EXPERIMENTAL SEISMIC PERFORMANCE EVALUATION OF MODULAR LIGHTWEIGHT STEEL BUILDINGS WITHIN THE ELISSA PROJECT

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Abstract
Lightweight steel buildings made up by cold formed steel (CFS) members as main structural components are growing in popularity in the most industrialized countries. CFS buildings start to be adopted also in seismic regions thanks to their efficient fabrication, reduced site work, short time of construction and good structural performance. However, the dynamic properties and full seismic performance of CFS buildings completed with finishing is an open question. This paper attempts to provide a contribution to this research question, by a full experimental campaign aiming at investigating the dynamic properties of a modular building developed within the "Energy efficient LIghtweight Sustainable SAfe steel construction" (ELISSA) research project. The work shows the results of an international collaboration between universities and industrial partners aimed at developing a CFS prefabricated dry construction system with improved anti-seismic properties and energy performance. This work will discuss the design of the modular building named ELISSA house, the experimental investigation going from small scale tests of components, to static tests of shear walls, up to shake table tests of a two storey Mock-up building. It will analyse the dynamic properties of the structural system compared to the building completed with all the finishing, focusing on fundamental period of vibration, damping ratio, building drift and observed damage.

Keywords:
Cold-formed steel, Gypsum-based panels, Lightweight structures, Seismic design, Shake table tests, Shear walls

INTRODUCTION

Lightweight steel buildings that use cold formed steel (CFS) members as main load bearing elements have been developed since the Second World War, taking advantage of the economy and the efficiency of the system (Schafer et al. [1]). But, until recently, CFS systems have been mostly used for secondary structure, having full application as structural system only in the last 20 years. They are mostly used for low and medium rise housing buildings, although they have demonstrated the capability to be used in a variety of market applications. While CFS systems provide the potential to support the need for resilient and sustainable housing, the state of understanding their structural behaviour in response to extreme events remains relatively limited. In the attempt to improve the knowledge about the behaviour of CFS systems, with the final aim to exploit their adoption in seismic areas, many research teams around the world have worked in this field in the last years. In Europe, Dubina and colleagues performed tests on wood and plaster sheathed shear walls and developed numerical models (Fülöp et al. [2], [3]), up to seismic design procedures (Dubina [4]). This research group together with the Authors of this paper are now proposing the update of Eurocodes, in order to introduce this structural typology within the Eurocode 8.

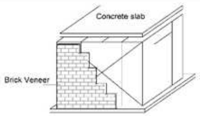
In Australia, Gad et al. [5] investigated the real performance of CFS buildings with shake table tests on single storey building, in which the lateral resisting systems was mainly composed on walls braced with diagonal straps in X-bracing configuration. The building was finished with bricks on the exterior and gypsum boards in the interior. The house was tested with racking cycles, swept sine wave and horizontal earthquakes in both direction and at different stages of construction, to evaluate the influence of the different structural and non-structural components on the seismic behaviour. Gad found that although X bracing governs the behaviour when the only bare structure exists, if non-structural components are added, then the stiffness strongly increases as well as loading bearing capacity and damping.

In North America, Rogers and colleagues investigated the behaviour of wood sheathed and steel sheathed shear walls panels (Branston et al.[6]) and X-braced shear walls (Al-Kharat et al.[7]) up to multi-storey walls by shake table tests (Shamim et al. [8,9]). Shamim and Rogers in particular carried out shake table tests on seven wood based and ten steel sheathed CFS shear walls, with the aim of investigating if the shear wall behaviour was consistent with the previous wall static tests results. Each specimen was subjected to free vibration tests and earthquake tests. They found that the response in terms of strength- versus-drift and the failure mode was consistent with previous static tests results.

Kim and colleagues [10] investigated the seismic properties of CFS walls braced with flat straps in X-bracing configuration with shake table tests of a two-storey building, in which the floors were made of reinforced concrete slab and the walls were 2.8 x3.0 m X-bracing CFS structures. The building was tested with random vibration tests and uniaxial natural earthquake applied with different scaling factors. They observed that X-bracing straps showed a very strong ductile behaviour for earthquake with high scaling factors, and that the columns contributed to the shear strength mostly when the bracing was no longer effective.

Schafer and colleagues [1,11] have just completed the CFS –NEES research project to advance knowledge in the field of the dynamic behaviour of CFS buildings, and provided all the necessary info to advance simulation tools for the seismic performance design evaluation of CFS buildings. Their work included experimental tests on connections, shear walls, and a shake table test of a two-storey building, tested under a series of dynamic excitations during different phases of construction. The building in particular, was designed considering only the CFS wall segments sheathed with OSB panels as main seismic resisting systems, while the remaining walls were considered to carry only vertical loads. Floors were considered as flexible diaphragms. The full building was tested in two phases, in a first phase only the structure was tested, while in a second phase the building completed with finishing and partition walls was tested. Key findings were that the non-structural elements strongly modify the behaviour. The building experience a very modest damage, with small inter-storey drift, and no residual drift. These tests opened up new questions in the field of the design of CFS buildings that this paper aims to address.
Just recently, a study of earthquake and post-earthquake fire behaviour of a six-storeys CFS-framed building was developed by a group of researchers and industries (Wang et al. [12]; Hoehler et al. [13]). The centrepiece of the project involved full-scale earthquake and fire testing of a full-scale six-story CFS building, with uniform plan dimension of 10.4 m × 7.3 m and total height of 19.2 m. The tests were carried out after the shake table tests presented in this paper. The main seismic resistant system was composed of steel sheathing stud walls. The building was subjected to low-amplitude white noise and ambient vibration during construction and seismic testing phases, then it was subjected to seven earthquake tests of increasing motion intensity. Earthquake motions were scaled to impose service, design, and maximum credible earthquake (MCE) demands onto the test building. Subsequently, live fire tests were conducted on the earthquake-damaged building at two select floors. Finally, for the first time, the test building was subjected to two post-fire earthquake tests. The building showed minimal damage during the service level earthquake tests and remained largely in the quasi-linear range, with very low drift. During the design level earthquake test, some damage started to occur with some damage in the form of gypsum panel crushing and fastener withdrawal when the inter-storey drifts at these two levels reached about 1.0%. From this point the fundamental period increased by more than 50% and the damage continued to progress. Globally, the building structural components performed satisfactorily throughout the pre-fire earthquake test sequence. The most significant damage to the structural system occurred in the form of buckled sheet steel.
The main findings of the cited previous research campaign which investigated the seismic behaviour of CFS buildings by shake table tests are summarized in Table 1, in terms of recorded natural period of vibration (T), recorded damping ratio (ξ), maximum inter-storey drift ratio (d/h) and observed damages.
In the last years the Authors, after analysing the seismic behaviour of CFS structures [14–19], through experimental analysis on connection systems [20–23], shear walls [24], strap-braced stud walls [25–27], up to propose a design methodology [28,29] and design a building for the British defence estate [22], they recently have led a new European effort to advance CFS constructions, improve the knowledge of the full system seismic behaviour and to develop modular construction packages that could be readily adopted in seismic areas having at the same time improved thermal and fire safety performance. This research is part of a FP7 European project Energy Efficient LIghtweight-Sustainable SAfe Steel construction (ELISSA), under the umbrella of "Energy-efficient Buildings (EeB)", Nanosciences, Nanotechnologies, Materials and new Production Technologies – NMP. The project is a collaborative work of three universities (National Technical University of Athens, University of Federico II in Naples, University of ULSTER in United Kingdom), one research centre (STRESS SCARL from Italy), and seven industrial partners (Farbe SPA (Italy), Woelfel Beratende Ingenieure GmbH & Co KG (Germany), Ayerisches Zentrum fur Angewandteenergieforschung ZAE EV (Germany), Knauf Gips GK (Germany), Haring Nepple AG (Switzerland), Knauf of Lothar Knauf SAS (Italy), VA-Q-TEC AG (Germany)). The Federico II research team primary aim was to develop understanding of seismic behaviour of CFS systems braced with high-impact gypsum based boards, having as central focus the full-scale shake table testing of a two storey house complete of all finishing.

Table 1. State of art on shake table tests on CFS walls and buildings

Image	Authors	Bracing system	Dynamic excitation	Recorded natural period of vibration (T)	Recorded damping (ξ)	Maximum inter-storey drift ratio (d/h)	Observed damage
	Gad et al. 1999	X-bracing	El-Centro Earthquake SF from 100% to 300%	From 0.22 s to 0.26 s	From 4.2% to 10%	0.21% (for 100% El-Centro Earthquake record) 0.46% (for 200% El-Centro Earthquake record)	No damages for SF of 100% Substantial yielding of the brace and significant slip at the brace fasteners for SF of 200 and 300%.
	Kim et al. 2006	X-bracing	Random vibration, SE 32 accelerogram (one of the SAC ground motions provided by Somerville et al. (1997)) SF from 2% to 100%	0.61 s	7.2%	2.76%	Permanent deformation of straps, local bucklings of the bottoms of columns and deformation of anchors
 	Shamim et al. 2010	DFP, CSP panels (one single storey CSP sheathed wall specimen was also sheathed with gypsum wallboard on the opposite side) and Steel panels	Free vibration tests20005, NBCC elastic design spectra representative of M7.0 earthquake in Quebec City	From 0.27 s to 0.36 s and from 0.53 s to 0.67 s respectively for single and double storey wood sheathed walls From 0.23 s to 0.33 s and from 0.48 s to 0.59 s respectively for single and double storey steel sheathed walls	6.0% average value for wood sheathed walls 7.6% average value for steel sheathed walls	*2.48% and 2.50% respectively for single-storey and double-storey wood sheathed walls *2.00% and 2.80% respectively for single-storey and double-storey steel sheathed walls	Bearing damage in the sheathing surrounding the screw, screw pull-through sheathing, sheathing tear-out, screw shear failure and stud local buckling for both wood and steel sheathed walls and screw pull-out accompanied with chord stud flange twisting for some steel sheathed walls
	Schafer et al. 2016	OSB panels	White – noise tests, Canoga Park record, 1994 Northridge earthquake SF: 100%	0.15 s (only structure) 0.32 s (full building, i.e. structure and non-structural components)	4% only structure 9% complete building	1.18% only structure (for 100% Canoga Park record) 0.67% complete building (for 100% Rinaldi record)	Damage only in the interior non-structural walls, largely confined to corners near openings
	Hutchinson et al. 2016	Steel panels + gypsum panels for fire protection	Rio Dell Overpass from the 1992 Cape Mendocino earthquake, Canoga Park and Rinaldi Receiving Station both from the 1994 Northridge earthquake, and Curico from the 2010 Maule earthquake in Chile	0.28 s at the beginning 0.54 s after design event 0.84 s after the 100% EQ event	From about 7% to 20% for longitudinal mode From about 6% to 13% for torsional mode	0.89% (for 100% Canoga Park record) 1.70% (for 150% Canoga Park record) 12.15% (for 150% Rinaldi Receiving Station record)	Gypsum bulging on the bottom edge, gypsum screw popping ,buckling of sheet steel of corridor shear wall at level 1 and 4, sheathing panel crushing between the shear wall and gravity wall at level 4 corridor and buckling of rim track flange above gravity wall at level 4

