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THEORETICAL AND EXPERIMENTAL EVALUATION OF SEISMIC BEHAVIOUR OF CFS WALLS WITH UHS STEEL BRACINGS

Alessia Campiche

Department of Structures for Engineering and Architecture
via Forno Vecchio 36, 80134, Naples
e-mail: alessia.campiche@unina.it

Abstract

During last decades the focus of civil engineering market has been moving towards safe, energy-efficient and low-impact constructions. In this perspective, many research activities focused on Ligthweight Steel (LWS) Structures made of Cold-Formed Steel (CFS) skeleton, developing many innovative solutions. To this aim, the Department of Structures for Engineering and Architecture of University of Naples "Federico II" in collaboration with the Italian company Lamieredil s.p.a. developed a load force resisting system (LFRS) made of CFS members and pretensioned Ultra-High Strength (UHS) steel diagonals in V configuration.
A wide experimental campaign, consisting of tests on materials, components and full-scale walls, was carried out to evaluate the seismic behaviour of this LFRS. The effectiveness of the LFRS was also proved on site, through the construction of a full scale building prototype on company ground. The paper describes in detail the innovative LFRS, starting from the concept and theoretical approach used to predict the structural response, summarizes the experimental campaign, presenting the main results and shows all the construction phases of the building prototype.

Keywords: Cold-formed steel, Full-scale prototype, Load force resisting system, Ligthweight steel, Seismic behaviour, Ultra-high strength steel

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1 INTRODUCTION

A joint synergy between different engineering fields can lead the path of innovation and hence making our constructions increasingly safe, sustainable and comfortable. Recent research activities have highlighted the ability of Steel Structures [1-18] and, in particular, Lightweight Steel (LWS) systems made of Cold Formed Steel (CFS) profiles [19-31], to guarantee high performances, both from structural and environmental point of view. In this perspective, a research project has been just concluded at University of Naples “Federico II” in cooperation with Lamieredil S.p.A. Company, which aims to develop innovative technological solutions with higher structural and environmental performances.

In this framework, a new system was developed, which mainly consists of CFS framing (tracks and studs) braced by pre-tensioned Ultra High Strength (UHS) steel bars in “V” configuration. The bracing acts as an anti-seismic device and it can limit global displacement of structure by working as an elastic spring and dissipating the seismic energy through the yielding. In order to investigate the seismic performances of the innovative system developed, an experimental campaign has been carried out at Laboratory of Department of Structures for Engineering and Architecture of University of Naples “Federico II”.

Moreover, as a conclusion of the research project, a full-scale prototype was designed and built in Calabria, South Italy, on the Company property, in order to validate all the solutions adopted. To investigate on structural behaviour and thermal performances of the prototype on site, structural and thermal monitoring has been installed.

The current paper describes in detail the concept of the innovative wall, the experimental tests carried out and discusses the results obtained. Moreover, a description of the prototype building is presented.

2 WALL CONCEPT

The innovative wall system developed (Figure 1) is mainly composed of: (1) UHS steel bracing; (2) pre-tensioning devices; (3) chord studs and intermediate studs; (4) tracks; (5) ad-hoc designed hold-downs; (6) blocking profiles and flat straps.

The UHS steel bracing consists of two diagonal braces, which are pre-tensioned dog bone shaped round bars having thread ends to allow their connection and pre-tensioning. For the chord studs and intermediate studs, a back-to-back C section and a simple lipped C section, respectively, were selected. For tracks a box section was chosen. The pre-tensioning device is a U shape profile connected to the hold-down through a cylindrical hinge, which allows the rotation in the plane of the wall. Blocking was made with box profiles.

A UHS steel (fy=1300 MPa, fu=1450 MPa) is adopted for the diagonal braces; studs, tracks, blocking and flat straps are made of S280 GD+ Z steel grade; pre-tensioning devices and hold-downs are made of S355 steel grade (fy=355 MPa, fu=470 MPa).

