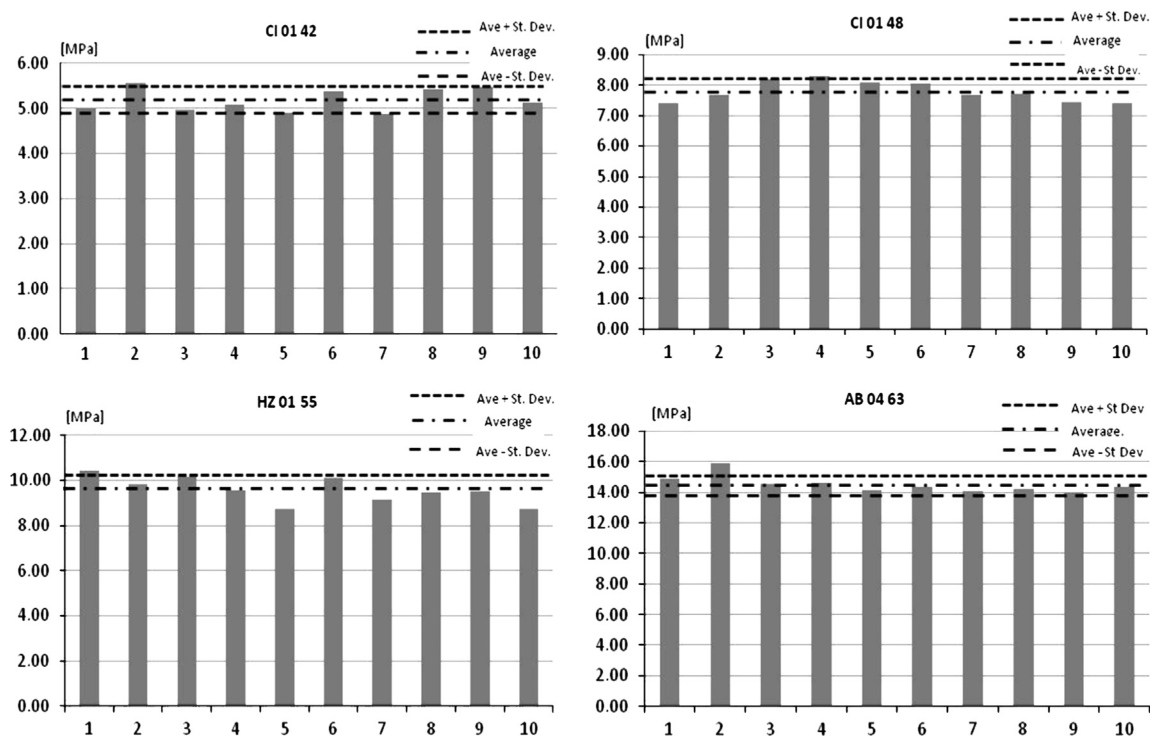


Fig. 9. Shear strength recorded in each test varying the screw typology.

Table 8
Screws tests results: shear strength.

Typology		Diameter [mm]	Number of tests	Average value [kN]	Standard deviation [kN]	C.O.V.	Characteristic value [kN]	Value provided by the producer [kN]	Characteristic value assumed in design [kN]
1	CI 01 42	4.2	10	5.18	0.26	0.05	4.68	4.55	4.50
2	CI 01 48	4.8	10	7.80	0.34	0.04	7.15	6.59	6.50
3	HZ 01 55	5.5	10	9.58	0.58	0.06	8.48	8.97	6.50[a]
4	AB 04 63	6.3	10	14.51	0.57	0.04	13.42	12.68	12.50

[a] During the construction, HZ 01 55 screws have been adopted to connect joists and sheathing, instead of the CI 01 48 assumed in the design process, for simplicity of assembling. Therefore, in the table the reference characteristic value assumed in design for CI 0148 and HZ 01 55 are the same.

panels are composed of wood strand oriented along a principal direction, as for the shear tests on panels presented in the previous section, two different configurations were investigated: panels with strands in direction parallel (OSB∥) and perpendicular (OSB⊥) to the applied loads. Sheathings were connected to the top steel member by three screws (tested connections) and to the bottom members by two rows of eight screws (oversized connections). The tested connections presented an edge distance equal to 20 mm. The