SF: scaling factor
DFP: Douglas Fire Plywood, CSP: Canadian Softwood Plywood

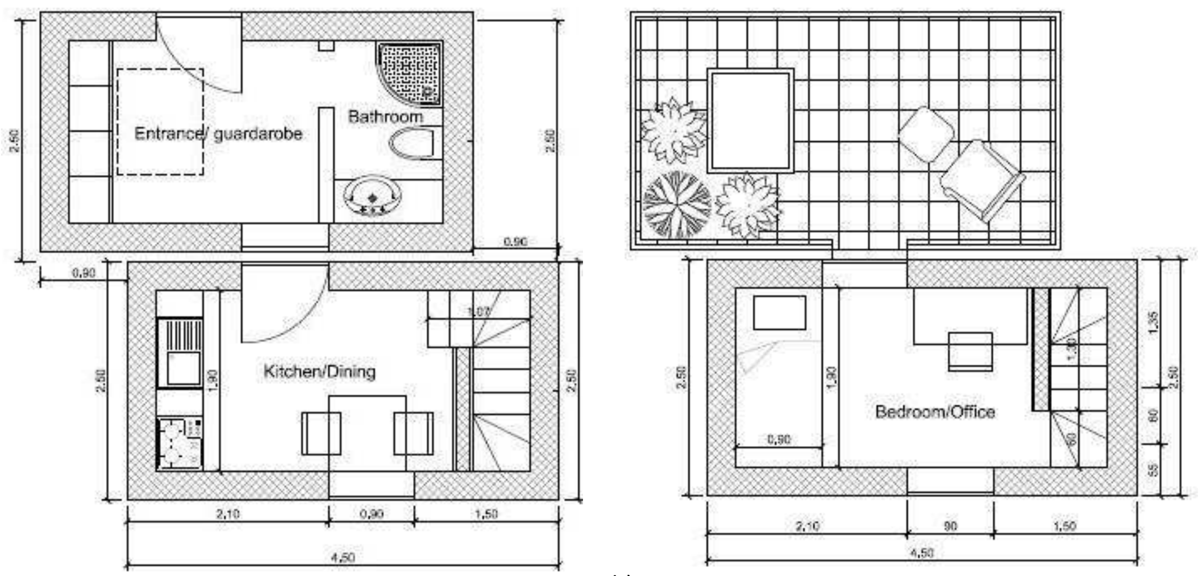
*Storey displacement average of the maximum and the minimum storey displacements

This paper summarizes the overall ELISSA research effort at the University of Naples, which spanned from small-scale tests on connections, to full-scale tests on walls, and final shaking table tests of a two-storey building. The ultimate objective was identifying the seismic response of the ELISSA proposed structural solution, in which the lateral force resisting (seismic resistant) system is based on CFS floors and walls sheathed with gypsum-based panels. One of the peculiarity of the investigated system is the use of quick connecting systems, composed of clinking connections between steel profiles and ballistic nails between steel and gypsum panels. These connection systems were preferred to traditional connection systems to speed up the construction process and minimize the possible errors. Section 2 introduces the ELISSA house architectural and structural concepts. Section 3 summarizes the experimental program, aiming at characterizing the seismic capacity and dynamic identification of the structural system. Section 4 provides the experimental database of the monotonic and cyclic performance of the main connecting systems, namely clinching for steel-to-steel connections and ballistic nails for panel-to-steel connections (micro-scale tests). Section 5 analyses the monotonic and cyclic tests on full-scale shear walls (macro-scale tests). Section 6 details the shake table tests of the ELISSA house full-scale two-storey building. Finally, Section 7 examines the key findings and the future research needed to develop cold-formed steel systems in seismic area.

2. CONCEPTUAL DESIGN OF THE ELISSA HOUSE

Central to the research project was the conceptual design of the “ELISSA House” (Figure 1), a two-storey building. The concept has been developed based on two main constraints: the house aimed to represent a real-life condition, able to show case and contain all the required equipment for a single person dwelling; and, the dimensions in plan and elevation were defined in order to allow the production of a full-scale prototype to be tested in the laboratory of the Department of Structures for Engineering and Architecture at the University of Naples Federico II.

The ELISSA house was made of three modules that were horizontally and vertically jointed (Figure 2). In a single floor module, the entrance with wardrobe and the bathroom are located, while in a two-storey floor module, the kitchen / living area is located on the ground floor and a single bedroom is arranged on the second floor. Each module has a 2.5 x 4.5m plan. The total usable area is of 34m^2 plus a terrace accessible from the bedroom and located on the roof of the single storey module. The maximum height is 5.4m. Light and fresh air are guaranteed through the main door and ceiling window in the single storey module and through windows and balcony in the two- storey building.



a) b)

Figure 1. Elissa house plans: a) first floor, b) second floor

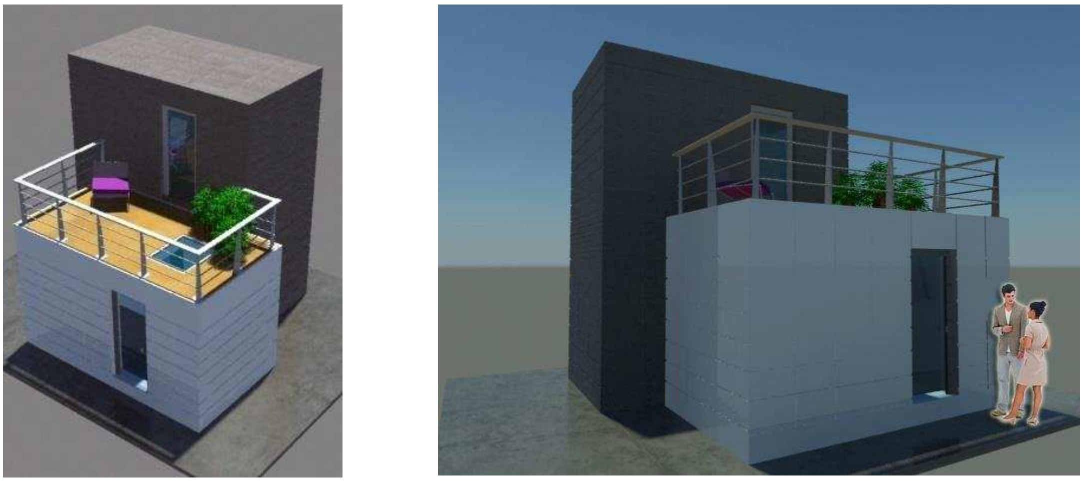


Figure 2. “ELISSA house” axonometric views.

From the structural viewpoint, the load-bearing structure of the ELISSA house is based on CFS frames (walls and floors), obtained by the “Transformer” system patented by COCOON (by Haring Nepple AG), and sheathed with gypsum-based board panels produced by KNAUF (Diamant boards for walls and GIFAfloor boards for floors). The main peculiarity of the “Transformer” system compared to other CFS typologies is the adoption of back-to-back coupled C sections for the floor joists, which allows a limited depth of floors also for long spans, and the adoption of ballistic nails between sheathing panels and steel profiles to reduce construction time of walls and floors. The “Transformer” system obtained the European Technical Approval for static loads [30], and specific objective of the ELISSA research project was to study its potential seismic performance and propose its upgrades to be adopted in seismic area.

The load bearing structures consisted of floors and walls built with CFS profiles made with S320GD+Z steel (characteristic yield strength: 320 MPa, characteristic ultimate tensile strength: 390 MPa) sheathed with gypsum based panels.

Floors and roof (Figure 3) were made of back-to-back coupled 197x50x2.0 mm (outside-to-outside web depth x outside-to-outside flange size x thickness) C (lipped channel) section joists spaced at about 500 mm on the center. The joists were connected at the ends to 200x40x1.5 mm (outside-to-outside web depth x outside-to-outside flange size x thickness) U (unlipped channel) section floor tracks. The connections among the steel profiles were made by 4.8 mm diameter self-drilling screws. Top side of floors and roof were sheathed with 28 mm thick Knauf GIFAfloor boards (gypsum fiber panels). The GIFAfloor boards were glued together with a polyurethanic adhesive (Knauf klebstoff). They were connected to floor steel frame by means 3.4 mm diameter ballistic nails spaced at 100 mm.

Similar to the floors, the walls (Figure 3) were made with studs having 147x50x1.5 mm C sections. The studs were connected at the ends to 150x40x1.5 mm U section wall tracks. The connections among the steel profiles were made by 4.8 mm diameter self-drilling screws. The wall steel frame was sheathed with 15 mm thick Knauf Diamant boards (impact resistant gypsum panels) on both sides. Sheathing panels were connected to wall steel frame by 2.2 mm diameter ballistic nails spaced at 150 mm both at the field and at the perimeter of the panels.

In order to withstand the axial force due to overturning phenomena, in the ELISSA house, ad hoc designed hold-down devices were placed at the ends of wall segments. The hold-down devices were connected to studs by four M22 (8.8 grade). The tension connections at the wall ends were made of M20 (8.8 steel grade) bolts or threaded rods. In particular, for the ground floor each bottom hold-down of first level walls was connected to the foundation structure by a M20 bolt, as well as for the first floor each top hold-down of first level walls was connected to the relevant bottom hold-down of second level walls by a M20 threaded rod. The shear connection was obtained by connecting wall and floor tracks by 5.5 mm diameter self-drilling screws spaced at 200 mm. The shear connections between the tracks of ground floor and the foundation structure were made by M10 bolts (8.8 steel grade) spaced at 300 mm.

The different layers used in order to obtain the complete construction (with finishing) are shown in Figure 3 for floors and walls.