Since the system under investigation is not covered by European building code [32], following the prescriptions of Italian building code [33] for the seismic design of non-dissipative systems, which suggests a behaviour factor in the range 1.0 through 1.5, a behaviour factor equal to 1.0 was used in the design, in order to have an assumption on the safe side. On the other hand, since the plastic behaviour of the system is matter of investigation, the walls were designed following the Capacity design approach, in order to evaluate its dissipative capacity. Therefore, according to Capacity design approach, the diagonal braces are the dissipative elements and act as fuse of the system, in which a pretension f_{pt} is applied. Due to the geometry of the system, diagonal braces work only in tension field. Lateral capacity of the wall means the tension collapse of the diagonal brace, in correspondence to its middle zone, where bar diameter is reduced. In particular, the length of the bar with reduced diameter, l_b, is designed to allow a minimum lateral story drift ratio of the wall equal to about 2.5%. Under this assumption both the collapse in the bar threaded area and the failure of nuts, used for pre-tension the diagonals, should be avoided. According to capacity design approach, all non-dissipative elements of the wall systems, i.e. pre-tensioning devices, hold-downs, tracks and chord studs were designed for the expected resistance of the diagonal braces.

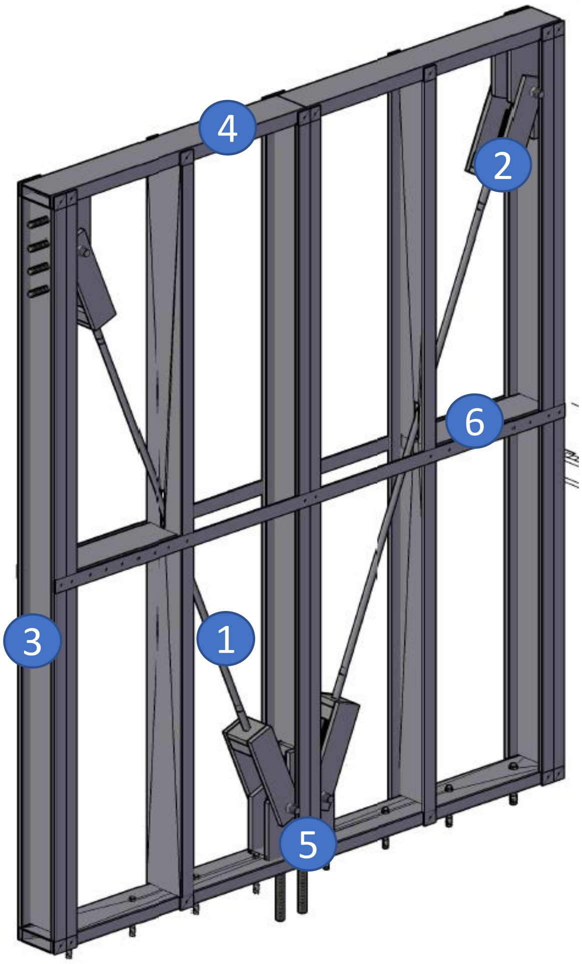


Figure 2: Innovative wall.

Under these hypothesis, the analytical lateral behaviour, when the wall is subjected to a horizontal in-plane increasing load, can be described by three main phases:
(1) in the initial phase, the system presents a linear elastic response in which both diagonal bars work in elastic field, i.e. a bar is subjected to a reduction of tensile stress, whereas in the other bar the tensile stress increases. In this phase the stress acting in the bar subjected to a reduction of loading is lower than f_{pt}.
(2) in the intermediate phase, the bar subjected to a reduction of tensile stress does not work, i.e. the bar is not able to react under compressive stress, whereas the bar subjected to an increasing tensile stress works in elastic field. This phase starts when the stress acting in the bar subjected to a reduction of loading becomes equal to zero.
(3) in the final phase, only the bar subjected to an increasing of tensile stress works in anelastic field, i.e. the bar is subjected to strain hardening. This phase starts when the stress acting in the bar subjected to an increasing loading becomes higher than nominal yield strength f_y and it ends with the collapse, due to tensile rupture of the bar, i.e. when the strain acting in the bar reaches a value equal to the ultimate strain of the steel ε_u. In order to guarantee the behaviour

described above, the brittle failure of nut should be avoided, choosing appropriately the number and property Class of nuts.
In order to guarantee different lateral resistance levels, three different configurations were designed: Light wall (L), Medium wall (M), Heavy wall (H), which the main properties are summarized in Table 1.