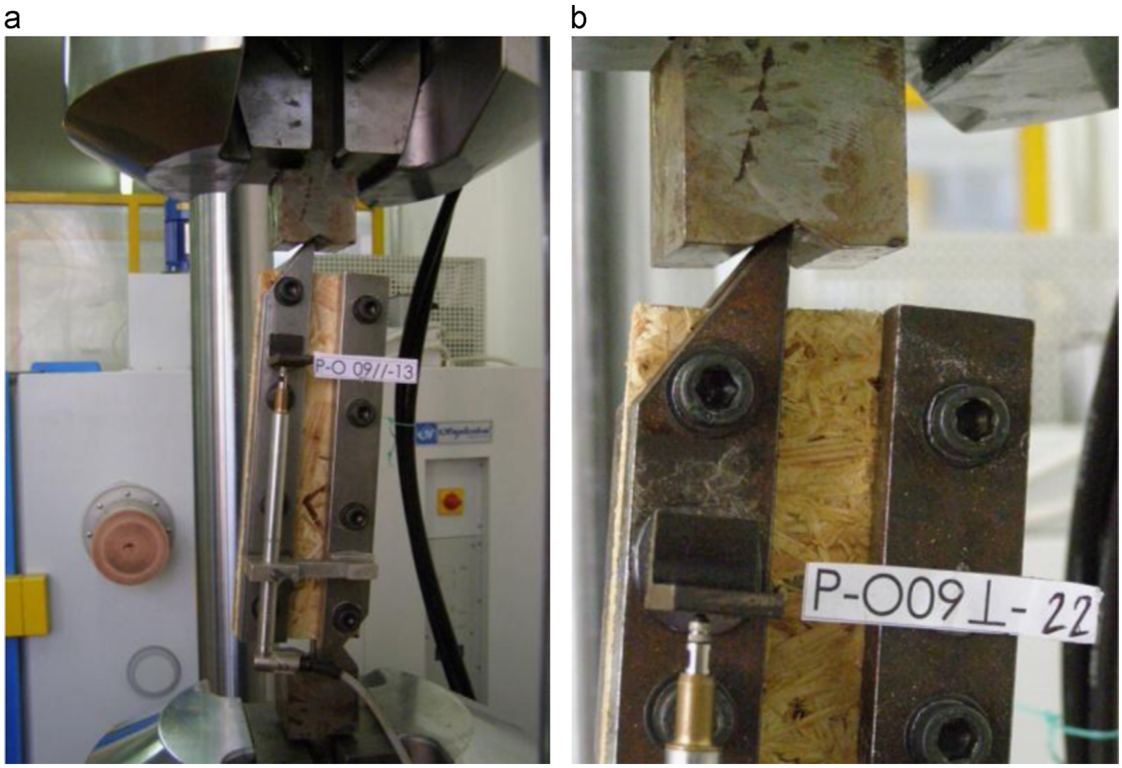


Fig. 10. OSB panel test specimen. (a) Set up and (b) failure.

Table 9
OSB tests results.

Typology	Number of tests	Shear strength					Shear modulus	
		Average value [MPa]	Standard deviation [MPa]	C.O.V.	Characteristic value [MPa]	Characteristic value assumed in design [MPa]	Average value [MPa]	Average value assumed in design [MPa]
P-O 09 ‖	15	7.62	0.87	0.11	6.17	3.00	1156	750
P-O 09 ⊥	15	7.06	0.56	0.07	6.19	3.00	953	750