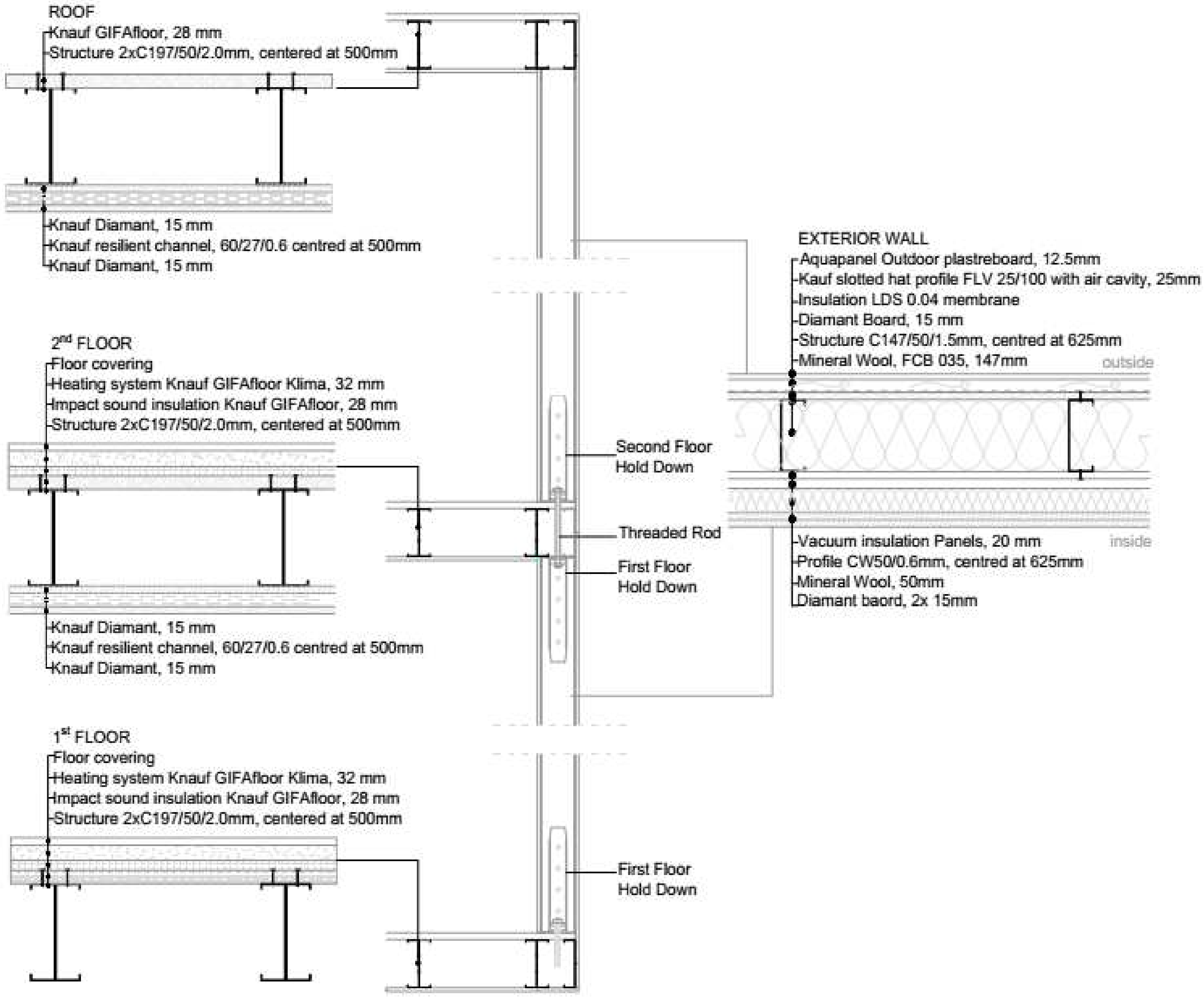


Figure 3. “ELISSA house” section with details of floors and external walls.

2.1. STRUCTURAL DESIGN AND ANALYSIS

The structural design was performed according to the European codes EN 1991 – Part 1.1 [31]for the actions definition, EN 1993 – Part 1.1 [32] and Part 1.3 [33] for the design of steel structures, EN 1998 – Part 1 [34] for the seismic design, wherever applicable. Dead loads of main constructive components, live loads, snow and wind loads are summarized in Table 2. With respect to the horizontal seismic actions, the structural design of ELISSA house was carried out following the “sheathing braced” approach, which considers the interaction between steel profiles and sheathing panels. Under this hypothesis, floor decks and walls act as diaphragms [Dubina et al. 4] Moreover, horizontal actions acting on the lateral-resisting walls were evaluated according to the “Segment method”, that considers as resistant only the wall segments without openings [24] (Figure 4). Walls are considered as cantilever vertical diaphragms subjected to horizontal force acting on top edges, and at the end of each resisting wall, hold-downs are located to avoid any overturning phenomena. The global structural response of the wall diaphragms depends on the local response of the wall components (steel studs, anchors, sheathing panels and steel – to – sheathing panel connections). For each wall component, it is possible to individuate the failure mechanisms and the smallest associated strength value defines the lateral wall resistance. When a CFS wall is under horizontal action, the chord studs are subjected to bending deformation, the steel frame deforms in a parallelogram, the sheathing tends to rigidly rotate, the sheathing fasteners are subjected to the relative displacement between steel frame and the rigid panel rotation (Figure 5). Previous studies demonstrates that a good ductile behaviour is achieved when the steel – to – sheathing connections governs the behaviour. Hence, for the ELISSA house capacity design criteria have been adopted. Studs, anchors and panels have been considered as non-dissipative elements, while the steel – to – sheathing connections have been considered as dissipative elements, and for this purpose, the spacing of the fasteners was chosen in such a way that nonlinear behaviour occur in the

connection prior in other (non-dissipative) building elements. For the seismic action distribution, floors were considered as in-plane rigid diaphragms.
Note that the only steel skeleton without sheathing would have very low capacity and rigidity values, as demonstrated by Fulop et al 2004 [2].
Taking into account that no specific regulations for the seismic design of CFS structures are provided by the EN 1998 – Part 1 [34], a behaviour factor equal to 3.0 and an over-strength factor equal to 1.2 were initially assumed according to previous research [16]. Later, based on the results of tests on full-scale shear walls representative of the ELISSA mock-up the over-strength factor was increased up to 1.8.
The reference period for the seismic action was assumed equal to 50 years (nominal service life for ordinary buildings) and the seismic designed was carried out according to the following performance objectives: (1) Immediate Occupancy (IO), for earthquakes having 63% probability of exceedance in 50 years and (2) Life Safe (LS), for earthquakes having 10% probability of exceedance in 50 years. The seismic intensity was assumed according to typical European values for areas with medium seismic hazard and 10% in 50 years probability of exceedance, equal to 0.25 g.The foundation soil was assumed as Type B. The values assumed for 5% damped elastic spectra in the design phase in terms of PGA and maximum spectral acceleration corresponding to different hazard levels are given in Table 3.
Seismic effects were calculated through the modal dynamic analysis, considering the modes Complete Quadratic Combination (CQC) technique. The earthquake was conventionally assumed as acting separately in the two plane main directions perpendicular to each other. In the analysis a structural damping of 5 % is assumed. Accidental torsional effects are considered by a linear redistribution of the storey loads in relation to the accidental eccentricity.

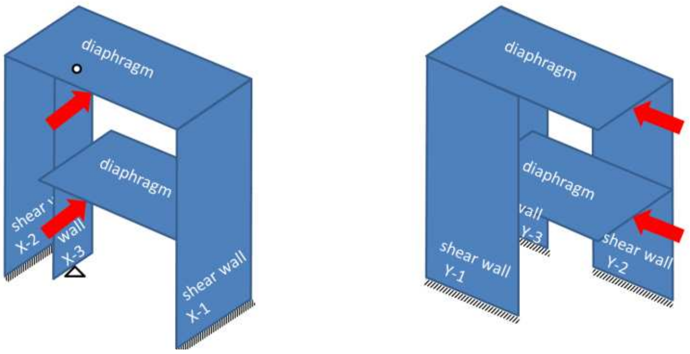


Figure 4. Structural model for horizontal load transfer

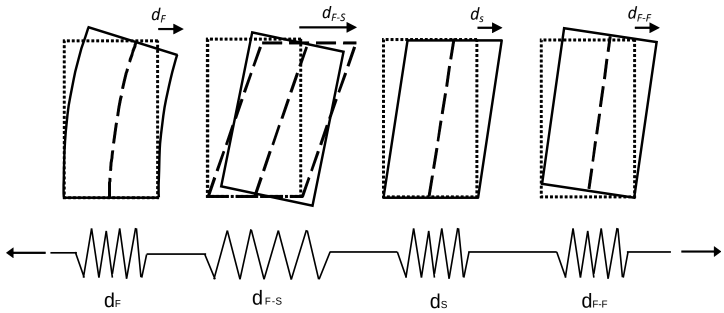


Figure 5. Deformation contribution of a sheathed wall under horizontal loads

Table 2. Unit loads and factors for loads combinations assumed in the design phase.

Load type	G_1	G_2	Q_i	Ψ_{2i}
	kN/m^2			
Dead – external walls	0.40	0.70		
Dead – internal partition walls		0.40		
Dead – first floor	0.40	0.50		
Dead – second floor	0.60	0.60		
Dead – roof	0.60	0.20		
Live – floors (residential)			2.00	0.24
Live – roof (not accessible except for normal maintenance and repair)			0.40	0.00
Snow			2.00	0.00
Wind			0.85	0.00

Table 3: Parameters for the seismic design

Hazard Level (probability of exceedance)	**PGA [g]**	**Maximum Spectral Acceleration [g]**
63% in 50 years	0.12	0.29
10% in 50 years	0.29	0.72
2% in 50 years	0.43	1.08

3. EXPERIMENTAL PROGRAM

The main objective of the experimental phase is the evaluation of seismic response of the structural system. In this structural typology the seismic response is significantly influenced by the behavior of shear walls, which represent the main lateral seismic resistant elements. The global seismic response of this shear walls is strongly affected by the local behavior of their components. Therefore, the experimental campaign has been articulated in three phases: 1. “micro – scale” tests on the main connection typologies; 2. the “meso – scale” tests on shear walls; 3. the shake table tests. Table 4 provides an overview of the full experimental program and in the following sections the tests set-up, instrumentation and results will be described by category.

Table 4. Test matrix for the overall ELISSA Project.**Typology**		**Load type**	**Total No. tests**
Connections	SS	Monotonic and cyclic	15
	SPW and SPF	Monotonic and cyclic	18
Walls	WS	Monotonic and cyclic	3
	WF	Cyclic	1
Mock-up	Bare structure	White-noise	5
	Complete Building	White noise and Earthquake tests	50

SS : Steel-to-steel; SPW: Steel-to-Panel connections for Walls; SPF: Steel-to-Panel connection for Floor; WS: Shear walls without finishing materials; WF: Shear walls with finishing materials.