Configuration	$d_i^{(1)}$ [mm]	$d_t^{(2)}$ [mm]	$d_e^{(3)}$ [mm]	f_{pt} [MPa]	$H_{y,d}^{(1)}$ [kN]
Light (L)	16	20	21	300	83
Medium (M)	19	24	26	250	117
Heavy (H)	24	30	31	250	199

(1) d_i = minimum diameter of the bar; (2) d_t =diameter of threaded part, (3) d_e =maximum diameter of the bar

Table 1: Main capacity parameters of L, M and H walls.

3 EXPERIMENTAL TESTS

The general plan for the experimental assessment of the seismic response provided twenty-four tension tests for the mechanical characterization of materials used in the wall systems, two creep tests of UHS steel eight tension tests on bar-nut assemblies, three monotonic tests on full-scale walls (two for the light configuration and one for medium configuration) and two cyclic tests on full-scale walls (one for light configuration and one for medium configuration). All experimental activities have been carried out at the Lab of the Department of Structures for Engineering and Architecture, University of Naples "Federico II". Table 2 summarizes the experimental program.

Material tests: steel plate for frame members, hold-downs and pre-tensioning devices	**Label**	S3	S4	S20	S25	S30
	Thickness	3	4	20	25	30
	Steel grade	S280	S280	S355	S355	S355
	No. of tests	3	3	3	3	3
Material tests: steel cylinder for diagonal bars	**Label**	TL		TM	TH	
	Minimum diameter	12		14	18	
	Maximum diameter	21		26	31	
	No. of tests	3		3	3	
Material tests: creep tests on UHS steel	**Label**	CT				
	Diameter	26				
	Length	1200				
	No. of tests	2				
Component tests	**Label**	D1-10		D1-12	D2-10	
	Nut property Class	10		12	10	
	Nut number	1		1	2	
	No. of tests	2		3	3	
Full-scale wall tests	**Label**	WL			WM	
	No. of monotonic tests	2			1	
	No. of cyclic tests	1			1	

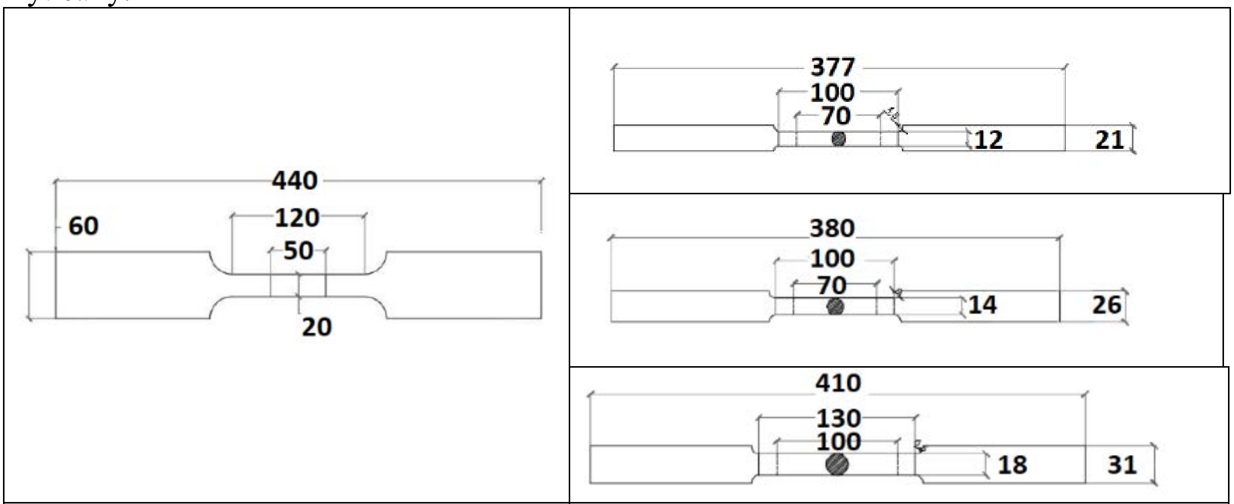
Table 2: Test matrix.

3.1 Material tests

Material coupons of frame members, diagonal bars and hold-down devices were subjected to conventional tension tests according to EN ISO 6892-1 [34]. In particular, tests were performed on nine specimen types: S3, S4 and S5 (Tab.2) for frame members; S20, S25 and S30

(Tab. 2) for hold-down devices and TL, TM and TH (Tab. 2) for diagonal bars. Coupons tested are shown in Figure 2 and tension test results are summarized in Table 3.