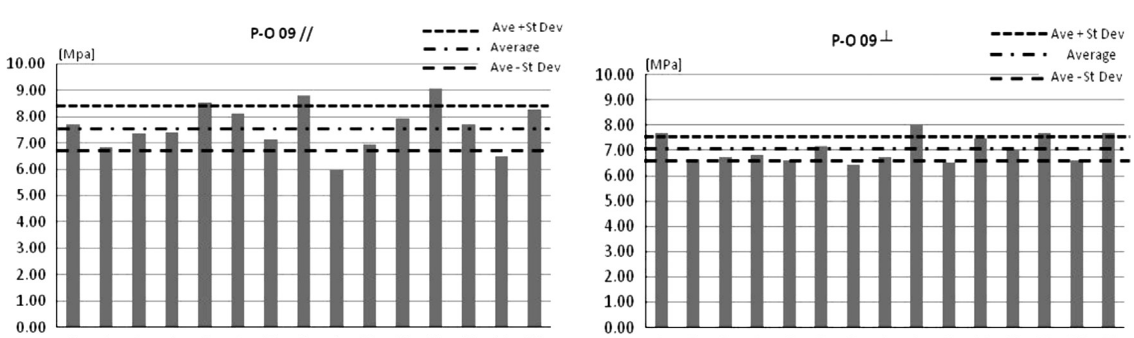


Fig. 11. Shear strength recorded in each test.

specimens were assembled by the men working on the construction site, in order to verify the ability of the workers and the susceptibility of the test results to the assembling. For all tests the failure mechanism was tilting and pull-through the sheathing. The results in terms of peak strength (F_u), elastic stiffness (k_e), ductility (μ) and dissipated energy (E) for single connections are presented in

Fig. 12. Sheathing-to-frame connections specimen.

Table 10
Sheathing-to-frame connections results in terms of strength.

Specimen	Screw diameter [mm]	OSB thickness [mm]	Number of tests	F_u				
				Average[kN]	Standard deviation [kN]	C.O.V.	Charact. Value [kN]	Charact. value assumed in design [kN]
09∥S15D42	4.2	9	12	1.50	0.10	0.08	1.36	1.35
09⊥S15D42	4.2	9	12	1.57	0.13	0.07	1.34	1.35
18∥S15D55	5.5	18	5	2.61	0.23	0.09	2.08	2.00
18⊥S15D55	5.5	18	5	2.62	0.16	0.06	2.25	2.00
18∥S30D55	5.5	18	5	2.98	0.34	0.12	2.18	2.00
18⊥S30D55	5.5	18	5	3.17	0.18	0.06	2.74	2.00

Table 11
Sheathing-to-frame connections results in terms of stiffness, ductility and dissipated energy.

Specimen	Number of tests	k_e			μ			E		
		Average [kN/mm]	Standard deviation [kN/mm]	C.O.V.	Average	Standard deviation	C.O.V.	Average [kN∗mm]	Standard Deviation [kN∗mm]	C.O.V.
09∥S15D42	12	0.81	0.29	0.36	13.84	3.63	0.26	12.43	2.00	0.16
09⊥S15D42	12	0.93	0.23	0.25	14.34	3.30	0.23	12.43	2.51	0.16
18∥S15D55	5	1.87	0.34	0.18	17.54	3.93	0.22	20.38	1.60	0.08
18⊥S15D55	5	1.34	0.24	0.18	13.13	1.85	0.14	21.72	4.01	0.18
18∥S30D55	5	2.03	0.34	0.17	11.59	3.03	0.26	15.23	3.94	0.26
18⊥S30D55	5	1.38	0.19	0.14	7.91	1.74	0.22	17.15	2.50	0.15