4. MICRO – SCALE TESTS: CONNECTION TESTS

The global behavior of the seismic resistant system depends on the local behavior of the basic components (steel members, sheathing panels and connection systems). In particular, connection behavior is crucial, especially when sheathing braced approach is adopted for the seismic design of lightweight steel buildings. As mentioned in the previous sections, the main connections involved in this construction systems are: clinching connection between steel members (SS) and ballistic nails for steel to panel connections (SP), and their behavior has been investigated to evaluate shear strength and stiffness.

Table 5 summarizes the experimental program for SS and SP connections, specifying the construction element in which are they used, the load type, the loading protocol and number of tests.

Table 5: Test matrix for the monotonic and cyclic tests on connections

Typology	Element	Load type	Loading protocol	No. tests
SS_M	Wall	Monotonic	-	5
SS_C	Wall	Cyclic	CUREE 2000	5
SS_E	Wall	Cyclic	ECCS 1986	5
			Total n. of SS tests	**15**
SPW_M	Wall	Monotonic	-	5
SPW_C	Wall	Cyclic	CUREE 2000	6
SPF_M	Floor	Monotonic	-	2
SPF_C	Floor	Cyclic	CUREE 2000	5
			Total n. of SP tests	**18**

SS : Steel-to-steel; SPW: Steel-to-Panel connections for Walls; SPF: Steel-to-Panel connection for Floor
_M: Monotonic, _C: CUREE 2000: CUREE Cyclic Protocol [35]*, _E: ECCS 1986: ECCS Cyclic Protocol* [36]

4.1. SS CONNECTIONS: CLINCHING

Clinching are often used in automotive manufacturing process, because of their improved fabrication efficiency. They are well known for their advantages in terms of: simplicity and cleanness of the process, low run time, reduced energy used, the possibility to automate the process, the easy quality checks and the lacking of fasteners or other consumables in the process (Lambiase [37]). As such, clinching is used in the Transformer system to simplify and automate the connection between steel profiles. Their shear response under monotonic loadings has been investigated in recent years by many groups as Pedreschi and Sinha [38], Di Lorenzo and Landolfo [39], Mucha and Witkowski [40] to cite but a few, however the response under cyclic loading is not been fully investigated. Therefore, this research investigated the response of clinching connections under monotonic and cyclic loadings. The test set-up adopted for clinched SS connections reproduces the connection between the stud and the track of the wall steel frames. Clinching joints are TOX-Round Joint with 8 mm diameter connecting two 1.5 mm thick steel sheets of S320GD grade (characteristic yield strength f_y: 320 MPa; characteristic ultimate tensile strength f_u: 390 MPa). The connection specimen consists of two CFS profiles connected together by means of two 8 mm diameter clinching points on the profile flanges (one clinching point per flange). The load was applied to the specimens by means of two steel holders, clamped to the test machine wedge grips and bolted at the ends of each profile. The tests were performed using a universal test machine, and two linear variable differential transducers (LVDTs) were used for measuring the relative displacement between the two steel profiles (C147/50/1.5mm and U150/40/1.5mm, Figure 6).

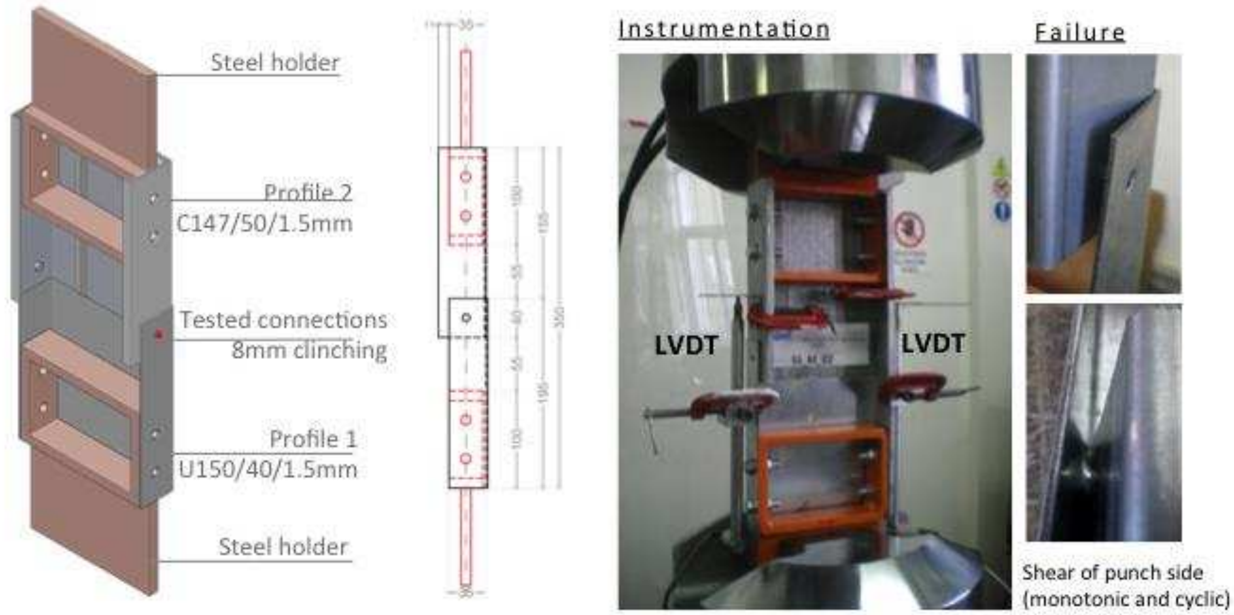


Figure 6: Test setup of an SS connection, real specimen and failure modes on the right.

Main results of the tests are presented in Table 6, where the load (*F*) is the average unit load, assumed as the total recorded load divided by the number of tested fasteners, which was equal to two, and the displacement (*d*) is the relative movement between the connected elements evaluated as the average of the recorded measures of the two LVDTs. In particular, the main parameters used to describe the experimental behaviour are: the connection strength corresponding to the maximum recorded load (F_p); the conventional elastic stiffness (k_e) assumed equal to the ratio between the conventional elastic limit load equal to 0.40 F_p and relevant displacement. In the case of cyclic tests, the result parameters were obtained as the average of the values evaluated on positive and negative envelopes. The results of tests on SS specimens in terms of load versus displacement curves are shown in Figure 7.

The characteristic strength value of the connections ($F_{p,k}$) are evaluated through the following formula:

$$F_{p,k} = F_{p,m} - k_s \quad (1)$$

where $F_{p,k}$ is the characteristic value, $F_{p,m}$ is the average value, s is the standard deviation and k is a coefficient depending on the number of performed tests, which is equal to 2.33 for 5 tests and 2.18 for 6 tests, according to the EN 1993-1-3 (2007).

The failure mode is the shear of the punch side for both monotonic and cyclic tests. The comparison between the results of monotonic and different cyclic protocols shows that the strength and stiffness reduction due to cyclic is very low for CUREE protocol, specifically -3% and -2%, respectively, Higher reduction are recorded in the case of ECCS protocols, in which the reduction are equal to -10% for strength and -16% for stiffness. On the basis of obtained results, the suggested characteristic value of the shear strength under cyclic loads per single 8 mm clinching point connecting two 1.50 mm thick S320 steel grade profiles can be set equal to 3.00 kN. More info about tests on connections can be found in Fiorino et al. [21].

Table 6: Average test results for SS connections.

Loading		***k_e* (kN/mm)**	***F_p* (kN)**
Monotonic	**Average**	91.5	3.38
	CoV	0.22	0.01
	Characteristic		**3.05**
CUREE Cyclic protocol	**Average**	90.1	3.30
	CoV	0.33	0.02
	Characteristic		**3.15**
ECCS cyclic protocol	**Average**	76.5	3.06
	CoV	0.19	0.02
	Characteristic		**2.94**

k_e: conventional elastic stiffness
F_p: connection strength

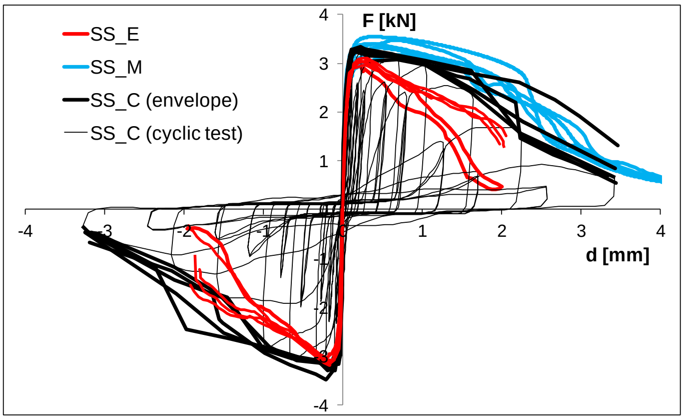


Figure 7: Experimental load vs. displacement curves for SS connection specimens

4.2. SP CONNECTIONS: BALLISTIC NAILS

The fasteners used for panel-to-steel connections in shear walls were ballistic nails, which consist of compressed air-driven special nails with ballistic shape of the nail tip and knurled shank. Main advantage of this connecting system is the optimization of the assembling operations allowing a more efficient level of prefabrication. Although ballistic nails are quite typically used in dry construction as connection system between steel and panels, there is no codified design procedure to predict their strength, therefore, experimental tests are required for structural applications. Wall panel-to-steel connections (SPW) were made of 1.50 mm steel thick profiles, 2.2 mm diameter ballistic nails and 15 mm thick impact resistant gypsum board panels, whereas floor panel-to-steel connections (SPF) were made of 2.00 mm thick steel profiles, 3.4 mm diameter ballistic nails and 28 mm thick gypsum fiber panels.

For panel-to-steel connections two different set-ups were used (Figure 8, Figure 9). Both set-ups were a modified version of the set-up defined by EN 520 [41] for connections between gypsum-based panel and wood frame. Each panel was connected to the profile by two ballistic nails with edge distance and spacing equal to 70 mm at the top side (tested connection), whereas four ballistic nails were used to connect the bottom side panel to the profile (oversized connections). The load was applied by means of two steel holders bolted to the studs. Tests on connections were conducted under displacement control, involving three different loading protocols: monotonic, ECCS cyclic and CUREE cyclic.