Since the UHS steel diagonals are pretensioned in the system and the UHS steel has not yet been investigated about its viscous properties, creep tests at ambient temperature were performed. For this purpose, in the research project two nominally identical set-ups were developed and installed in controlled temperature room to measure the creep behaviour of UHS steel bars. Set-ups consist of tubular section with holes to read the instruments and T-shaped welded profiles. Two 26 mm diameter and 1200 mm long UHS steel bar specimens were preloaded at 800 Nm, corresponding to a pretension of 175 kN, equal to about 50%f_{yd}. Specimens were equipped of centesimal dial gauges and invar wire to measure strains during the time. The measures continued for two years and the average result is shown in Figure 3 in term of ΔN in the time, where ΔN is the reduction of tension in the bar and was evaluated analytically.



a) Steel plate coupons for frame members, hold-downs and pre-tensioning devices

b) Steel cylinder coupons for diagonal bars

Figure 2: Tested coupons.

Nominal values					Experimental results					
Stell grade	$f_{y,n}{}^{a}$ [MPa]	$f_{u,n}{}^{b}$ [MPa]	$f_{u,n}/f_{y,n}{}^{a,b}$	Thickness/ diameter [mm]	Label	$f_{y,exp}{}^{c}$ [MPa]	$f_{u,exp}{}^{d}$ [MPa]	$f_{y,exp}/f_{y,n}{}^{a,c}$	$f_{u,exp}/f_{u,n}{}^{b,d}$	$f_{u,exp}/f_{y,exp}{}^{c,d}$
S280GD	280	360	1.3	3	S3.1	307.18	399.93	1.10	1.11	1.30
					S3.2	307.84	400.70	1.10	1.11	1.30
					S3.3	296.68	387.29	1.06	1.08	1.31
					AV[e]	303.90	395.97	1.09	1.10	1.30
				4	S4.1	330.54	414.75	1.18	1.15	1.25
					S4.2	327.81	414.98	1.17	1.15	1.27
					S4.3	326.18	4122	1.16	1.15	1.27
					AV[e]	328.18	414.22	1.17	1.15	1.26
S355JR	355	510	1.4	20	S20.1	356.98	561.91	1.58	1.10	1.57
					S20.2	336.43	533.31	1.50	1.05	1.59
					S20.3	339.85	533.83	1.50	1.05	1.57
					AV[e]	344.42	543.02	1.53	1.06	1.58
				25	S25.1	342.37	548.19	1.54	1.07	1.60
					S25.2	334.48	533.35	1.50	1.05	1.59
					S25.3	356.26	571.63	1.61	1.12	1.60

					AV[e]	344.37	551.06	1.55	1.08	1.60
				30	S30.1	374.02	560.79	1.58	1.10	1.50
					S30.2	414.10	620.95	1.75	1.22	1.50
					S30.3	249.19	354.50	1.00	0.70	1.42
					AV[e]	345.77	512.08	1.44	1.00	1.47
UHS	1300	1450	1.1	12	TL1	1236.95	1401.86	1.08	0.97	1.13
					TL2	1630.57	2089.71	1.61	1.44	1.28
					TL3	1424.64	1693.20	1.30	1.17	1.19
					AV[e]	1430.72	1728.26	1.33	1.19	1.20
				14	TM1	1494.47	1734.72	1.33	1.20	1.16
					TM2	1300.68	1548.66	1.19	1.07	1.19
					TM3	1301.40	1505.99	1.16	1.04	1.16
					AV[e]	1365.52	1596.46	1.23	1.10	1.17
				18	TH1	1346.45	1623.07	1.04	1.12	1.21
					TH2	1290.01	1487.99	0.99	1.03	1.15
					TH3	1300.00	1533.02	1.00	1.06	1.18
					AV[e]	1312.15	1548.03	1.01	1.07	1.18

[a] $f_{y,n}$: nominal yield stress;
[b] $f_{u,n}$: nominal ultimate stress;
[c] $f_{y,exp}$: experimental yield stress;
[d] $f_{u,exp}$: experimental ultimate stress;
[e] AV: average of the experimental values.

Table 3: Tension test results.