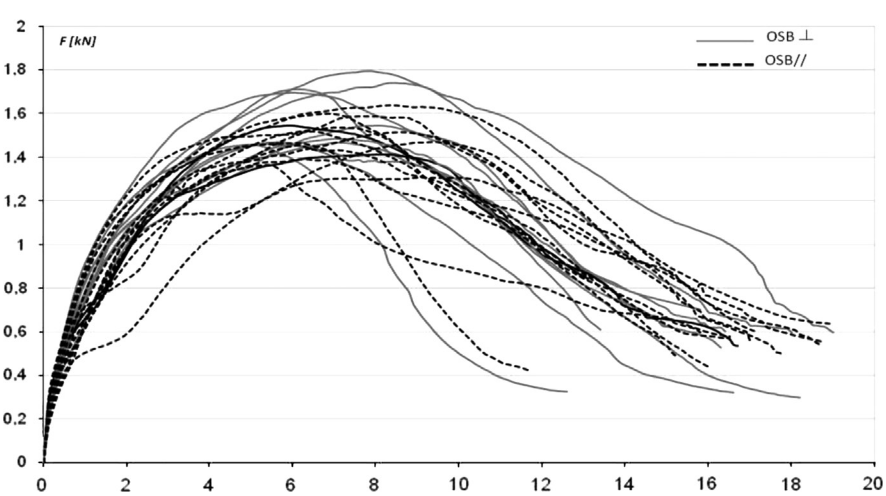


Fig. 13. Force vs. displacement response curves.

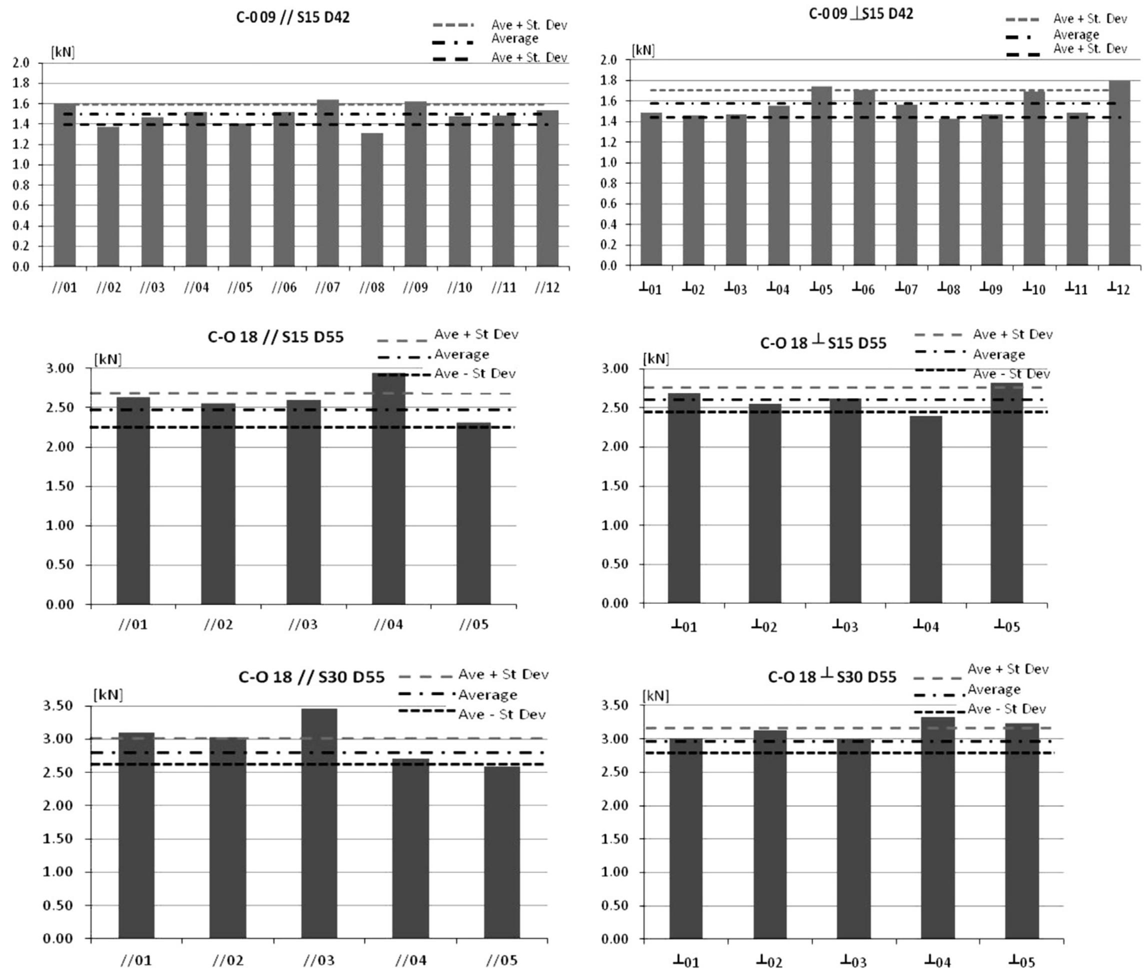


Fig. 14. Test results in terms of shear strength.