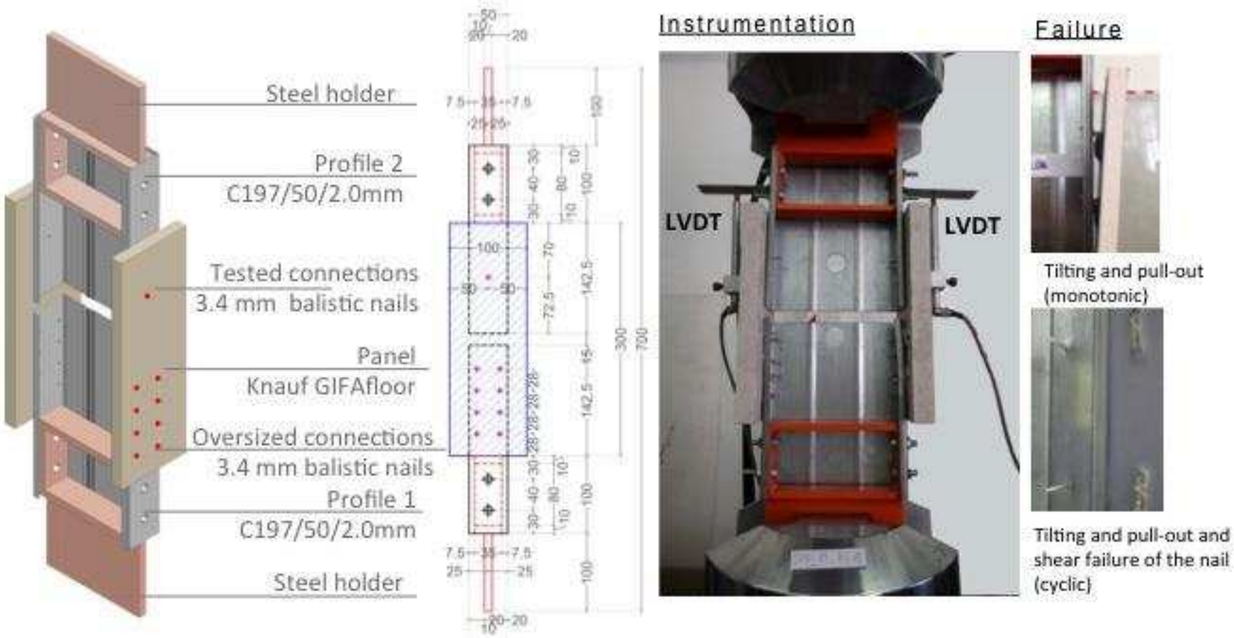


Figure 8: Test set-up for wall SPW connections, real specimen and failure modes on the right.

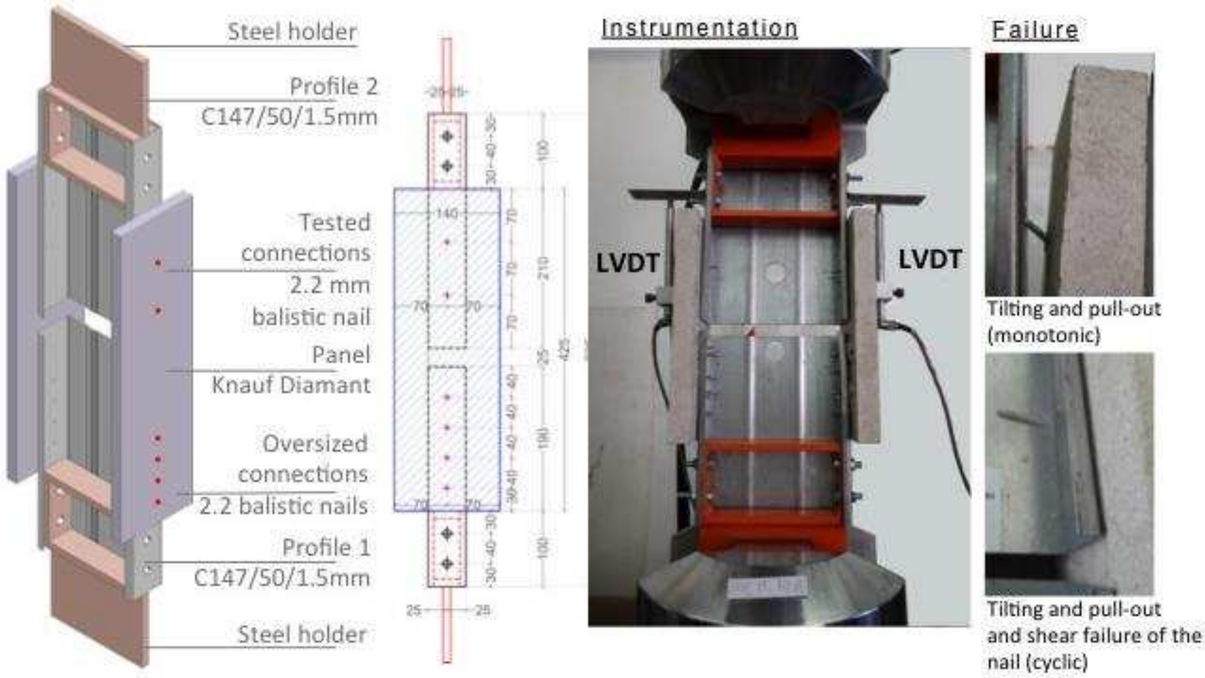


Figure 9: Test set-up for floor SPF connections, real specimen and failure modes on the right.

The tests are performed by a universal test machine and two linear variable differential transducers (LVDTs) with a maximum displacement of 50 mm are used for measuring the relative displacement (*d*) between the stud (Profile 2 in fig 9) and the panels. Figure 10 shows the experimental load vs. displacement curves. In this case, the average unit load (*F*) is obtained as the total recorded load divided by number of the tested connections (i.e. four).
For SPW connections the failure mode was always the tilting and the pull-out of the nail from the profile but, in the case of cyclic tests, it is followed by the shear failure of the nail. The results (Table 7) in terms of strength (F_p) show a reduction due to cyclic loads of -16%. In terms of stiffness (k_e), the results of cyclic loads show an increase of 68% than monotonic values.
Also in the case of SPF connections, the failure mode is the tilting and pull-out of the nail and, in some cyclic tests, the shear failure of the nail occurred. A strength reduction of -14% for cyclic tests is observed. A similar trend is detected for stiffness values with a reduction of -12% in cyclic tests.
Characteristic values of the shear strength under cyclic loads per single 2.2 mm ballistic nail connecting a 1.50 mm thick S320 steel grade profile to a 15 mm thick impact resistant gypsum board panel can be set equal to 0.80 kN. Characteristic value of the shear strength under cyclic loads per single 3.4 mm ballistic nail connecting a 2.00 mm thick S320 steel grade profile to a 28 mm thick gypsum fiber panel can be set equal to 4.30 kN. More info about tests on connections can be found in Fiorino et al. [21].

Table 7 Test results for SPW connections

Loading		k_e **(kN/mm)**	F_p **(kN)**
Monotonic	**Average**	0.50	1.07
	CoV	0.45	0.02
	Characteristic		**1.00**
CUREE Cyclic protocol	**Average**	0.84	0.90
	CoV	0.15	0.04
	Characteristic		**0.83**

k_e : conventional elastic stiffness
F_p : connection strength

Table 8 Test results for SPF connections

Loading		k_e **(kN/mm)**	F_p **(kN)**
Monotonic	**Average**	1.53	5.54
	CoV	0.30	0.01

CUREE Cyclic protocol	**Average**	1.35	4.76
	CoV	0.09	0.04
	Characteristic		**4.36**

k_e : conventional elastic stiffness
F_p : connection strength

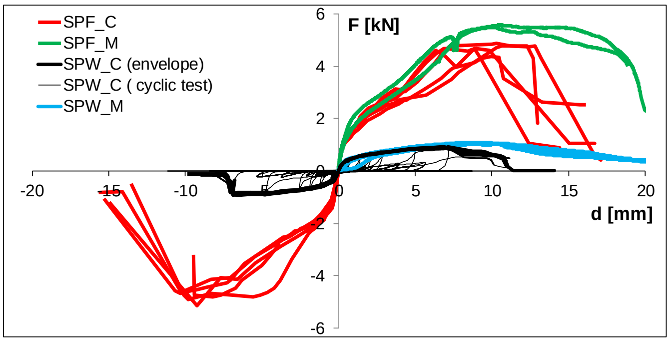


Figure 10: Experimental load vs. displacement curves for SPF and SPW connection specimens

5. ELISSA SHEAR WALL TESTS

Tests on shear walls were aimed at investigating the behaviour of the seismic resistant system of the ELISSA house. In particular, four tests on full-scale shear walls were performed. The wall configurations were selected in order to evaluate the influence of the aspect ratio (different wall length), the type of loading (monotonic and cyclic) and the effect of the presence of finishing materials. The wall test program is summarized in Table 10, in which each tested configuration is illustrated. Only tests on 2.4 m long walls have been considered in this paper because test results obtained for the specimen WS_4100_C (4.1 m long wall) were affected by the presence of imperfect connections between the panel edges and the internal studs, as discussed in Macillo et al [17].

Table 10. Shear tests on walls

Label	Geometry (length x height)	Finishing	Loading protocol	No. Tests
WS_2400_M	2.4 m x 2.3 m	NO	Monotonic	1
WS_2400_C	2.4 m x 2.3 m	NO	Cyclic	1
WS_4100_C	4.2 m x 2.3 m	NO	Cyclic	1
WF_2400_C	2.4 m x 2.3 m	YES	Cyclic	1
			Total	**4**

The structural elements of wall specimens were defined on the basis of the seismic design of the ELISSA mock-up. The seismic action was evaluated by assuming a reference period of 50 years (nominal service life for ordinary buildings) and an intensity corresponding to a typical value of European areas with medium seismic hazard, i.e. a reference peak ground acceleration (PGA) of 0.25 g for Type A soil (10% probability of exceedance in 50 years) for life safety limit state. The foundation soil was assumed as type B. The total seismic weight of the building was 76 kN and, considering the maximum acceleration on the design spectrum of 0.24 g (0.24 g = 0.72/3.0, i.e. maximum acceleration on the elastic spectrum divided by assumed behaviour factor), the total seismic demand in terms of base shear was 18 kN (18 kN = 0.24 · 76 kN). The distribution of seismic actions acting on each wall was evaluated according to the "Segment method", considering floors as in-plane rigid diaphragms. The design was carried out on the walls of the first floor oriented along the short side of the ELISSA mock-up plan (2 walls having a length of 2.4 m corresponding to a total wall length of 4.8 m), then the seismic demand per unit wall length (H_d) was 3.8 kN/m (3.8 kN/m = 18 kN/4.8 m). This action was compared to the unit wall lateral design capacity (H_c) evaluated as the wall resistance associated to the shear failure of panel-to-sheathing connections, which represents, the most ductile mechanism and, according to capacity design criteria, it should govern the whole wall strength. In particular, the unit wall lateral capacity can be obtained as follows:

$$H_c=n/s\cdot k_{mod}\cdot F_{p,k}/\gamma_M=11.7\text{kN/m} \qquad (2)$$

with: n=2 number of wall side sheathed with structural panels; s=0.15m panel-to-steel connection spacing along wall tracks; $F_{p,k}$=0.80kN characteristic shear strength of panel-to-steel connection, assumed according to cyclic test results; k_{mod}=1.10 coefficient dependent on loading duration and moisture content, assumed according to EN 1998-1 [34] and EN 1995-1-1 [42] ; γ_M=1.00 partial safety factor assumed according to EN 1998-1 [34] and EN 1995-1-1 [42].