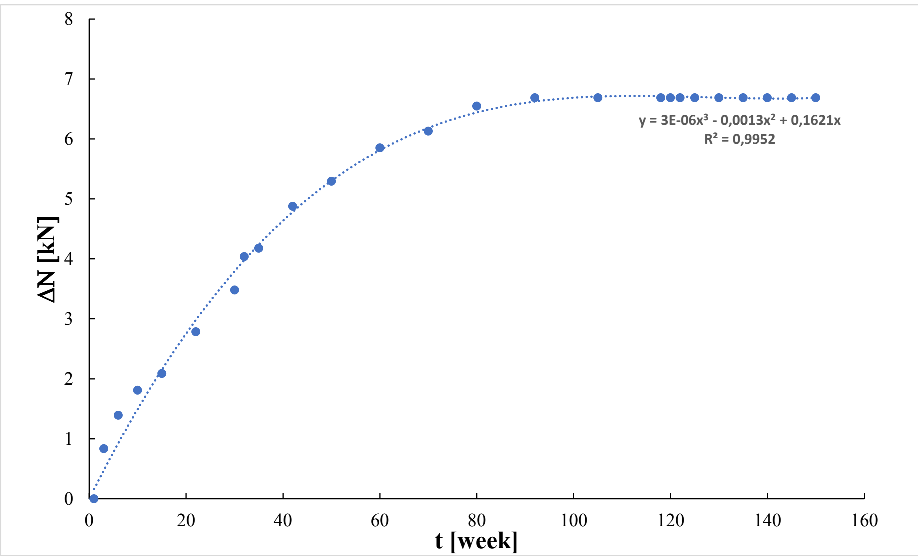


Figure 3: Creep test results.

3.2 Component tests

Since the assembly between nut and UHS steel bar is not codified and the behaviour of the innovative wall system can be affected by nut failure, a series of eight tests on different configurations was carried-out. In particular, the experimental tests aimed to select the nut property Class (10 or 12) and the number of nuts (1 or 2) to use, in order that the collapse mechanism of the wall consisted in the tension failure of diagonal bar in reduced section. Specimens are named as D number of nuts (D1 or D2), nut property Class (10 or 12),_ test number (1, 2 or 3), e.g. D1_10_1 means the test n°. 1 carried out on the specimen with one nut having nut property Class equal to 10. The tests were performed by using a universal test machine. In particular, specimens were subjected to imposed displacements at a rate of 0.05

mm/s without any preloading due to the tightening of nuts. The data were recorded with a sampling frequency equal to 5 Hz. Ad hoc set-up was designed for the goal. Results for all the specimens in term of load vs. displacement curve are presented in Figure 4, where F represents the load measured, whereas δ represents the displacement imposed. According to experimental results, all assemblies with one nut having property Class equal to 10 (D1_10_1 and D1_10_2) had a brittle failure of nut and the ultimate tension resistance of bar was not achieved. Instead, for all the specimens with two nuts having property Class equal to 10 (D2_10_1, D2_10_2 and D2_10_3) and one nut having property Class equal to 12 (D1_12_1, D_1_12_2 and D1_12_3) the tension failure of bar happened.
Starting from results of bar-nut assembly tension tests, the use of one nut with property Class equal to 12 was selected for the walls in a first stage. However, this preliminary choice was not successful and in the first wall monotonic test (M_L1) a brittle failure of nut happened. For this reason, in all further wall tests two nuts with property Class equal to 12 were adopted.