Tables 10 and 11. The response curve are reported in Fig. 13 and the ultimate strength (F_u) recorded in each test are shown in Fig. 14 together with the average and the dispersion in terms of standard deviation value. The results show that the sheathing orientation does not influence the response with a clear trend, instead the steel thickness significantly influences the results. In fact increasing the steel thickness produces an increment of strength and stiffness. The average of the characteristic values between OSB‖ and OSB⊥ is always equal or higher than those considered in the design phase.

4.4. Hold-down devices

For market and economic reasons, in the presented project, specific hold-down devices were designed by the authors. The design was developed in order to assure overstrength of the device with respect to the connection between hold-down and concrete foundation. The geometry of the hold-down device is presented in Fig. 15a.

Nine tests were carried out to verify the overstrength of the device to the steel rod. The test setup has been defined in order to reproduce the on-site loading conditions and it was tested with an universal testing machine. Therefore, as shown in Fig. 15b, a M24 steel threaded rod was connected through the hole of the hold down base to the upper clip of the universal testing machine. Three 20 mm thick steel plates were bolted to the device and connected to the machine bottom clip in order to center the axial tension loads. In all tests the collapse was due to the rod failure, without any appreciable device deformation (Fig. 15c). The results in terms of strength recorded in each test are shown in Fig. 16 together with the average value and the dispersion in terms of standard deviation value. Moreover, as shown in Table 12, the obtained characteristic value was about 27% higher than the minimum nominal collapse load given by the rod manufacturer [6].

5. Conclusive remarks

The lack in codification of specific rules for the design of CFS structures can hold back the implementation of these systems in seismic area. The presented case study is an exceptional example of application of the system in Italy. The adoption of the stick-built system for six of eight buildings was possible thanks to the knowledge of the client (British Command Force) about the potentiality of the system in terms of advanced quality, safety, durability and sustainability that makes the system largely used in North European countries for low-rise buildings, and because of the accredited background of the structural designers. On the other hand, the lack in the Italian code of regulation for the use of CFS constructions and prominently of a sheathed-braced solution required a long, complex and expensive verification process. The global behavior (lateral response of walls) was verified by two on site wall tests and the local behavior response (materials and components) was checked by 123 tests.