Note that, since the demand-to-capacity ratio is low (3.8/11.7 = 0.32), the walls would seem oversized. This is related to the assumed spacing of panel-to-steel connections (150 mm), which is the technological upper limit. Greater spacing,

corresponding to higher and more usual values of demand-to-capacity ratios (e.g. from 0.7 to 0.9) is not allowed in common practice.
In order to satisfy the capacity design criteria, the non-dissipative wall elements (chord studs, tracks, sheathing panels, tension and shear anchors) were designed considering an over strength factor equal to 1.2.
Tests on full-scale wall specimens were carried out by using a specifically designed testing frame for in-plane horizontal loading Figure 11. Two potentiometers were used to record the horizontal displacements of the loading beam (W1) and at the top of the wall (W2), and four LVDTs measured the vertical (L1 and L3) and horizontal (L2 and L4) displacements at the bottom corners of the wall. A load cell was used to measure the applied loads. Tests on wall prototypes were conducted under displacement control in quasi-static monotonic and reversed cyclic regime. Under monotonic loading history, specimens were subjected to progressive displacements up to failure. Cyclic tests followed the "CUREE ordinary ground motion reversed cyclic load protocol". This loading procedure (Figure 12) is a reversed cyclic protocol, developed for wood-frame structures by Krawinkler et al. [35]. The displacement amplitudes of each cycle were defined starting from a reference displacement $\Delta = \gamma\, \Delta_m = 39.0mm$, where the values of Δ_m was calculated on the basis of monotonic test results. The considered displacement rate involved displacements at a constant rate of 0.50 mm/s up to cycle 28 (maximum applied displacement equal to 9.0 mm) and 2.00 mm/s for cycle 29 and higher. The typical experimental response in terms of acting load (*H*) vs. top displacement curve (*d*) of the 2.4 x 2.3 m shear wall under monotonic and cyclic load are provided in Figure 13.

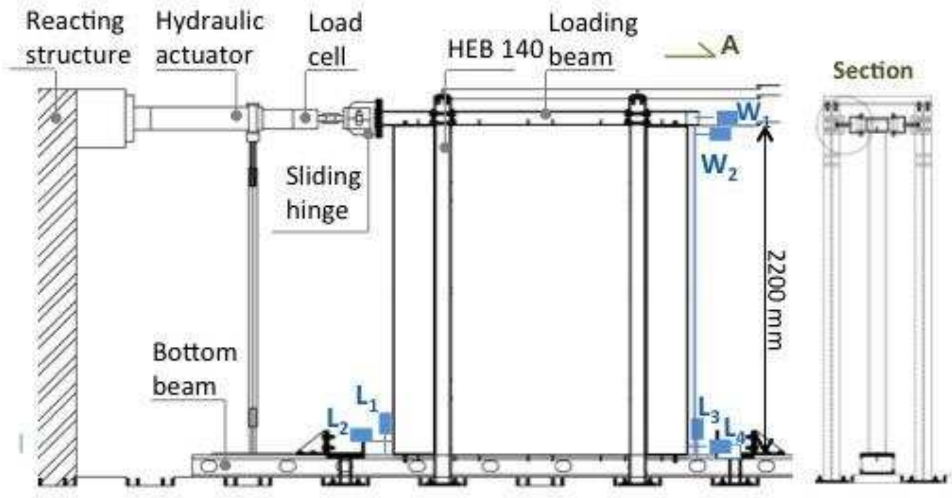


Figure 11. Wall test set-up.

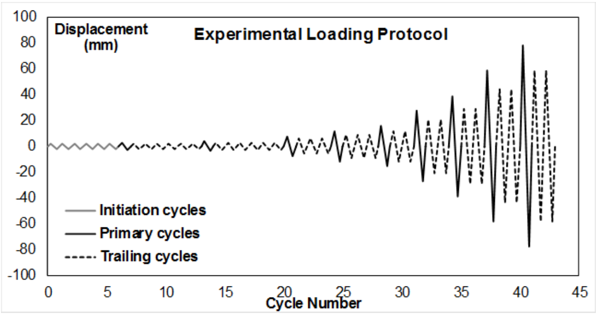


Figure 12. CUREE cyclic protocol

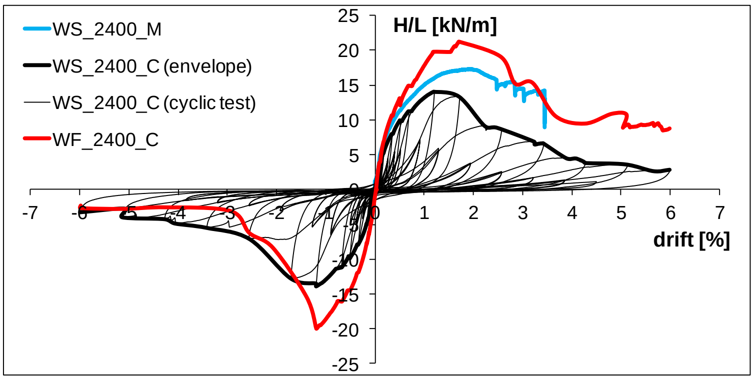
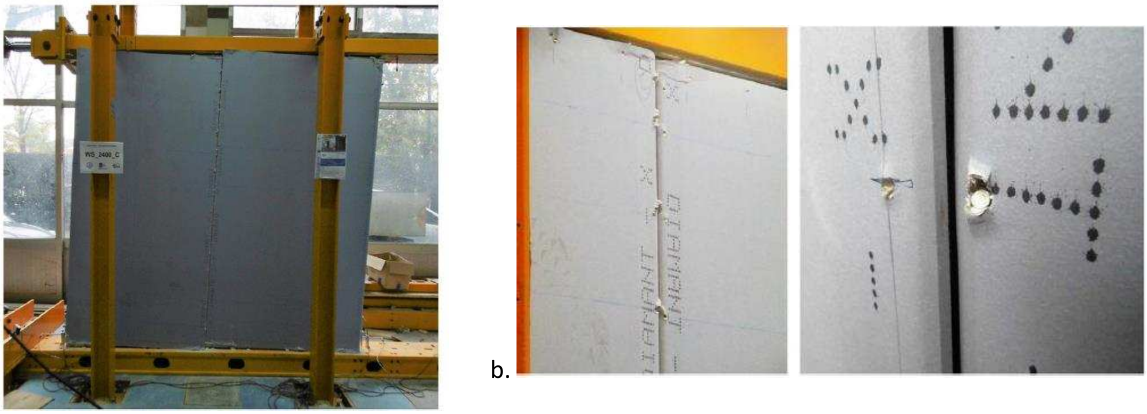


Figure 13 Experimental response curves for WS_2400, WS_2400_C and WF_2400_C wall specimens



a. b.

Figure 14. Wall collapse: a. overall deformation, b. details of the connection failures.

Test results revealed that for all specimens the wall collapse was governed by the sheathing-to-frame connections with the tilting and pull-out of the nails, as shown in Figure 14. At global level, the steel frame deformed as a parallelogram with a consequent rigid rotation of the sheathing panels, as shown in Figure 14a. In case of wall completed with finishing (WF_2400_C), the detachment of the sheathing panel together with the wall lining of the internal face was also occurred for displacements higher than 100 mm. As far as the cyclic tests are concerned, the experimental curves showed a substantially symmetrical response in the two loading directions

Although only one test for each wall configuration was performed, test results were compared in order to understand the influence of the type of loading and the effect of the presence of finishing materials. In order to evaluate the influence of the cyclic loads, the results of the WS_2400_M and WS_2400_C specimens were compared. In particular, the experimental results showed that, in the cases of cyclic loads, the unit wall strength decreased of 20% in average with respect to the monotonic results. This evidence is coherent with the results of connection tests, where the cyclic loads affect the strength with a reduction of 16%. The values of the wall stiffness in cyclic test showed a reduction of 32% with respect to monotonic one. The effect of non-structural parts and finishing on the lateral response of the wall can be evaluated by comparing the results of WF_2400_C and WS_2400_C. In particular, the presence of the finishing produced an increment of 48% for the wall strength and 39% for wall stiffness. For this reason, the value of the over-strength factor initially assumed for the design of the ELISSA mock-up, equal to 1.2, was increased up to 1.8 ($1.2 \cdot 1.48 = 1.8$). More info about tests on walls can be found in Macillo et al [17].

6. ELISSA FULL SCALE MOCK-UP TESTING

Full scale testing of the ELISSA mock-up was conducted in July 2016 at the University of Naples Federico II. The ELISSA mock up consists of the two-storey building presented in Section 2, in which the structural system was designed as "all steel" solution for vertical loads and "sheathing braced solution" for horizontal actions. The testing was conducted in two phases: in a first phase only the bare structure was tested (Figure 15), while in the second phase the building was completed with all the non-structural components before testing (Figure 16). For the sake of clarity, in the first phase, the building was composed of the load bearing walls and floors realized with structural steel members and the structural panels. In addition, the bottom side of the second floor and roof were sheathed with thick impact resistant gypsum board panels (Diamant Knauf), and also the internal partitions were built. The total weight of the mock-up in the first phase was 46 kN, while the total weight of the complete building in the second phase was 99kN. The seismic weights were 38 kN and 76 kN for the bare structure and the complete building, respectively, to which a supplementary mass of 5 kN was added on the second floor, in order to simulate the design seismic weight corresponding to the variable loads. The fundamental period of the mock-up was estimated equal to 0.17s, according to the EN1998-Part 1 [34].

The shaking table at the laboratory of the Department of Structures for Engineering and Architecture at the University of Naples has a 3.0 x 3.0 m base, a maximum payload of 2000kN, frequency range of 0-50Hz, peak velocity of 1m/s and displacement in the range of +/- 250mm. To allow the mock-up house to be tested on the table, it was necessary to design and realize an extension of the table, which would be able to accommodate the 2.7m x 4.7m building. For this reason, a very stiff 3D reticular steel structure was developed and on this an intermediate structure, made of stiffened C-profile was accommodated. The mock-up was connected to the extension table through the intermediate structure (Figure 17).