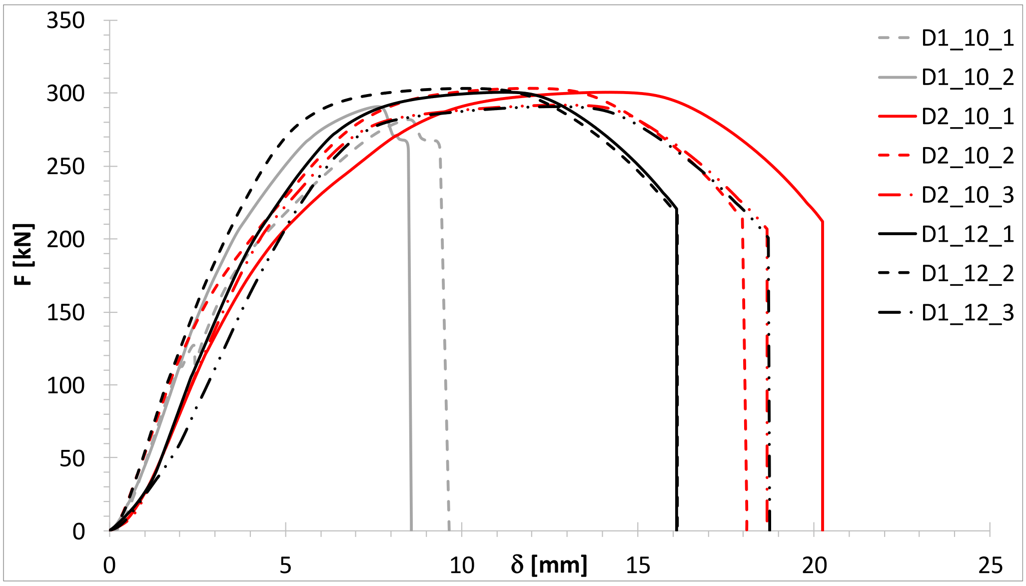


Figure 4: Component test results.

3.3 Full-scale wall tests

The lateral in-plane behaviour of the selected wall configurations has been investigated by means of five physical tests, including three monotonic tests and two cyclic tests on full-scale 2400 mm long and 2800 mm high wall specimens. WL and WM specimens were representative respectively of Light (L) and Medium (M) configurations developed. An available steel frame set-up for in-plane wall tests was modified and used for the experimental activity, more details are given by the Author in [35]. Six LVDTs and two potentiometers were used to measure the specimen displacements. In particular, three LVDTs were installed to record hold-down horizontal and vertical displacements, two LVDTs for the upper beam vertical displacements, one LVDT for wall vertical displacements and two potentiometers for wall horizontal displacements. Monotonic tests were carried out with displacements imposed at a rate of 0.10 mm/s until the collapse of specimens occurred, whereas 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.[36], modified according to the prescription given in Velchev et al. [37].

Results of monotonic and cyclic tests are provided in Table 4, considering the following parameters:

- H_0: wall strength corresponding to the end of the Phase (1);
- H_e: wall conventional yield strength evaluated according to ECCS procedure [38];
- H_y: wall strength corresponding to the end of the Phase (2);
- H_p: maximum recorded load corresponding to the end of the Phase (3);
- d_e: yield displacement evaluated according to ECCS procedure [38];
- d_{max}: maximum displacement, evaluated in correspondence of H_p
- $d_{r,e}$: yield inter-storey drift ratio, equal to d_e/h;
- $d_{r,max}$: maximum inter-storeydrift ratio, equal to d_{max}/h;
- k_e: initial elastic stiffness corresponding to the tangent to initial part of the response curve, equal to H_e/d_e;
- μ: ductility, equal to the ratio between the conventional ultimate displacement and displacement at conventional elastic limit load $d_{max}/\ d_e$ or equally $d_{r,max}/\ d_{r,e}$.

Globally, for all the specimens the monotonic response confirmed the three-phase lateral behaviour described. In particular, the curves are characterized by three different branches: (1) the first linear branch with a stiffness k_e; (2) once achieved H_0, the second branch characterised by a linear response having a stiffness smaller than k_e; (3) once achieved H_y, the third branch characterised by a nonlinear response. M_L2 and M_M1 specimens showed the same collapse due to the tension failure of the bar, whereas in M_L1 specimen, during the phase (3), nut failure happened, since only one 12 property Class nut was employed. The cyclic responses obtained showed a behaviour rather symmetrical. The three-phase behaviour also characterized the cyclic response, but only in pushing. In fact, for both specimens collapse happened immediately reached the Phase (3) in pushing (positive range), whereas in pulling (negative range) the Phase (3) was not reached.