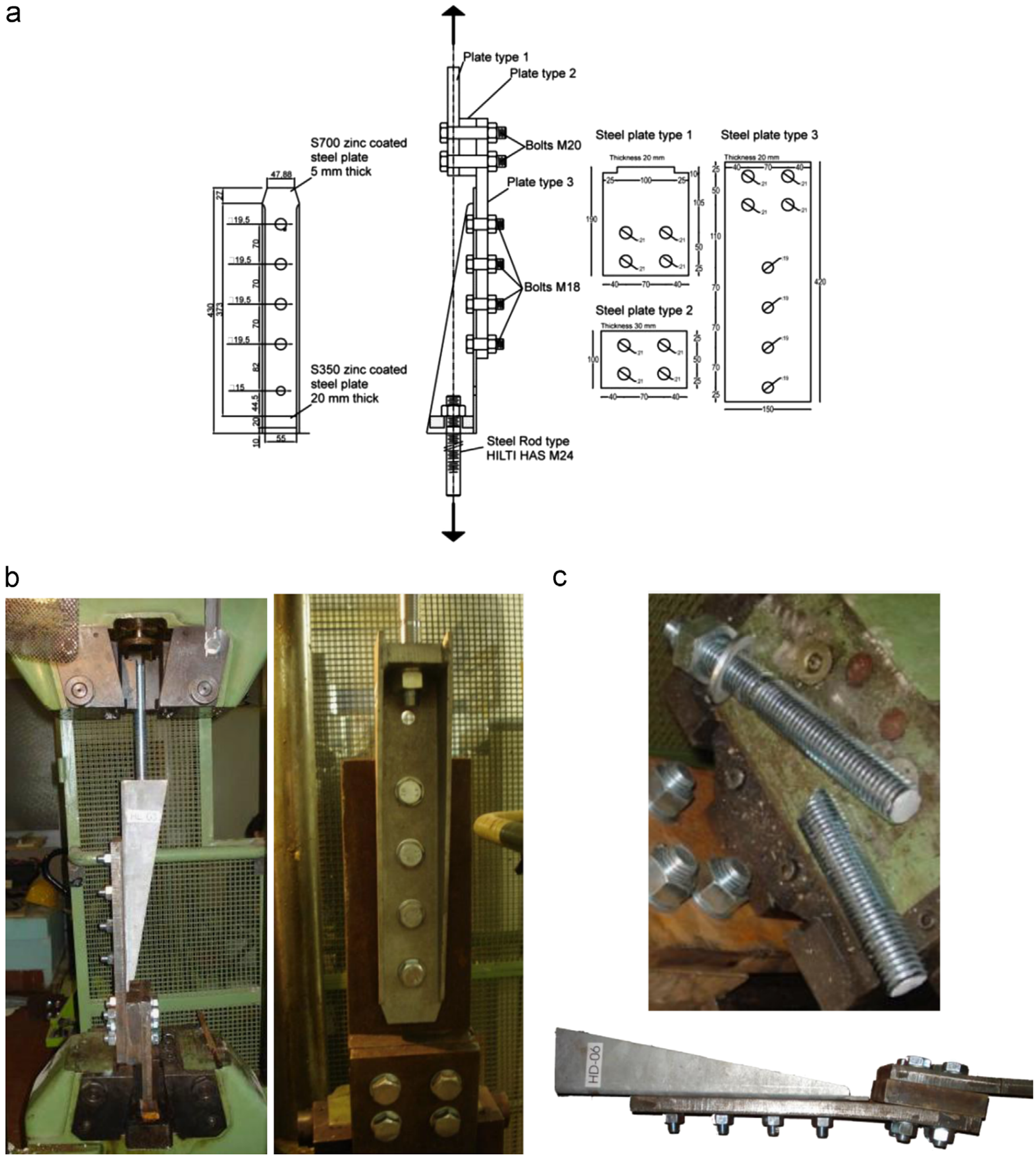


Fig. 15. Hold-down test. (a) Hold down device and set up, (b) hold down test and (c) specimen failure.

Table 12
Hold-down strength results.

Typology	Number of tests	Average value [kN]	Standard deviation [kN]	Coefficient of variation	Characteristic value [kN]
HD	10	241	22	0.09	203

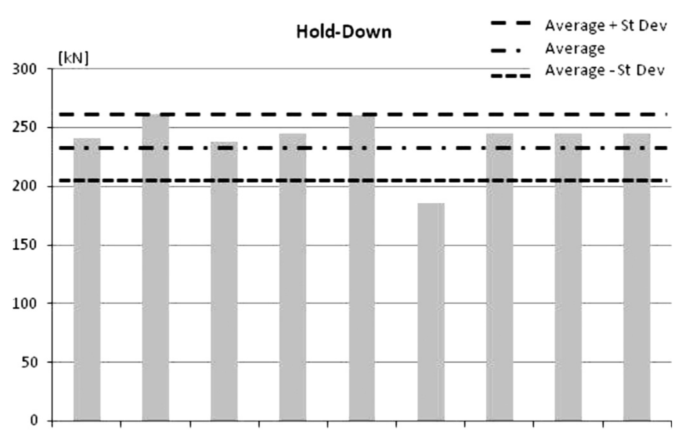


Fig. 16. Hold-down test results.

In particular, the two presented wall tests showed that the response, under vertical and horizontal loads, of shear walls with non-common geometry confirmed the design calculations with accurate prediction of the response in terms of strength and stiffness. At the local scale, the tests on materials and components provided a large experimental characterization of the main mechanical properties and demonstrated that the design assumption were reasonable.

The presented experience could be considered as a milestone for the Italian Code updating and represents a case study of great relevance for size and design solutions, with reference to Italian construction market.

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