The building was tested with horizontal-uniaxial motions in the transversal plane (short side of the building) through the L'Aquila Earthquake occurred on the 6th April 2009 in Italy, corresponding to an event with Richter magnitude of 5.9. The original ground motion has a PGA of 6.44 m/s^2 and the corresponding accelerogram is shown in Figure 18a. The ground motion was scaled by a factor of about 20% for the probability of occurrence equal to 63% in 50 years (corresponding to immediate occupancy design level), 50% for the probability of occurrence equal to 10% in 50 years (corresponding to the life safety design level), and 75% for the probability of occurrence equal to 2% in 50 years (corresponding to the near collapse design level), as shown in Figure 18b. The response of the building during testing was captured by twelve triaxial accelerometers and nine laser sensors (Figure 19).

The results of the dynamic identification tests were used to define the dynamic properties, such as the fundamental period and damping ratio (Figure 20). The fundamental period of the bare structure before earthquake tests (undamaged) was in the range from 0.12 to 0.13 s, whereas the fundamental period of the complete building before earthquake tests (undamaged) was about 0.10 s. This indicates that the finishing material decreased the fundamental period of about 20% and increased the stiffness of the building from three to four times. After the earthquake tests, due to the damage, the dynamic response changed and an increment of the fundamental period was registered up to a period equal to 0.17 s (+70%) at the end of all the earthquake tests, which corresponds to a decreasing of the lateral stiffness equal to about 70%. The damping ratio ranged between 1.4% and 3.1% for the bare structure (undamaged) and between 1.2% and 2.0% for the complete building before earthquake test (undamaged). After the earthquake tests (damaged) the damping ratio ranged between 1.7% and 5.4%.

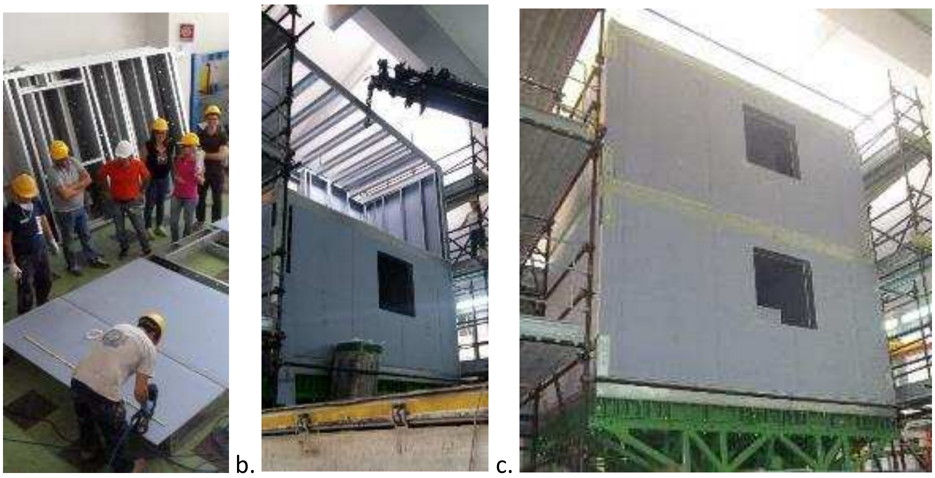

a. b. c.

Figure 15: Mock-up construction: a. first floor installation, b. second floor and wall, c. full bare structure

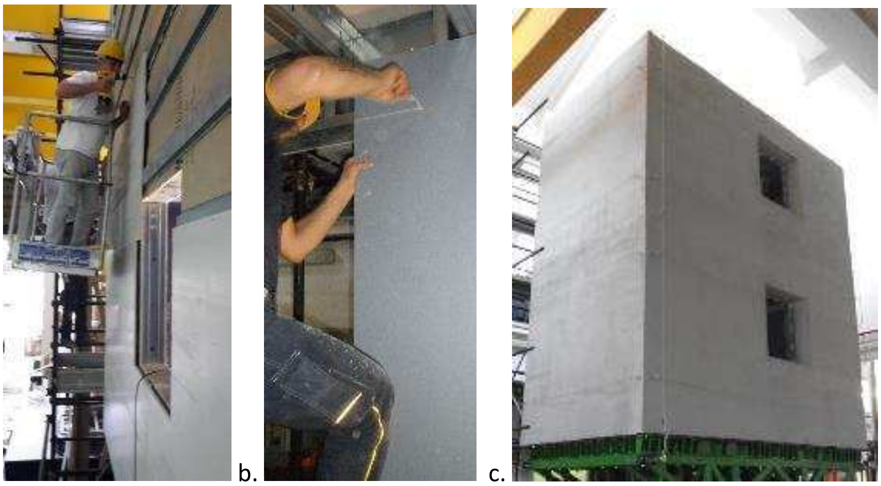

a. b. c.

Figure 16 Completed building: a. external wall panel fixing, interior wall panel fixing, final building view

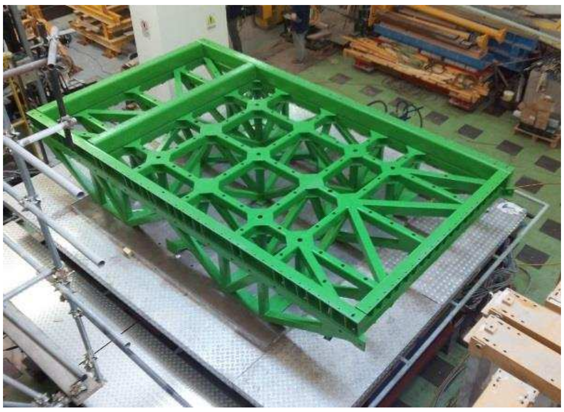
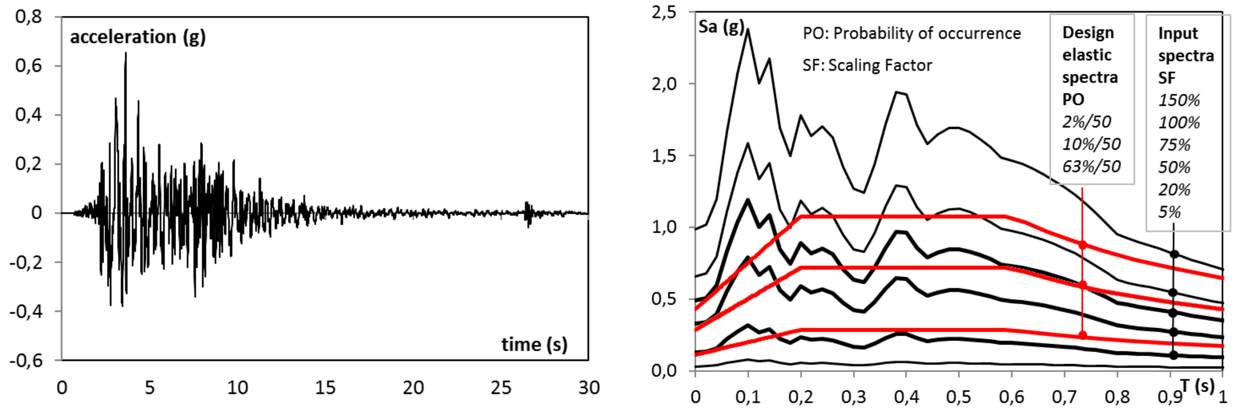


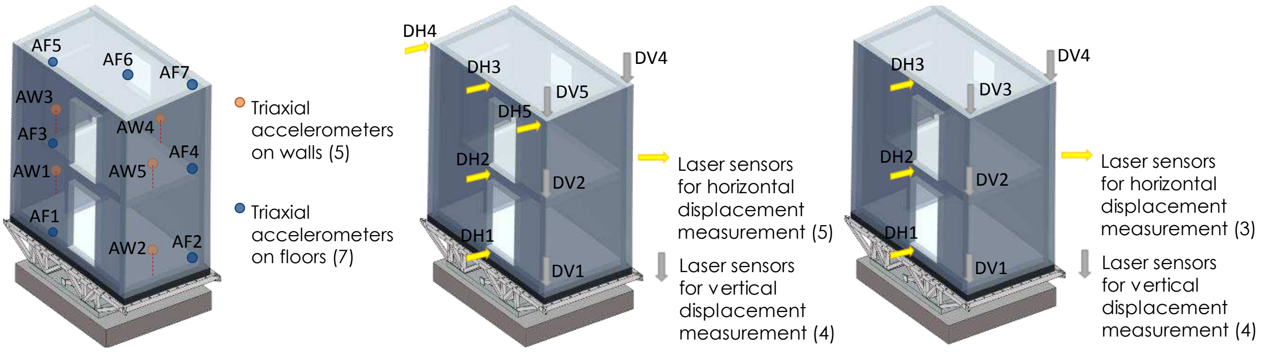
Figure 17: Extension and intermediate structure on the shaking table



a. natural accelerogram (record from the AQV Station Est-West component)

b. Input spectra vs. design spectra

Figure 18. Ground motion selection.

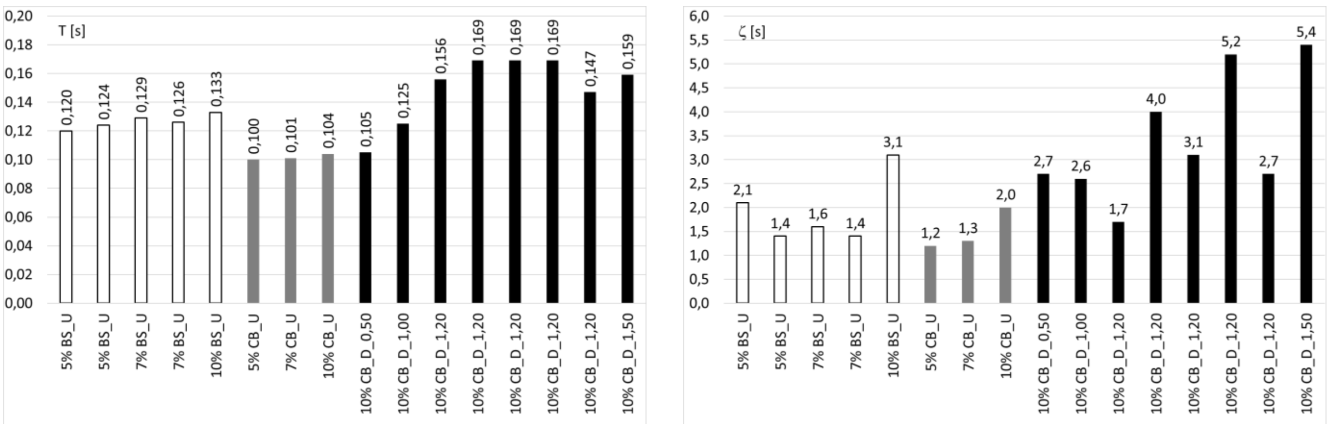


a. First phase - Inst. conf. 1

b. Second phase - Inst. conf. 2

c. Second phase - Inst. conf. 3

Figure 19. Instrumentation



a. fundamental period

b. damping ratio

Figure 20. Results of dynamic identification.