Test	M_L1	M_L2	M_M1	C_L1	C_M1
H_0 [kN]	37.99	47.73	71.01	22.00	21.00
H_e [kN]	90.00	99.00	124.80	89.00	128.00
H_y [kN]	103.99	106.90	144.32	86.75	146.22
H_p [kN]	120.54	121.89	162.18	116.24	151.40
d_e [mm]	26.80	33.04	30.80	32.24	32.82
d_{max} [mm]	87.21	179.01	118.86	70.60	71.84
$d_{r,e}$ [%]	0.95	1.18	0.85	0.92	0.87
$d_{r,max}$ [%]	3.11	6.39	4.25	3.32	2.57
k_e [kN/mm]	3.36	3.00	4.02	2.76	3.90
μ [-]	3.25	5.42	3.86	2.19	2.19

Table 4: Full scale wall test results.

4 PROTOTYPE BUILDING DESIGN AND EXECUTION

The prototype is a two-storey building located in Sellia Marina, Calabria region (Italy), it covers an area of about 42 m^2 with a maximum height of about 8 m. The entrance to the building is located on the south-east side at road level. The wide openings on this front are protected by movable shielding elements in wooden slats, which ensure the optimization of natural inputs and the control of solar radiation during the summer months. The north-west front has small openings, useful for the exploitation of natural ventilation (cross-ventilation). On the inclined roof, 11 m^2 of photovoltaic panels and a mini-wind turbine will be installed. A solar chimney on the roof will serve for heating purposes. Natural ventilation mechanisms will allow better indoor air circulation. A heat recovery ventilation system (MVHR) can additionally contribute to the control of the internal microclimate and energy consumption. It will allow to use outdoor fresh air, filter and preheat it in winter, before feeding into indoor environments. The building can also be equipped with a system for recovery and reuse of rainwater and grey water, as well as an "intelligent" system able to manage the microclimatic and lighting conditions. The vertical opaque and transparent closures shall ensure high performance not only in relation to thermal transmittance, but also in relation to its behaviour in terms of thermal inertia, phase shift and attenuation of the thermal wave. It is also important to underline the possibility to build the enclosure using dry construction processes (structure / covering), that allows to save money and time, with high technological and environmental performances.

The building is structurally designed according to Italian building code [33] and EN1993-1-3 [32]. The structure will be realized as a CFS solution by using stick-built system. All CFS elements will be made of S280GD+Z steel grade (with yield, f_y, and ultimate, f_u, nominal strength equal to 280 MPa and 360 MPa, respectively).

The design under vertical and horizontal loads is conducted following the “all steel” approach, neglecting the contribution of panels to the resistance. Floors consist of 270×60×30×3.0 mm (out-side-to-outside web depth × outside-to-outside flange size × lip size x thickness) lipped C section joists spaced at about 600 mm on the centre. The joists are connected at the ends to unlipped U section floor tracks. The roof is made of CFS profiled trusses with maximum span equal to 6 m, spaced at centre-to-centre distance of 600 mm. The top and bottom chords are made up of 70 × 40 x 15 × 1.0 mm Ω-section, while studs and diagonals have a C-section of dimensions equal to 40 × 30 ×10 × 1.0 mm. Above the trusses, 60 × 40 × 15 × 1.0 mm Ω-section joists are placed, with centre-to-centre distance of 1200 mm. The walls consist of 150 × 60 × 20 × 1.5 mm lipped C section studs, connected at the ends to unlipped U section wall tracks.

As far as horizontal loads, mainly wind and seismic loads, are concerned, they are resisted by the new innovative wall system, proposed here. In particular, two wall systems will be placed for each direction at both floors. The prototype building is shown in Figure 5.

Figure 5: The prototype building.

5 ACNOWLEDGEMENTS

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6 CONCLUSIONS

In conclusion, an innovative wall system with better structural and thermal performances was developed. A wide experimental program was carried out to evaluate the seismic behaviour of the innovative system. It consisted of small-scale experimental tests on materials (tensile tests on structural materials and creep tests on UHS), tensile tests on bar-nut assemblies, full-scale tests on walls in light and medium configuration. The experimental results showed satisfactory seismic responses, in line with the theoretical previsions, if all the prescriptions are followed. In particular, if not well designed, the nut used for the bar pre-tensioning can cause a premature failure of the system and the displacement capacity is strongly reduced. The walls, in which all the design prescriptions were respected, exhibited satisfactory force and displacement capacity. In the end, the effectiveness of the innovations developed, and the optimization of production processes and execution phase were proved by means of the design and erection of a prototype building on the company ground.

For the future, it could be interesting testing the prototype building to evaluate the structural and thermal performances on site.

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