The earthquake tests were performed only on the complete building and the seismic response was evaluated mainly in term of inter-storey drifts, damages on the building, acceleration amplification, and roof diaphragm behaviour. The measurement of the horizontal displacement at each building level showed that the inter-storey drifts were very small, with a maximum of 0.80% for the first storey and 0.52% for the second storey (reached for scaling factor of 150%) and a minimum of 0.37% and 0.17% for the first and second storey, respectively (Fig. 21). Due to the very small inter-storey drifts reached during earthquake tests, damage was limited to non-structural system, in particular it consisted in presence of gypsum dust and small detachment of cover paper at some corner joints on the internal faces of walls, and it occurred for earthquake tests with scaling factor higher than 100%. The acceleration amplification recorded during earthquake tests was in the range from 1.11 to 2.23 for the second floor and from 1.60 to 3.10 for the roof (Fig. 22). The roof diaphragm response monitored during earthquake test with scaling factors up to 120% showed that the diaphragm can be assumed as rigid in its plane according to ASCE 7-10 [43] definition, i.e. the ratio between maximum diaphragm deflection (MDD) and average drift of vertical element (ADVE) was always less than 2 (Tab. 11).

As results of the shake table tests it is possible to confirm, in accordance to the latest American research, that the finishing material and box-building behaviour strongly modify the seismic response of lightweight constructions, by increasing significantly the lateral building stiffness and resistance. For this reason, the ELISSA mock-up exhibited a very modest anelastic behaviour without structural failures and residual deformations during shake table tests. In particular, the structural model assumed in the design phase was more flexible than the real behaviour of the complete construction mock-up. However, due their low fundamental periods, the lack of accuracy in the estimation of lateral stiffness does not affect significantly the seismic force evaluation for these structural typologies. As far as the damping ratio is concerned, the experimental results showed that the common seismic design assumption of damping ratio equal 5% can be considered certainly reasonable. Finally, for the adopted typologies of floors, the design assumption of in plane rigid diaphragm can be assumed, because the condition given by ASCE 7 [43] was widely fulfilled. Further details are available in Fiorino et al. [44].

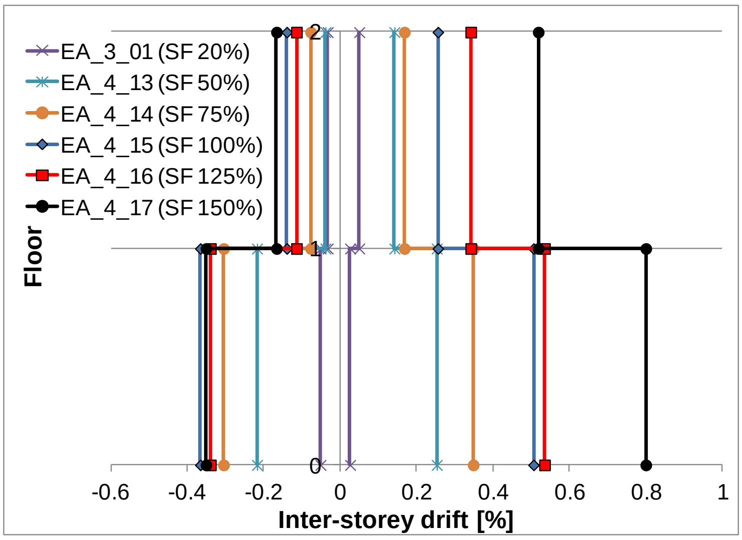


Figure 21. Results of earthquake tests in term of inter-storey drifts

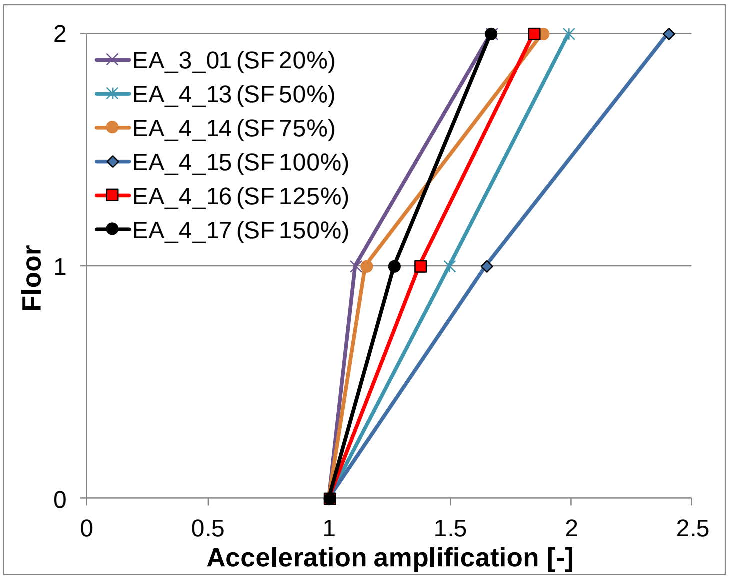


Figure 22. Results of earthquake tests in term of acceleration amplification

Table 11. Results of earthquake tests in term of diaphragm behaviour

Test label (SF)	MDD/ADVE
EA_2_01(5%), 2_02(10%), 2_03(15%), 2_04(20%), 2_05(25%), 2_06(30%), 2_07(40%), 2_08(50%)	0.025, 0.136, 0.106, 0.052, 0.060, 0.050, 0.048
EA_3_01(20%), 3_02(40%), 3_03(50%), 3_04(60%), 3_05(75%), 3_06(90%), 3_07(100%)	0.022, 0.028, 0.027, 0.045, 0.023, 0.026, 0.00.5
EA_3_08(50%)	0.057
EA_4_01(50%), 4_02(100%), 4_03(120%), 4_04(100%)	0.018, 0.021, 0.009, 0.016
EA_4_05(100%)	0.018
EA_4_06(50%)	0.034
EA_4_10(50%)	0.019

CONCLUSIONS

This paper presents the main findings of a European research projects (ELISSA) that have seen academia and industry working together to advance knowledge about the dynamic properties of lightweight steel systems in seismic area. The novelty of the research sits in the study of an industrial kit for modular housing, the Transformer system by Cocoon, which has been improved for seismic applications during the project thanks to the collaboration with the colossus Knauf and a number of SMEs. The full project of a demonstrative residential house has been developed and presented in this paper. The building was designed according to a typical European seismic intensity for areas with medium seismic hazard (PGA of 0.25g) with a behaviour factor equal to 3.0 and an over-strength factor equal to 1.8. The mechanical characterization of the main structural components and the evaluation of the global dynamic performance have been defined with an extensive experimental campaign, articulated in the 3 phases (micro, macro and shake table tests) and presented in this paper.

The micro scale tests performed on clinched steel-to-steel connections and connections between gypsum-based panels and steel profiles by means of ballistic nails allowed to characterize the shear behaviour in terms of strength and stiffness that are the main parameters for the prediction of the seismic response of sheathed shear walls. As conclusion, the suggested value for the characteristic shear strength under cyclic loads of the tested clinched steel-to-steel connections is 3.00 kN, whereas for tested panel-to-steel connections, the characteristic values of nominal shear strength under cyclic actions can be assumed equal to 0.80 and 4.30 kN for wall and panel connections, respectively.

The macro scale tests involved monotonic and cyclic tests of different configurations of shear walls laterally braced by gypsum boards connected to the CFS frame by ballistic nails. In particular, four full-scale walls were tested and the wall configurations were selected in order to investigate the effect of the type of loading, aspect ratio and finishing on the lateral/seismic wall response. The tests showed that the wall collapse always occurred for the failure of sheathing-to frame nailed connections. The experimental results revealed that the cyclic loads gave a reduction of wall lateral strength of 20%, whereas the variation of the aspect ratio from 1 to 0.5 resulted in an increase of strength of 35%. The presence of finishing material showed an increasing of strength of about 50%.

Shake table tests of the full building were performed. In particular, the building was subjected to white-noise tests and earthquake tests by applying the AQV 2009 Aquila (Central Italy) earthquake with scaling factors in the range from 5 to 100% (the life safety design level was obtained by a scaling factors of about 50%). The shake table tests confirmed, in accordance to the latest American research, that the finishing material strongly modify the behaviour of lightweight construction. Indeed, comparing the results of test on the bare structure with the one of the full building, can be concluded that the application of finishing resulted in a decreasing of the fundamental period of about 20% corresponding to an increase of the estimated lateral stiffness equal to about 3 times. The damping ratio measured before earthquake tests was in the range from 1% to 3% for the bare structure and from 1% to 2% for the complete construction, whereas higher values, in the range from 2% to 5%, were recorded after earthquake tests on the complete construction. Overall the building responded incredibly well to 100% earthquake, with a maximum inter-storey drift of 0.97% for 1st level and 0.58% for 2nd level, and with a negligible residual inter-storey drifts (under 0.06%). Moreover, according to the design assumptions, the diaphragms behaved as rigid in their plane. The damage was very small in both structural parts and finishing materials and was characterised by the presence of gypsum dust and small detachment of cover paper at some corner joints on the internal faces of walls observed only after earthquake tests with higher scaling factors. Therefore, methodologies for the prediction of the overall response of CFS buildings that are able to take into account the box building behaviour and the strengthen function of the non-structural materials should certainly be further investigated. In conclusion, it is possible to affirm that the ELISSA project presented in this paper, from the academic side advance the knowledge of the seismic performance of CFS building sheathed with gypsum based panels, opening the avenue for the improvement of the current seismic design prediction for CFS structures, and from the industrial side it defines a system that can be safely adopted in high seismic regions, and, as such, provides the possibility to extend the market of all the industries involved in the project, with consequent future industrial impact.

ACKNOWLEDGMENTS
The study presented in this paper is a part of the project "Energy Efficient LIghtweight-Sustainable-SAfe-Steel Construction" (Project acronym: ELISSA) coordinated by Prof. Raffaele Landolfo for the activities of the University of Naples “Federico II”. The project has received funding from the European Union Seventh Framework Programme (FP7/2007-2013) under grant agreement n° 609086. The authors would also like to thank the ELISSA consortium for the collaboration.

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